In cooperation with the
San Bernardino Valley Municipal Water District

Hydrology, Description of Computer Models, and Evaluation of Selected Water-Management Alternatives in the San Bernardino Area, California

U.S. Geological Survey Open-File Report 2005-1278

Pending release as USGS Professional Paper 1734

U.S. DEPARTMENT OF THE INTERIOR
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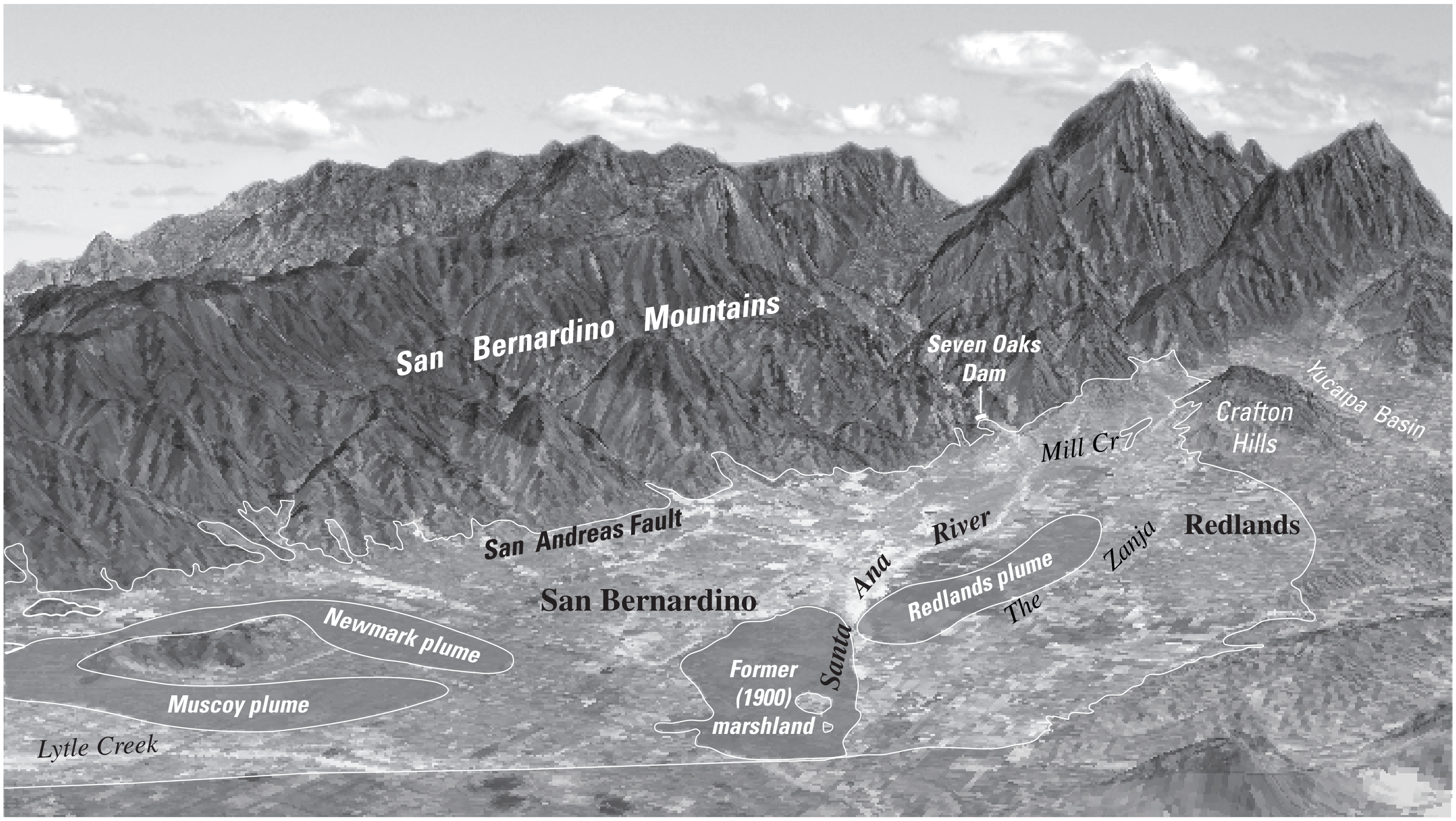

Cover. Oblique view showing the San Bernardino Mountains rising above the Bunker Hill and Lytle Creek basins. The area simulated with the ground-water flow model is outlined in white. Also shown are selected water-management issues including the former marshland, Seven Oaks Dam, and the Newmark, Muscoy, and Redlands plumes.

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By Wesley R. Danskin, Kelly R. McPherson, and Linda R. Woolfenden

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Conversions, Vertical Datum, Abbreviations, and Acronyms

Pound/SI

Multiply	By	To obtain
acre	4,047	square meter (m^2)
acre-foot (acre-ft)	1,233	cubic meter (m^3)
acre-foot per year (acre-ft/yr)	1,233	cubic meter per year (m^3/yr)
foot (ft)	0.3048	meter (m)
foot per day (ft/d)	0.3048	meter per day (m/d)
foot per foot (ft/ft)	1	meter per meter (m/m)
foot per minute (ft/min)	0.3048	meter per minute (m/min)
foot per second (ft/s)	0.1894	meter per second (m/s)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
foot squared per day (ft^2/d)	0.09290	meter squared per day (m^2/d)
inch (in.)	25.4	millimeter (mm)
inch per year (in/yr)	25.4	millimeter per year (mm/yr)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)

Temperature in degrees Fahrenheit (°F) may be converted to degrees Celsius (°C) as follows:

$$°C=(°F-32)/1.8$$

Vertical coordinate information is referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29).

Altitude, as used in this report, refers to distance above the vertical datum.

Transmissivity: The standard unit for transmissivity is cubic foot per day per square foot times foot of aquifer thickness [(ft^3/d)/ft^2]ft. In this report, the mathematically reduced form, foot squared per day (ft^2/d), is used for convenience.

Concentrations of chemical constituents in water are given either in milligrams per liter (mg/L) or micrograms per liter (µg/L).

Abbreviations and Acronyms

A^{Bot}	altitude of the bottom of the aquifer (L);
A^{Top}	altitude of the land surface (L);
B^{Low}	lower bound for a decision variable (units of the decision variable);
B^{Up}	upper bound for a decision variable (units of the decision variable);
$C^{ImpWater}_{iar,\,k}$	cost of imported water distributed to basin iar, during time period k ($\$/L^3/T$);
c^e	unit cost of electricity ($\$/L/L^3$);
c_i	concentration of fluoride in well i (M/L^3);
D	matrix of total drawdown at all control locations, at all time periods (L);
$D^{Target}_{o,\,to}$	target drawdown observed at control location o during time period to (L);
DBCP	dibromochloropropane
d^{Grad}_p	distance between the inside and outside control points for gradient pair p (L);
$d_{o,\,to}$	total drawdown from managed and unmanaged stresses and from initial and boundary conditions, at observation location o, during time period to (L);
e_i	efficiency of the well pump and motor for well i (decimal fraction);
G^{Target}_p	target gradient for gradient pair p (L/L);
GAMS	General Algebraic Modeling System
GIS	geographic information system
GPS	global positioning system
$H^{LimitLo}_{o,\,to}$	lower limit of hydraulic head at location o, during time period to (L);
$H^{LimitUp}_{o,\,to}$	upper limit of hydraulic head at location o, during time period to (L);
H^{Man}	matrix of managed hydraulic heads (L);
$H^{Initial}$	matrix of initial hydraulic heads (L);
h	hydraulic head (L);
$h_{o,\,to,\,s,\,ta}$	simulated hydraulic head observed at control location o, at the end of time period to, resulting from recharge or discharge (stress) s, applied at time ta (L);
h^{GradIn}_p	hydraulic head on the inside of gradient pair p (L);
$h^{GradOut}_p$	hydraulic head on the outside of gradient pair p (L);
$h_{Heap,\,k}$	simulated hydraulic head for the ground-water flow model cell containing the Heap well (row 70, column 79, layer 1) for time period k (L);
$h^{Ibc}_{o,\,to}$	hydraulic head resulting from initial and boundary conditions, observed at control location o, at the end of time period to (L);
h^{Man}	managed hydraulic head (L);
$h^{Initial}_o$	initial hydraulic head, observed at control location o (L);
h_{Heap}	simulated hydraulic head for the model cell containing the Heap well (row 70, column 79, layer 1), referenced to mean sea level (L);
InSAR	interferometric synthetic aperture radar;
i	location index;
iar	index for a specific artificial-recharge basin;
K_p	horizontal hydraulic conductivity between inside and outside control points for gradient pair p (L/T);
K_z	vertical hydraulic conductivity (L/T);

k	time index;
$L^{Initial}_{i}$	initial lift at well i (L);
L^{Man}_{i}	total lift at well i resulting from all managed stresses, in feet;
$L^{S}_{i,\ j}$	lift at well i caused by recharge or discharge (stress) at location j (L);
L^{Total}_{i}	total lift at well i (L);
ln	natural logarithm;
log	logarithm;
M_L	Richter or local magnitude for an earthquake (dimensionless);
MCM	middle confining member;
MODFLOW	ground-water flow model;
MODMAN	computer program used with MODFLOW;
MINOS	optimization software;
MPS	input dataset;
na	not applicable
nar	number of artificial-recharge basins;
ndv	number of decision variables (managed stresses) having response functions;
nio	
nol	number of observation locations;
ntp	number of time periods simulated in the management horizon;
nw	number of wells;
ny	number of years in a water-management period;
o	index indicating the location where simulated hydraulic head is observed, represented in the ground-water flow model by a single model cell;
PCE	perchloroethylene;
p^{e}_{p}	effective porosity between inside and outside control points for gradient pair p (dimensionless);
$P^{ImpWater}_{k}$	percentage deliverable imported water compared to the maximum entitlement, for calendar year k (percent);
P^{SARRO}_{k}	percentage runoff in the Santa Ana River measured at USGS station 11051501 for calendar year k, compared to the long-term average annual runoff (discharge) at the same station for 1928–98 (percent);
$Q^{ArtRech}_{iar,\ k}$	quantity of water distributed to artificial-recharge basin iar during time period k (L^3/T);
$Q^{GagedRO}_{i,\ k}$	gaged discharge in stream i during time period k (L^3/T);
$Q^{GagedRO}_{SAR,\ 1928–98}$	average gaged discharge in the Santa Ana River during 1928–98 (L^3/T);
$Q^{ImportedWater}_{i,\ k}$	imported water for area i during time period k (L^3/T);
$Q^{Pump}_{i,\ k}$	pumpage for well i during time period k (L^3/T);
Q^{Pump}_{Total}	total pumpage (L^3/T);
Q^{PumpNP}_{k}	total non-plaintiff pumpage for calendar year k (L^3/T);
Q^{PumpP}_{k}	total plaintiff pumpage for calendar year k (L^3/T);
$Q^{RisingGW}_{WarmCreek,\ k}$	ground water rising into Warm Creek during year k (L^3/T);
$Q^{UngagedRO}_{i,\ k}$	ungaged runoff from area i during time period k (L^3/T);
$Q^{UngagedRO}_{i,\ 1928–98}$	average ungaged runoff from area i during 1928–98 (L^3/T);
$Q^{Underflow}_{i,\ k}$	underflow for area i during time period k (L^3/T);
Q_z	vertical leakance (L/T);
Q	matrix of managed stresses (L^3/T);
$q_{s,\ ta}$	managed stress s, represented in the ground-water flow model by recharge or discharge for a model cell or group of model cells, observed at the end of time period ta (L^3/T);

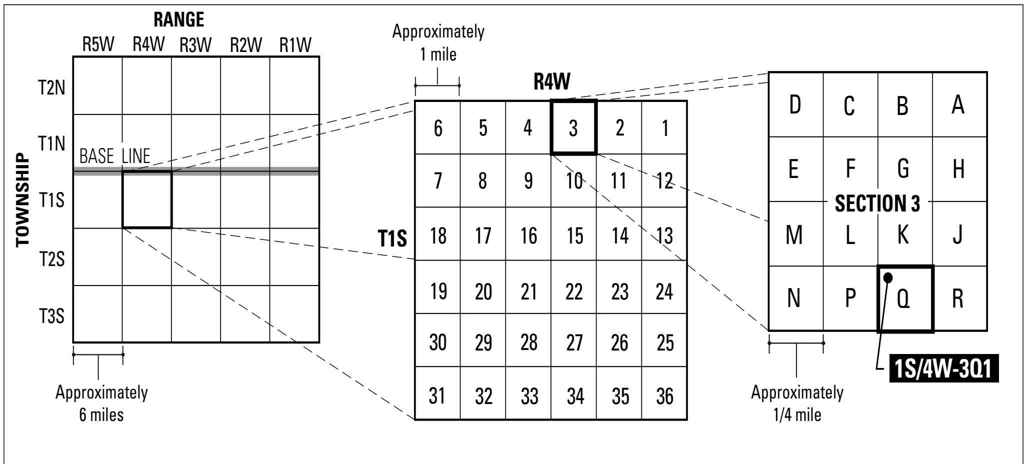
R	matrix of unit responses, at all control locations, at all time periods ($L/L^3/T$);
R^{Ibc}	matrix of responses, at all control locations, at all time periods, resulting from initial and boundary conditions and from all unmanaged stresses (L);
RHS	right-hand side;
$r^{Ibc}_{o,\, to}$	response of simulated hydraulic head from initial and boundary conditions and from all unmanaged stresses, observed at control location o, at the end of stress period to (L);
$r_{o,\, s}$	unit drawdown response observed at control location o that results from a unit stress applied at location s ($L/L^3/T$);
$rr_{iar,\, k}$	recharge rate for artificial-recharge basin iar during time period k (decimal fraction);
r^2	coefficient of determination for a regression equation;
S	storage coefficient (dimensionless);
STR1, STR2	streamflow routing package;
SWP	California state water project;
s	index indicating a managed stress;
T	transmissivity (L^2/T);
TCE	trichloroethylene;
t	time (T);
ta	index indicating the time period when the managed stress is applied, represented in the ground-water flow model by a pumping period;
to	index indicating the time period when the response is observed, calculated in the ground-water flow model at the end of a pumping period;
$U_{SandCanyon,\, k}$	underflow in the vicinity of Sand Canyon for calendar year k (L^3/T);
$U_{SanTimoteo,\, k}$	underflow in the vicinity of San Timoteo Canyon for calendar year k (L^3/T);
$U_{SAR,\, k}$	underflow across the San Jacinto fault near the Santa Ana River for calendar year k (L^3/T);
UCM	upper confining member;
UTM	Universal Transferse Mercator coordinate system;
UWB	upper water bearing;
V^{Target}_{p}	target velocity between inside and outside control points for gradient pair p (L/T);
W	combination of sources and sinks (L/T);
Wl_{Heap}	water-level altitude in the Heap well (1S/4W–3Q1) (L);
x, y, z	cartesian coordinates (L);
Z	objective (\$, L^3/T).

Organizations

SBVMWD	San Bernardino Valley Municipal Water District
SBVWCD	San Bernardino Valley Water Conservation District
USEPA	U.S. Environmental Protection Agency
USGS	U.S. Geological Survey

Well-Numbering System

Wells are identified and numbered according to their location in the rectangular system for the subdivision of public lands. Identification consists of the township number, north or south; the range number, east or west; and the section number. Each section is divided into sixteen 40-acre tracts lettered consecutively (except I and O), beginning with "A" in the northeast corner of the section and progressing in a sinusoidal manner to "R" in the southeast corner. Within the 40-acre tract, wells are sequentially numbered in the order they are inventoried. The final letter refers to the base line and meridian. In California, there are three base lines and meridians; Humboldt (H), Mount Diablo (M), and San Bernardino (S). All wells in the study area are referenced to the San Bernardino base line and meridian (S). Well numbers consist of 15 characters and follow the format 001S004W-003Q001. In this report, well numbers are abbreviated and written 1S/4W-3Q1. Wells in the same township and range are referred to only by their section designation, -3Q1. The following diagram shows how the number for well 1S/4W-3Q1 is derived.



Well-numbering diagram

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"And it never failed that during the dry years the people forgot about the rich years, and during the wet years they lost all memory of the dry years. It was always that way." John Steinbeck, *East of Eden*

Abstract

The San Bernardino area of southern California has complex water-management issues. As an aid to local water managers, this report provides an integrated analysis of the surface-water and ground-water systems, documents ground-water flow and constrained optimization models, and provides seven examples using the models to better understand and manage water resources of the area. As an aid to investigators and water managers in other areas, this report provides an expanded description of constrained optimization techniques and how to use them to better understand the local hydrogeology and to evaluate inter-related water-management problems.

In this report, the hydrology of the San Bernardino area, defined as the Bunker Hill and Lytle Creek basins, is described and quantified for calendar years 1945–98. The major components of the surface-water system are identified, and a routing diagram of flow through these components is provided. Annual surface-water inflow and outflow for the area are tabulated using gaged measurements and estimated values derived from linear-regression equations. Average inflow for the 54-year period (1945–98) was 146,452 acre-feet per year; average outflow was 67,931 acre-feet per year. The probability of exceedance for annual surface-water inflow is calculated using a Log Pearson Type III analysis. Cumulative surface-water inflow and outflow and ground-water-level measurements indicate that the relation between the surface-water system and the ground-water system changed in about 1951, in about 1979, and again in about 1992. Higher ground-water levels prior to 1951 and between 1979 and 1992 induced ground-water discharge to Warm Creek. This discharge was quantified using streamflow measurements and can be estimated for other time periods using ground-water levels from a monitoring well (1S/4W–3Q1) and a logarithmic-regression equation. Annual wastewater discharge from the area is tabulated for the major sewage and power-plant facilities.

The ground-water system consists of a valley-fill aquifer and a much less permeable bedrock aquifer. The valley-fill aquifer, which is the focus of this study, is composed primarily of highly transmissive unconsolidated and poorly-consolidated deposits. The bedrock aquifer is composed of faulted and fractured igneous and metamorphic rock. The valley-fill aquifer is underlain by the bedrock aquifer and and is bounded laterally by the bedrock aquifer and by faults with varying capabilities to transmit ground water. Some underflow occurs across faults in the valley-fill sediment, particularly beneath the Santa Ana River. Essentially no underflow occurs from the surrounding bedrock. Hydrogeologic units were defined for the valley-fill aquifer using driller's logs, geophysical logs, and hydrographs from multiple-depth piezometers. These units are shown on a detailed hydrogeologic section constructed along Waterman Canyon Creek. A large-scale aquifer test demonstrated the continuity of the hydrogeologic units and their hydraulic properties in the center of the Bunker Hill basin. Gross annual pumpage from the valley-fill aquifer for 1945–98 was compiled from reported and estimated data, and then used to estimate extraction from the upper and lower layers of the valley-fill aquifer and return flow to the upper layer. Annual values of the major components of recharge and discharge for the valley-fill aquifer are calculated for 1945–98. Average recharge occurs primarily from gaged streamflow (67 percent), ungaged mountain-front runoff (9 percent), and pumpage return flow (16 percent); average discharge occurs primarily as pumpage (88 percent).

Computer models, including a ground-water flow model and a constrained optimization model, are described. The ground-water flow model includes the Bunker Hill and Lytle Creek basins and simulates three-dimensional ground-water flow in the valley-fill aquifer using finite-difference techniques. The model consists of an upper layer representing the upper unconfined/semi-confined hydrogeologic unit and a lower layer representing a combination of several lower confined hydrogeologic units. The vertical connection between the model layers is approximated by Darcian flow. The flow-impeding effect of faults within the valley-fill aquifer is simulated by a horizontal flow-barrier package. The model also includes a streamflow-routing package that simulates the interaction of a complex network of streams with the valley-fill aquifer. Calibration of the flow model was for 1945–98.

The constrained optimization model uses linear programming to calculate the minimum quantity of recharge from imported water and pumpage from wells necessary to solve various water-management problems. A description of linear-programming techniques and a simplified example problem are provided. The optimal quantity of recharge or pumpage is determined by their availability and by constraints on ground-water levels and ground-water quality. The response of ground-water levels to recharge and pumpage is calculated by the ground-water flow model. The mathematically optimal solutions derived from the optimization model, in concert with field data and hydrogeologic concepts, can be used to guide water-management decisions.

Selected water-management alternatives for the San Bernardino area were evaluated with the aid of the ground-water flow and constrained optimization models. Seven scenarios were designed to answer specific water-management questions and to demonstrate key hydrogeologic characteristics of the area. A 32-year simulation period, 1999–2030, with annual values of recharge and pumpage, was used for each scenario. The scenarios include: (1) average historical conditions; (2) annually varying historical conditions; (3) additional artificial recharge provided by construction of Seven Oaks Dam; (4) increased ground-water pumpage; (5) optimal pumpage from barrier wells designed to prevent further spread of contamination from the Newmark U.S. Environmental Protection Agency superfund site; (6) optimal pumpage needed to control ground-water levels in an area with potential liquefaction and land subsidence; and (7) optimal recharge and pumpage to control ground-water levels and to prevent migration of the Newmark contamination.

Results of the evaluation include the following conclusions. Additional pumpage in the vicinity of the former marshland is needed to prevent a reoccurrence of dangerously high ground-water levels similar to those experienced in 1945 and 1980. High ground-water levels are a water-management concern because they indicate soil is saturated near land surface and is susceptible to liquefaction during an earthquake. The optimal location of additional pumpage is near Warm Creek in an area of historically rising ground water. The high ground-water levels occur primarily during short periods following abundant natural recharge. About 15,000 acre-feet per year of additional, areally distributed pumpage are needed to control the high ground-water levels.

Demand for water in the San Bernardino area is projected to increase during the next 25 years by as much as 50,000 acre-feet per year. Part of this increased demand can be met by additional pumpage needed to prevent a rise in ground-water levels (15,000 acre-feet per year) and part by increased local supply (3,000 acre-feet per year) resulting from construction of a conservation pool behind Seven Oaks Dam. As much as 70,000 acre-feet of additional pumpage is theoretically available from existing wells using excess pumping capacity. However, additional pumpage greater than about 15,000 acre-feet per year likely will result in a longterm decline in ground-water levels such as occurred during the 1960’s, a decline which prompted land subsidence. To meet future demand for water in excess of about 15,000 acre-feet per year and to prevent a reoccurrence of land subsidence, imported water probably will need to be used, either for direct delivery or for recharge of the ground-water system.

Much of the recharge to the valley-fill aquifer occurs during years with unusually abundant runoff, which occurs on average about once every 5 to 10 years. Maintaining and enhancing capabilities to artificially recharge native runoff are likely to be necessary to meet increased demand for water from the valley-fill aquifer. Since 1945, significant fluctuations in ground-water storage have become common because of the abundant, but highly variable, recharge combined with relatively constant ground-water pumpage. Annual fluctuations in ground-water storage from 50,000 to 100,000 acre-feet are common. Cumulative fluctuations in ground-water storage greater than 500,000 acre-feet in a 10-year period also are common, but this magnitude will not significantly affect the availability of ground water, as long as historic recharge capacities are maintained or enhanced.

Hydraulic control of the Newmark contamination site is unlikely to occur using only the five planned extraction (barrier) wells; another four wells may be needed, each with a capacity of about 3.5 cubic feet per second. Without additional extraction wells, contaminated ground water tends to migrate around the barrier wells, especially to the west. Minimum total pumpage at nine barrier well sites, located across the leading edge of the contamination, needs to be at least 14,000 acre-feet per year, based on results from the optimization model.

Control of maximum and minimum ground-water levels in the vicinity of the former marshland does not require significant additional recharge of imported water, but does require additional pumpage. Additional pumpage from as many as 29 potential production wells located along the proposed eastern and southern extensions of the Baseline feeder pipeline is unlikely to sufficiently control high ground-water levels. Additional pumpage needs to be areally distributed in the vicinity of Warm Creek, just north of the San Jacinto fault. Extraction needs to occur from the highly permeable deposits of the upper water-bearing unit to prevent an upward hydraulic gradient and from the less permeable near-surface deposits that remain saturated even as hydraulic head in the underlying production zone is declining.

The high probability of a major earthquake on either the San Jacinto fault or San Andreas fault in the San Bernardino area makes control of high ground-water levels a pressing economic concern. Significant mitigation of this threat by additional extraction of ground water is possible, especially if a use can be found for the surplus water. Reoccurrence of land subsidence is a continuing concern and can be monitored with multiple-depth piezometers, extensometers, and satellite-borne interferometry, particularly if pumpage near the former marshland is increased. Contamination of the valley-fill aquifer is widespread both areally and vertically. Plans for clean up will be aided by continued mapping of the hydrogeologic units, which strongly influence ground-water flow paths. The large magnitude of proposed ground-water extraction to clean up several areas of contamination suggests that these plans need to be coordinated with plans to prevent liquefaction and land subsidence.

Introduction

During the past 100 years, water management in the San Bernardino area of southern California (*figs. 1* and *2*) has become increasingly complex. Water purveyors in this area, as in other parts of the world, have applied a variety of techniques to solve their water-supply and water-quality problems. They have adjusted ground-water pumpage both areally and seasonally; enhanced recharge of native water along streams; redistributed water with canals and aqueducts; artificially recharged aquifers using imported water; treated, blended, and reused water; and employed various forms of water sales and exchanges. Although most water purveyors rely on conjunctive use of surface and ground waters, seldom is there optimal management of the entire system. Much of the management is by trial and error in response to crisis situations, such as floods, droughts, and contamination problems. Often, management options are limited by relatively inflexible court-imposed plans that are the outgrowth of litigation between competing purveyors or water-use interests.

A comprehensive understanding of the hydrology of the San Bernardino area combined with prudent use of ground-water flow and constrained optimization models can aid in the effective management of such a complex and changing system. A ground-water flow model incorporates a spatial and temporal discretization of recharge, discharge, and aquifer characteristics and can simulate the historical operation of the ground-water system and the effect of proposed water-management decisions on ground-water levels, recharge, and discharge. Results of such simulations commonly provide an improved understanding of a ground-water system as well as specific guidance in making future water-management decisions. A constrained optimization model can combine predicted results from a ground-water flow model with other water-management issues, such as cost of operations. Results of the optimization model identify the mathematically optimal solution to a water-management question that can involve literally hundreds or thousands of ground-water-level, water-quality, and economic constraints. Although constrained optimization techniques have been used in a variety of technical fields for more than 50 years, applications to surface-water/ground-water systems have been limited (Bachmat and others, 1980; Gorelick, 1983; Rogers and Fiering, 1986; B.J. Wagner, 1995). Most of these applications have been academic studies applied to hypothetical basins; the few applications to real basins typically have not involved a close interaction with the actual individuals and agencies responsible for implementing water-management decisions.

This study was initiated with two major objectives. The first objective was to analyze the water resources of the San Bernardino area and provide quantitative guidance to local water purveyors on how to better manage their water resources. Achieving this objective required a close interaction with the more than 20 local purveyors and regulatory agencies. Hydrologic understanding and computer models developed as part of this study will be used by the purveyors to enhance their ability to analyze and manage future water-management problems. The second major objective of this study was to demonstrate the successful use of optimization techniques in a basin with a large number of water problems, purveyors, and vested interests. Major components of the study and flow of information between the components is shown diagrammatically in *figure 3*.

Purpose and Scope

The purpose of this report is to present the hydrology of the San Bernardino area, California; to describe ground-water flow and constrained optimization models that were developed for analysis of local water-resource operations; and to evaluate selected water-management alternatives using results from the computer models and an improved understanding of the hydrologic system.

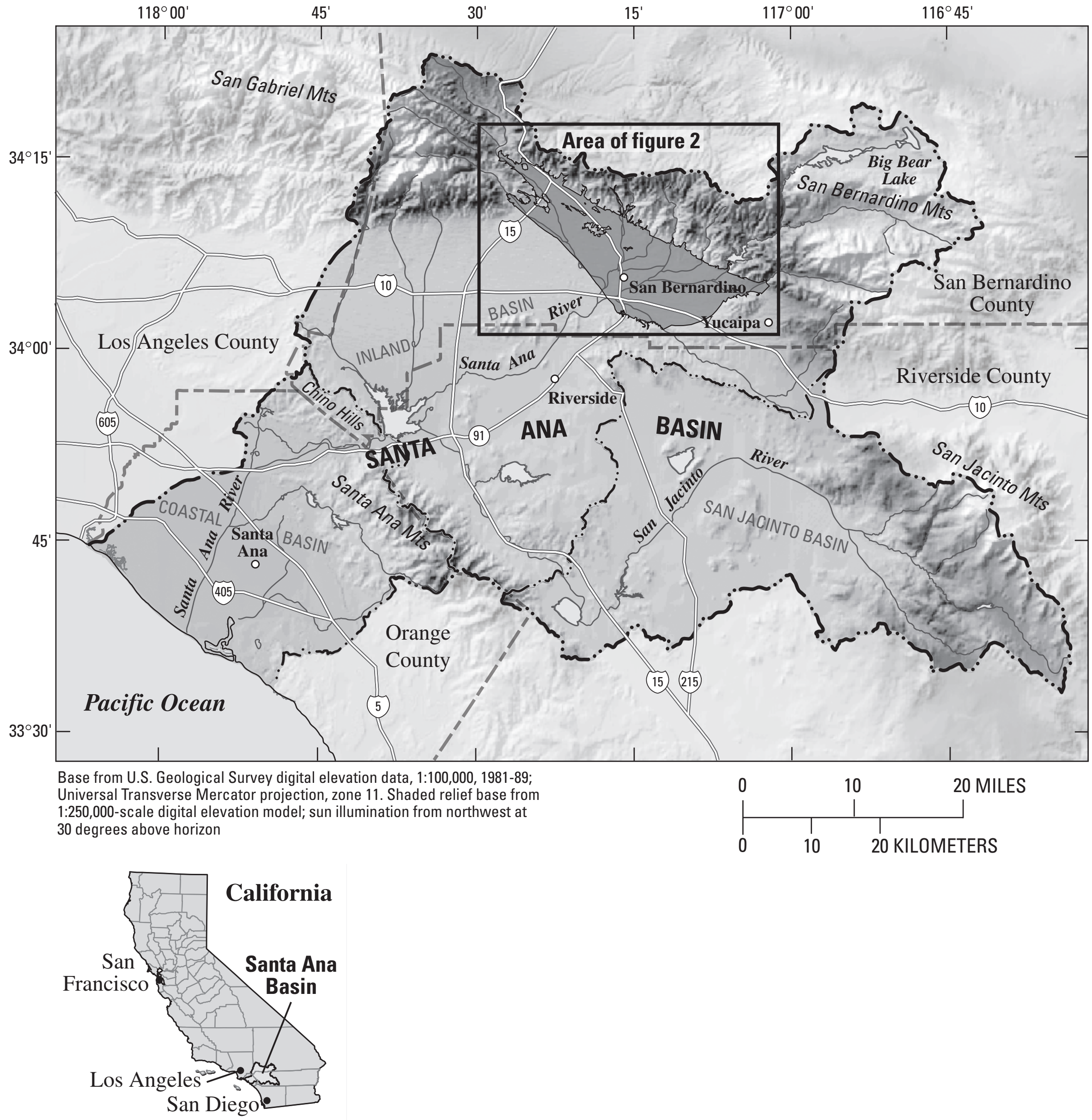
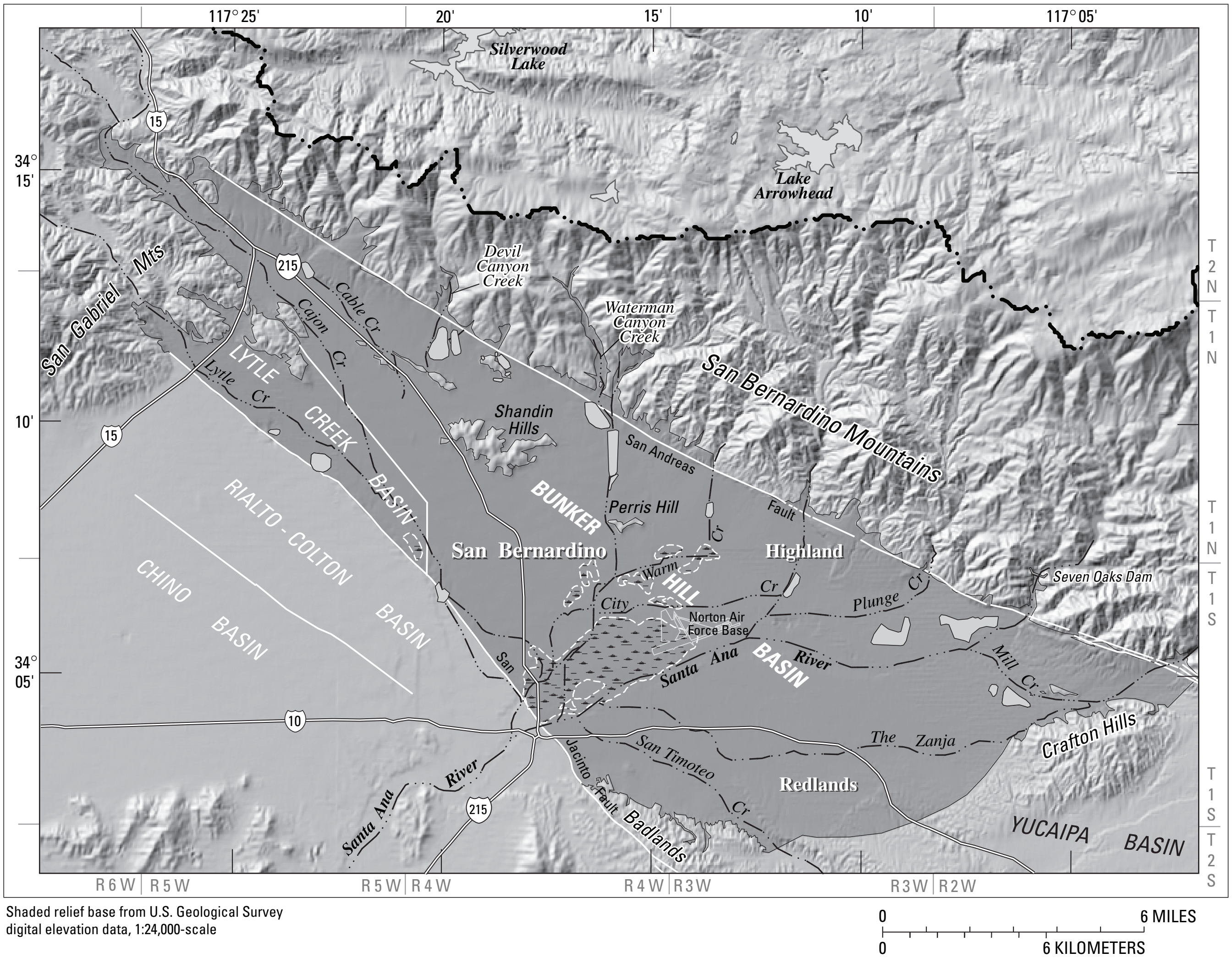


Figure 1. Location of the San Bernardino area in the Santa Ana River drainage basin, southern California.



EXPLANATION

Basin boundary—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray comprise the San Bernardino area, as defined in this report

Fault or ground-water barrier—May be concealed or approximately located

Former marshland

Artificial-recharge basin

Boundary of Santa Ana River drainage basin

Figure 2. Geographic setting of the San Bernardino area, California.

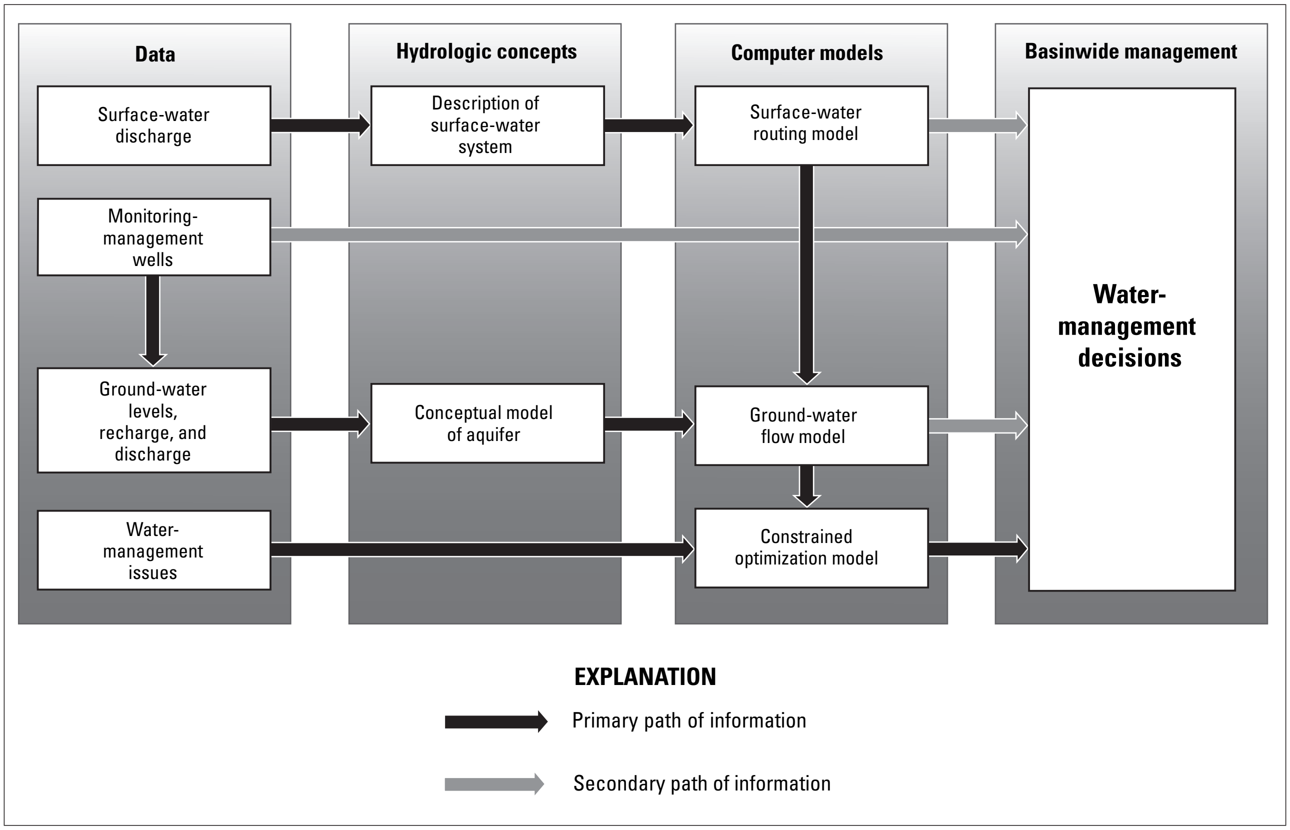


Figure 3. Major components and flow of information in this study of the San Bernardino area, California.

The scope of this report includes the Bunker Hill and Lytle Creek basins that compose the San Bernardino area (*fig. 2*). The hydrologic system consists of a surface-water system and a ground-water system. The surface-water system includes stream courses, canals, ditches, and reservoirs; the ground-water system consists of the valley-fill aquifer, which includes all saturated materials from land surface down to igneous and metamorphic basement rocks.

This report includes analysis of data from 1871 through 1998, and review of publications from 1888 through 2000. Hydrologic interpretations were based primarily on data for calendar years 1945–98 for both the surface-water and ground-water systems. Data for calendar years 1928–98 were used whenever possible, in particular for analysis of precipitation and surface-water runoff. Calendar years are used throughout this report, unless otherwise stated, primarily because they had been adopted as part of adjudication of the area for reporting pumpage and surface-water allocations.

Background information for this report included a thorough literature review of published and unpublished geologic and hydrologic reports, streamflow records, surface-water routing diagrams, water-quality data, ground-water-level measurements, pumpage data, aquifer-test data, driller's logs, geophysical logs, hydrogeologic sections, and reports from local water agencies and consultants. Data collected include ground-water-level measurements, in particular from multiple-depth well sites. Data from these sites, which were equipped with pressure transducers and data loggers to monitor ground-water-level changes every 60 minutes, were used to refine concepts of ground-water flow in the valley-fill aquifer and to provide real-time data via satellite. A 50-foot-deep well (1S/4W-22D7) was drilled to monitor ground-water-level fluctuations in saturated near-surface deposits that are susceptible to liquefaction during an earthquake. An areally extensive aquifer test was conducted to document the hydraulic properties of the aquifer. Map data were organized and interpreted using a geographic information system (GIS) developed for this project.

A previously developed numerical ground-water flow model by Hardt and Hutchinson (1980) was revised in order to simulate the ground-water flow system more accurately and to include key recharge-discharge relations explicitly in the model. Both previous and revised models simulate transient ground-water flow in the valley-fill aquifer of the Bunker Hill and Lytle Creek basins, using two model layers and annual stress periods. Revisions include use of a more flexible numerical code (McDonald and Harbaugh, 1988); interpolation of prior parameter data sets to conform to the new discretization; conversion from lumped, average values of recharge and discharge to discrete, annually varying values that represent different physical processes; extension of the historical simulation to include calendar years 1945–98; and addition of a streamflow-routing package (Prudic, 1989). The streamflow-routing package permits simulation of interaction between the surface-water and ground-water systems and includes an accounting of flow in the streams. All model parameters were included in the GIS.

Development of a constrained optimization model required identifying the major water-management issues in the San Bernardino area. These issues involve water supply, ground-water levels, ground-water quality, and cost of supplying water. The issues are formulated as mathematical equations and are combined with information from the ground-water flow model to form the constrained optimization model. This model, which uses linear-programming techniques, can be used to define the mathematically optimal solution to a variety of water-management questions, such as "What is the minimum pumpage necessary to dewater the high-ground-water area near downtown San Bernardino?" Manageable items (decision variables) in the optimization model include recharge of imported water, additional pumpage near areas of contaminated ground water, and additional pumpage in an area with unacceptably high ground-water levels. Constraint equations set limits on: quantities of recharge and pumpage; ground-water gradients near areas of ground-water contamination; ground-water levels in different areas of both basins; and costs of distributing surface water or pumping ground water.

Selected water-management alternatives for the San Bernardino area were evaluated using the ground-water flow and optimization models. To aid in the evaluation process and to provide the necessary quantitative information, six scenarios were developed and simulated. Each scenario was designed to answer specific water-management questions and to demonstrate key hydrogeologic characteristics of the basins. A 32-year simulation period, 1999–2030, and annual values of recharge and pumpage were used for each scenario. The scenarios include simulation of average future conditions with no change in basin management, annual variations in recharge and pumpage, increased recharge and pumpage to meet increased demand, and redistribution of pumpage to control ground-water gradients and levels. The primary water-management objectives were to satisfy local demand, to prevent further migration of contaminated ground water from the Newmark superfund site, and to reduce the risk of liquefaction of saturated materials near the land surface during an earthquake. Simulated results from the six scenarios were combined with other hydrologic data and interpretations in order to evaluate possible changes in water management necessary to meet each objective.

Description of Study Area

The San Bernardino area is a semiarid inland valley of about 120 square miles in southwestern San Bernardino County, about 60 miles east of Los Angeles, in the upper part of the Santa Ana River drainage basin (*fig. 1*). The San Bernardino area was defined by Dutcher and Garrett (1963) as a northwest-trending area between the San Andreas and San Jacinto faults (*fig. 1*). The area is bordered on the northwest by the San Gabriel Mountains, on the northeast by the San Bernardino Mountains, on the south by the badlands and Crafton Hills, and on the southwest by a low, east-facing escarpment of the San Jacinto fault. Alluvial fans extend from the base of the mountains and hills that surround the valley and coalesce to form a broad, sloping alluvial plain in the central part of the valley (*fig. 4*). Altitude ranges from about 1,000 feet (ft) near the city of San Bernardino to more than 10,000 ft in the San Bernardino Mountains. In this report, the San Bernardino area is defined as the Bunker Hill and Lytle Creek basins (*fig. 2*), which also is the area simulated by the ground-water flow model described in this report.

The generalized geology of the San Bernardino area is shown in *figure 5*. Unconsolidated material that fills the basins includes river-channel deposits and younger alluvium of Holocene age, and older alluvium of Pleistocene age. Undifferentiated deposits of younger and older alluvium are present near Redlands and north of Perris Hill. Sedimentary rocks of Quaternary and Tertiary age crop out in the badlands mostly as the San Timoteo Formation, and north of the San Andreas fault mostly as the Potato Sandstone. A basement complex of Precambrian to Tertiary igneous and metamorphic rocks underlies the basin fill and crops out in the surrounding mountain ranges (Dutcher and Garrett, 1963). The San Bernardino area, which is located along the San Andreas fault zone, contains numerous sub-parallel faults including the San Andreas and San Jacinto faults (*fig. 5*). The area is tectonically active as indicated by numerous, nearby earthquakes—many with a magnitude greater than $5M_L$ (Richter or local magnitude) (Yerkes, 1985). During 1992, for example, the nearby Landers and Big Bear earthquakes measured $7.3M_L$ and $6.1M_L$, respectively (Roeloffs and others, 1995).

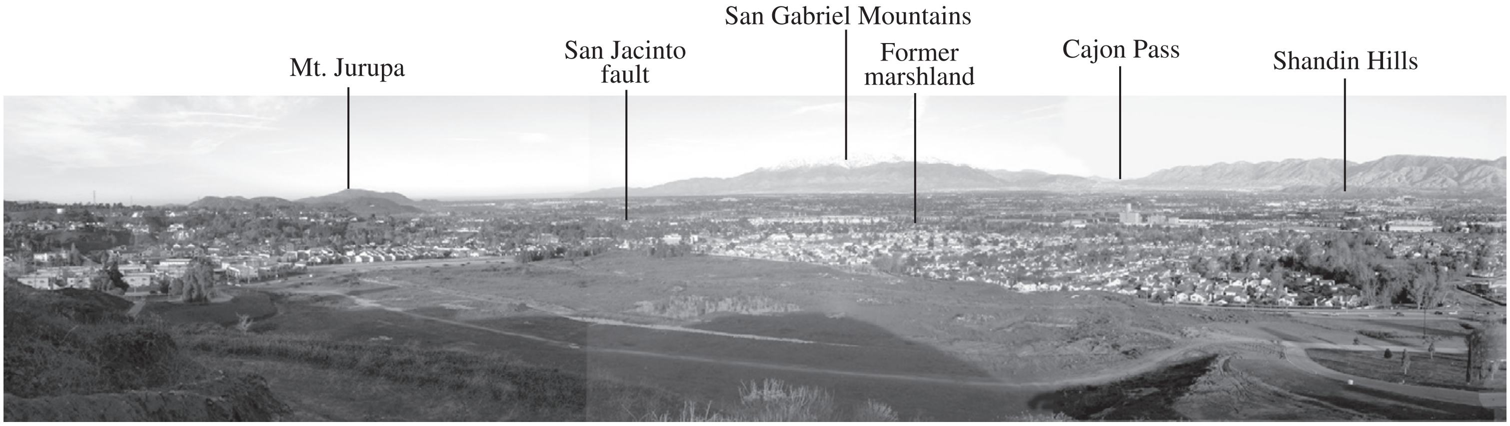


Figure 4. View of the San Bernardino area, California, looking northwest across the Bunker Hill basin, 2004. Photo by W. R. Danskin.

Climate in the San Bernardino area is characterized by relatively warm, dry summers and cool, wet winters. Temperatures range from daytime highs of about 90°F in summer to night-time lows of about 40°F in winter. Precipitation is nearly always in the form of rain in the lower elevations and mostly in the form of snow above an altitude of about 6,000 ft in the surrounding San Bernardino and San Gabriel Mountains. Mean annual precipitation ranges from less than about 15 inches over much of the valley floor, to about 20 inches along the base of the San Bernardino Mountains, to more than 30 inches along the crest of the mountains (*fig. 6*). Precipitation recorded at the city of San Bernardino from 1871 to 1998 indicates that a period of below-average precipitation can last more than 30 years, such as a dry period that extended from 1947 to 1977 (*fig. 7*). Periods of above-average precipitation have tended to be shorter with a few, very wet years.

The San Bernardino area is located in the upper part of the Santa Ana River drainage basin (*fig. 1*). Runoff, particularly from the San Bernardino Mountains, flows in several small streams, Lytle Creek, and the Santa Ana River (*fig. 2*). The streams merge mostly within the San Bernardino area, flow southwest through Riverside and Orange Counties, and eventually empty into the ocean (*fig. 1*). Seepage from the streams replenishes the valley-fill aquifer, which provides most of the water used within the San Bernardino area.

Population and land use in the San Bernardino area have changed significantly since its development as an important citrus-growing area of southern California in the early 1900s. Beginning in about 1940, the urban population began to steadily increase as a result of increased growth in the defense industry and an increased awareness of the southern Californian climate and lifestyle as depicted in Hollywood films (California Department of Water Resources, 1971, fig. 19). The progressive conversion of vacant and agricultural land to urban land is shown in *figure 8*. In 1980, population of the San Bernardino area exceeded 225,000, an increase of about 30 percent since 1960. In the year 2000, the population was about 320,000 and is projected to increase to more than 380,000 by the year 2020 (California Department of Water Resources, 1986, table 4).

A land-use survey of the upper Santa Ana River drainage area was conducted by the California Department of Water Resources (1985). The most extensive land use in the San Bernardino area in 1984 was urban, which accounted for approximately 50 percent of the land area. Unused land accounted for approximately 25 percent; citrus agriculture approximately 20 percent; and other uses approximately 5 percent. In the upper Santa Ana River drainage area, urban land use increased 23 percent between 1975 and 1984; acreage used for agricultural purposes decreased approximately 31 percent during the same period (California Department of Water Resources, 1985, p. 9). A similar pattern of changing land use has occurred in the San Bernardino area (Duell and Schroeder, 1989, p. 17).

The combination of increasing population, changing land and water use, and frequent tectonic activity has resulted in a large number of complex water-management issues. Local water purveyors are concerned about the reliability of the water supply and the quality of ground water. Near the base of the San Bernardino Mountains, ground-water levels rise and fall dramatically in response to changes in recharge from streams. During times of drought and increased pumping, ground-water levels fall as much as 200 ft and limit the ability of water purveyors to supply sufficient ground water to meet demand.

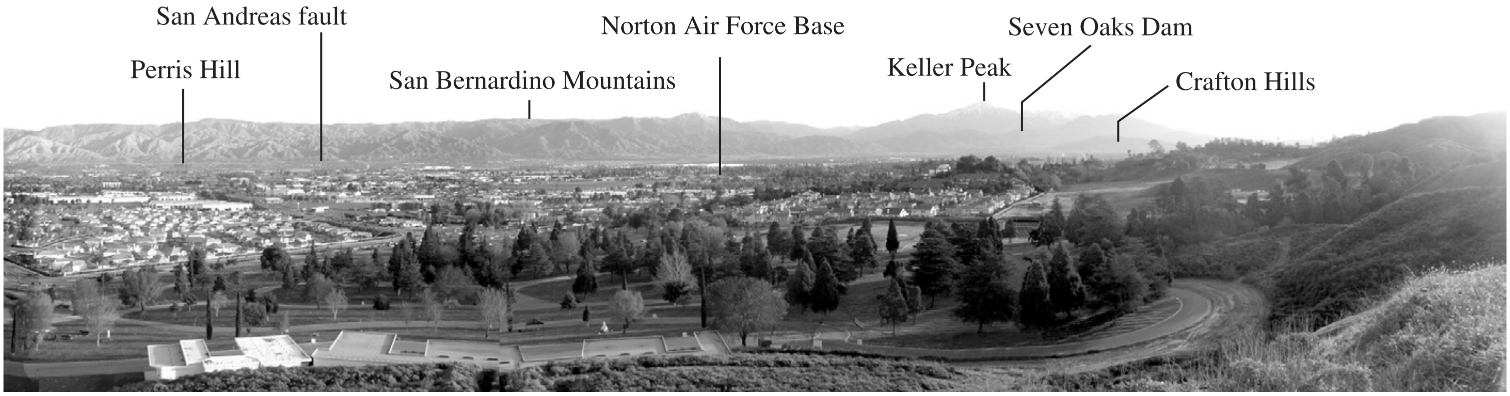


Figure 4.—Continued.

In the area where the Santa Ana River crosses the San Jacinto fault (*fig. 2*), ground-water levels also fluctuate though not as dramatically. During a period of extensive ground-water extractions from 1950 to 1970, ground-water levels fell as much as 100 ft and induced land subsidence of as much as 1 ft (Miller and Singer, 1971). After 1970, both natural and artificial recharge increased so that by 1980, ground-water levels had risen to within a few ft of land surface (Hardt and Freckleton, 1987). This increase in hydrostatic pressure in a former marshland caused a variety of problems, including buckled foundations, damaged flood-control structures, and severed utility lines (Danskin and Freckleton, 1992). The increase in pressure also creates the potential for liquefaction of near-surface earth materials as a result of ground-shaking during a severe earthquake (Matti and Carson, 1991).

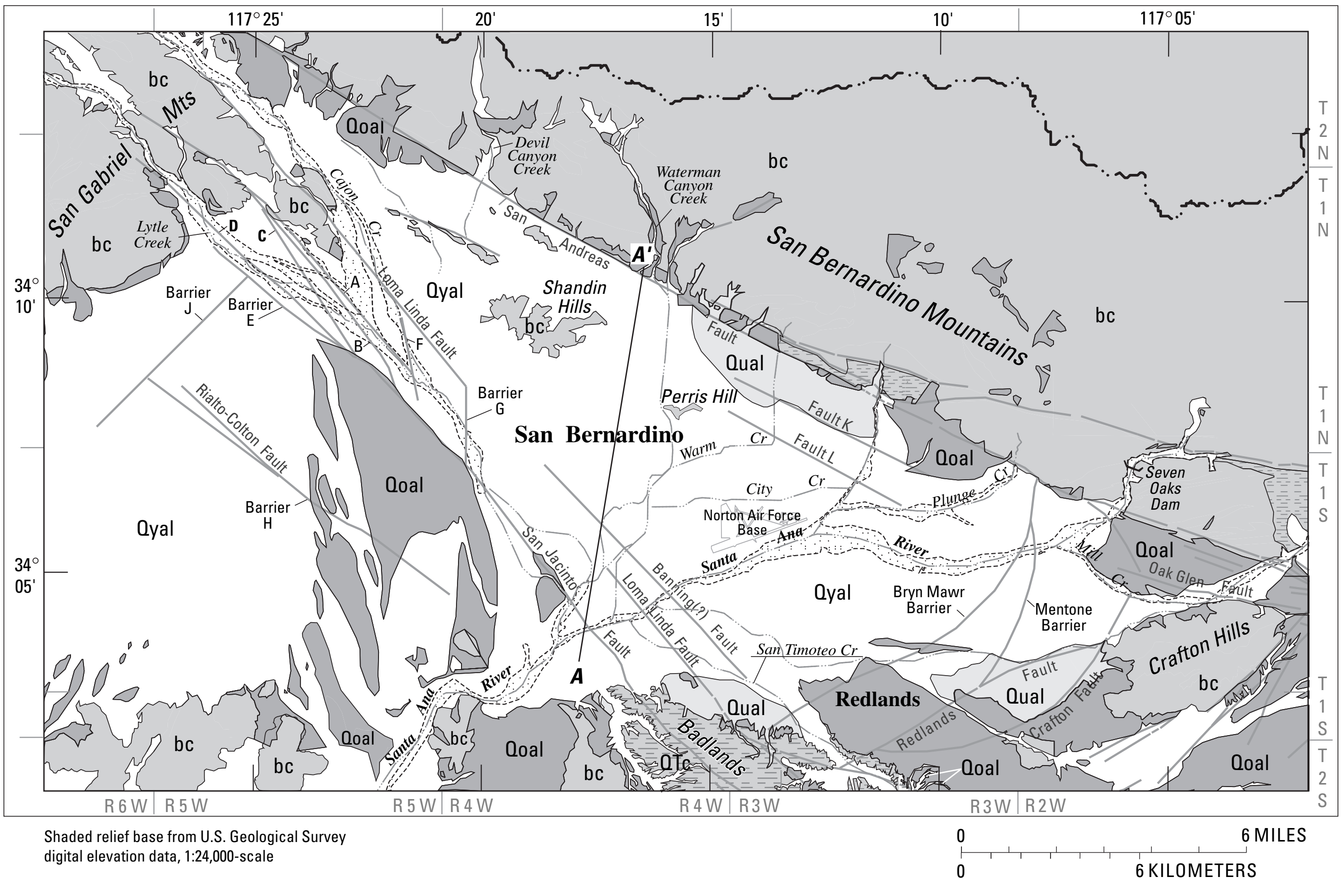
The shift from agricultural to municipal water use and the increasingly restrictive public drinking-water standards (U.S. Environmental Protection Agency, 1994; California Department of Water Resources, 1995a) have caused considerable concern about ground-water quality. Between 1985 and 1994, more than 40 municipal supply wells were closed, at least temporarily, because concentration of a constituent in ground water exceeded public health standards (NBS/Lowry, 1994). The water-quality constituents cited most frequently as exceeding standards include nitrate, organic solvents (TCE and PCE), pesticides (DBCP), and radioactivity.

Previous Investigations

The earliest study of the hydrogeology of the San Bernardino area includes maps showing surface-water courses, swamps, and irrigated land (Hall, 1888). The earliest comprehensive hydrogeologic analyses of the valley and region were made by Lippincott (1902a,b) and Mendenhall (1905, 1908). The hydrogeology and basin storage of California's south coastal basins, including the San Bernardino area, were summarized by Eckis (1934). Other studies of the hydrogeology of the San Bernardino area include those by Troxel (1954), California Department of Water Resources (1957, 1970, 1971, 1978, and 1979), Dutcher (1956), Dutcher and Burnham (1960), Burnham and Dutcher (1960), Dutcher and Garrett (1963), and Dutcher and Fenzel (1972).

Geologic mapping of the area was done by Dutcher and Garrett (1963), Morton (1974), Matti and others (1985), Burtugno and Spittler (1986), and Matti and Carson (1991). More recently, a geologic reinterpretation of the area has been initiated by D.M. Morton (U.S. Geological Survey, oral commun., 1993).

Geologic hazards of the area were mapped at a scale of 1:48,000 by Fife and others (1976). This information and additional earthquake data were compiled by Ziony (1985) in a comprehensive volume summarizing earthquake hazards in southern California and including an extensive discussion of specific hazards in the San Bernardino area. The probable damage to urban facilities that would result from an earthquake with magnitude $6M_L$ or greater in the San Bernardino area was assessed by the California Division of Mines and Geology (1993). Land subsidence in the area for the period 1935–70 was measured by Lofgren (1969) and Miller and Singer (1971). Detailed mapping of near-surface sediments in the San Bernardino area was done by Matti and others (1985) in order to calculate the susceptibility of these sediments to earthquake-induced liquefaction (Matti and Carson, 1991).



EXPLANATION

GEOLOGIC UNITS

Quaternary

- Qrc — River-channel deposits
- Qyal — Younger alluvium
- Qual — Undifferentiated younger and older alluvium
- Qoal — Older alluvium

Quaternary and Tertiary

- QTc — Sedimentary rocks

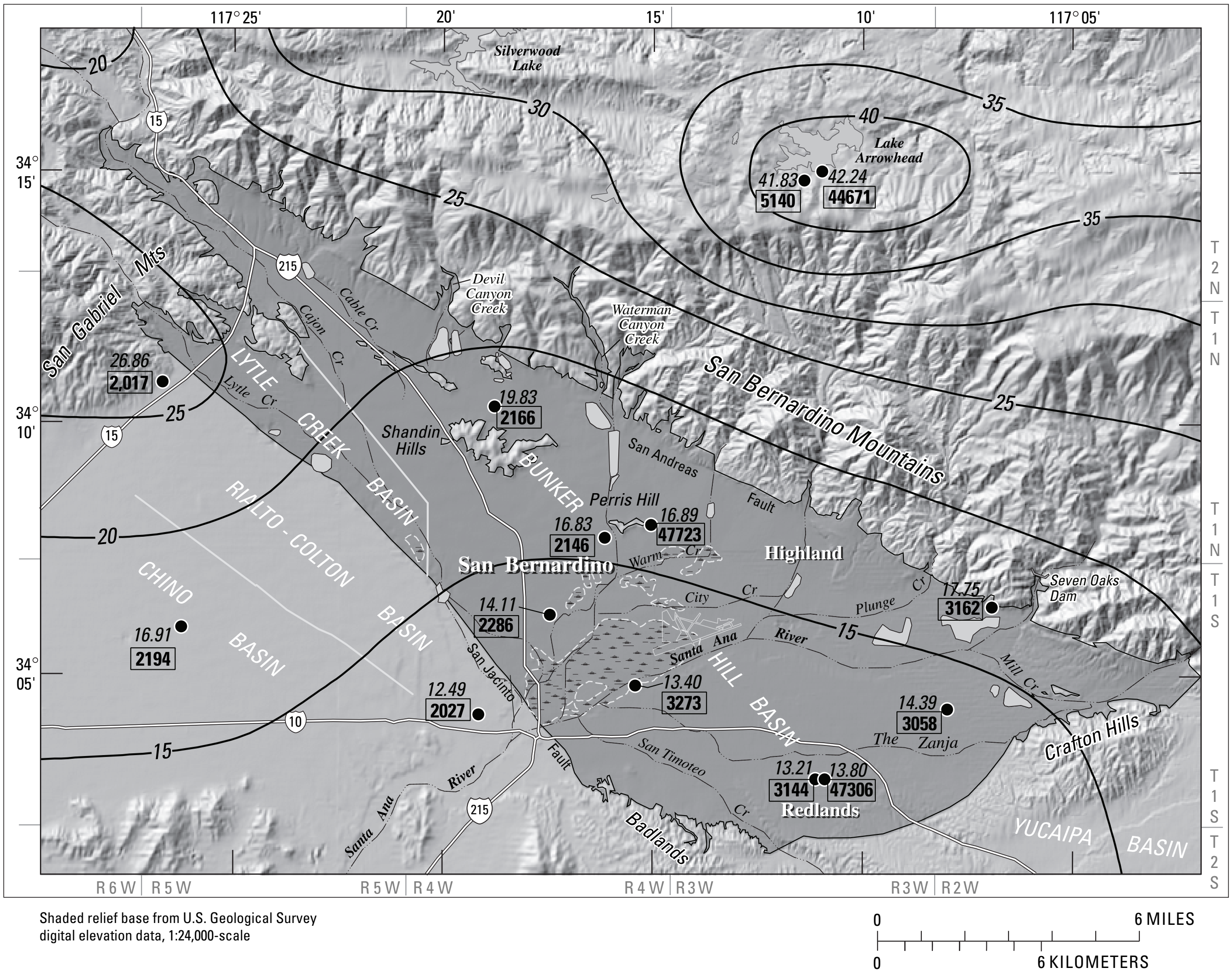
Tertiary and Pre-Tertiary

- bc — Basement complex of igneous and metamorphic rocks

MAP UNITS

- Fault / Barrier J — **Fault or ground-water barrier**—May be concealed or approximately located
- **Geologic contact**
- A —— A′ — **Line of section**—See figures 23, 24, 37
- **Boundary of Santa Ana River drainage basin**

Figure 5. Generalized surficial geology of the San Bernardino area, California, adapted from Morton (1974) and Dutcher and Garrett (1963).



EXPLANATION

Basin boundary—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray

Fault or ground-water barrier—May be concealed or approximately located

Former marshland

Artificial-recharge basin

— 20 — **Line of average annual precipitation (1928-98)**—Interval 5 inches per year

12.49 2027 **Data point**—Precipitation (top number), in inches and San Bernardino County Flood Control District station identifier (bottom number)

Figure 6. Average annual precipitation in the San Bernardino area, California, 1928–98. Data from the San Bernardino County Flood Control District (SBVFCD, 2000) with minor modifications to estimate missing monthly values.

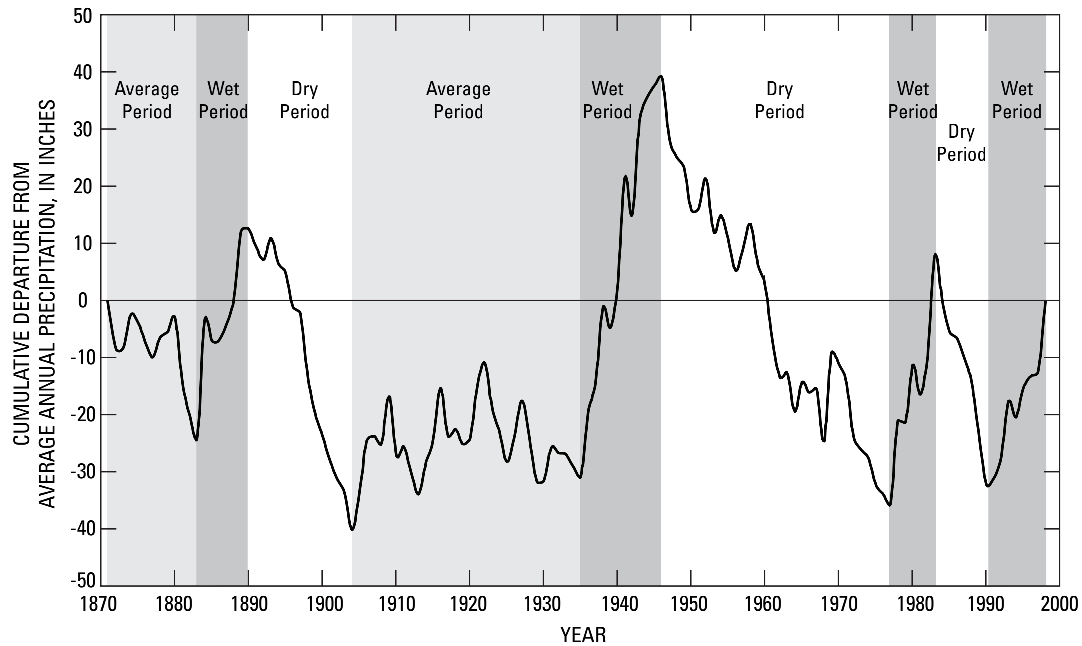


Figure 7. Cumulative departure from average annual precipitation at San Bernardino, California, 1871–1998. Data from the San Bernardino County Flood Control District, station 47723.

Ground-water-quality studies, particularly relating to nitrate problems, were done by Eccles and Bradford (1977), Eccles and Klein (1978), and Eccles (1979). Studies to quantify and delineate the distribution of nitrate and other forms of nitrogen in the unsaturated zone were done by Klein and Bradford (1979) for the Redlands area and by Klein and Bradford (1980) for the Highland area (*fig. 2*). The URS Corporation (1986) compiled a list of possible sources of perchloroethylene (PCE) and trichloroethylene (TCE) in the northwest part of the Bunker Hill basin and identified areas of contamination from analysis of soil-pore gas. Duell and Schroeder (1989) conducted an appraisal of ground-water quality in the San Bernardino area and analyzed for concentrations of inorganic constituents, nitrates, and volatile organic compounds. A map report delineating areas of ground-water contamination and listing municipal production wells with concentrations of volatile organics, pesticides, nitrate, or radiation exceeding public health standards was prepared by NBS/Lowry (1994). The isotopic character of ground water along a section from the base of the San Bernardino Mountains near East Twin Creek to the I-10–I-15 freeway interchange was investigated by Izbicki and others (1998) using depth-dependent sampling methods.

Two areas of ground-water contamination in the San Bernardino area—the Newmark Operable Unit north and east of Shandin Hills and the Muscoy Operable Unit south of Shandin Hills—are being investigated by the U.S. Environmental Protection Agency (USEPA) under the federal superfund cleanup program (URS Corporation, 1993, 1994, 1996). Ground-water contamination on, and immediately southwest of Norton Air Force Base is being investigated as part of the Installation Restoration Program (IRP) as required prior to base closure (CDM Federal Programs Corporation, 1997). An additional area of ground-water contamination north of Redlands is being studied by Lockheed Corporation at the request of the California Regional Water Quality Control Board (1994).

Artificial recharge to the valley-fill aquifer has been studied by Moreland (1972), Warner and Moreland (1972), and Schaefer and Warner (1975). Studies of specific aspects of the hydrologic system include analysis of geothermal resources (Young and others, 1981), generalized streamflow relations (Busby and Hirashima, 1972), and rising ground water (San Bernardino Valley Municipal Water District, 1981a).

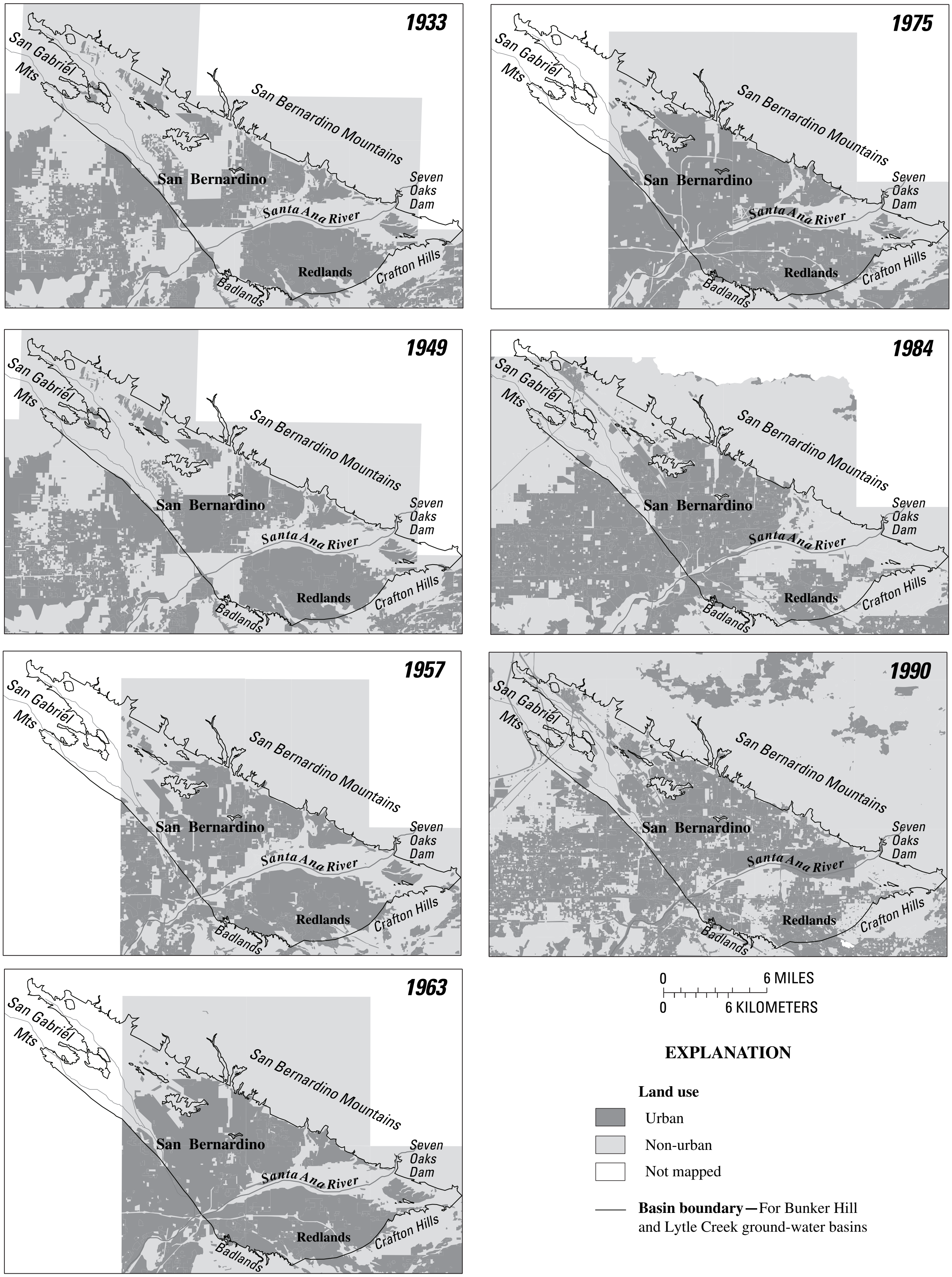


Figure 8. Changes in land use in the San Bernardino area, California, 1933–90.

Numerical simulation of the valley-fill aquifer system began with a simplified well-response model developed by Durbin (1974) and Durbin and Morgan (1978). A more complex ground-water flow model was developed by Hardt and Hutchinson (1980) in order to simulate aquifer response to the combined effects of natural recharge, artificial recharge using imported water, and ground-water pumping. This model was used by Hardt and Freckleton (1987) to simulate ground-water-level response to projected recharge and pumping through the year 2015. A regional salt-balance model was developed for the upper Santa Ana River watershed, including the San Bernardino area (Water Resources Engineers, Inc., 1969), and has been used and modified several times to answer questions about salt accumulation and nitrate loading (Wildermuth, 1991). A two-dimensional simulation of heat and ground-water transport through the igneous-metamorphic bedrock underlying the unconsolidated deposits in the San Bernardino area was done by Hughes (1992).

Several water-management studies have been done for the Santa Ana River drainage area and for the San Bernardino area in particular. A comprehensive assessment of water demands through the year 2015 was made by the California Department of Water Resources (1970, 1971, 1972). At about the same time, a detailed safe-yield study (San Bernardino Valley Municipal Water District, 1970) was done as part of the adjudication of water resources in the San Bernardino area (State of California 1969a,b). During the 1970s, water-management studies were required by the State of California to identify the amount of land subsidence that had occurred in the previous two decades and to estimate the additional subsidence that might occur if specific water-management plans were adopted in order to meet the projected increases in water demand (Lofgren, 1969; Miller and Singer, 1971). For example, implementation of one plan (A) would satisfy increased demand, but the proposed significant increase in pumpage would cause a permanent decline in ground-water levels of more than 300 ft and would induce an additional 6 ft of land subsidence (California Department of Water Resources, 1970, p. 62).

By about 1980, a new set of water-management studies was prompted by an overly full ground-water basin caused by increased recharge of abundant native runoff and imported water. These studies focused on removing ground water from the urbanized marshland in order to halt ongoing damage from increased hydrostatic pressure and to prevent damage from liquefaction in the event of a major earthquake. The quantity of recharge and pumpage needed to tilt the water table—keeping it high in the alluvial fan area and low in the marshland—was identified by S.S. Papadopulos and Associates, Inc. (1985) using constrained optimization techniques. Using similar techniques, but taking into account the nonlinear response of ground-water levels to evapotranspiration, Danskin and Freckleton (1992) identified the minimum pumpage from existing and proposed wells needed to solve the high ground-water problem while maintaining acceptably high ground-water levels near municipal and agricultural wells.

Previous investigations described most of the major geologic and hydrologic features of the area and presented potential solutions for some water-management issues. Despite this wealth of information, the present study was prompted by the complexity of solving multiple water-quantity and water-quality problems simultaneously and by the lack of an integrated understanding of the surface-water and ground-water systems.

A draft version of this report was completed in 1997, and following review, important revisions were made to the ground-water data and ground-water flow model. These revisions included increasing the areal extent of the ground-water flow model, extending the period of data analysis and simulation from 1994 to 1998, revising ground-water pumpage and return flow, modifying conductance values of streams to better simulate recharge in very wet years, revising underflow along the basin perimeter, changing the method of simulating ground-water outflow across the San Jacinto fault from a constant value to a head-dependent value, and adding an analysis of precipitation on the ground-water basin and how it contributes to surface-water outflow and ground-water recharge. Despite these several changes, performance of the ground-water flow model and hydrogeologic conclusions from the study remain remarkably similar. It is likely, however, that the credibility of both the model and conclusions were enhanced by the more rigorous, integrative analysis.

Acknowledgments

A large group of individuals from a variety of agencies aided in this study by providing data, interpretations, and valuable personal observations about the San Bernardino area. The authors especially thank Michael H. Lowe, Joseph F. Stejskal (retired), and James G. Dye (previously with), City of San Bernardino; Michael L. Huffstutler (retired), City of Redlands; David V. Garcia, Babs Makinde-Odusola, and Zahra Panahi, City of Riverside; Robert E. Martin, East Valley Water District; Hooite Rugge, Fontana Union Water Company; Eugene P. McMeans, Riverside-Highland Water District; Kenneth J. Mashburn and Kenneth A. Miller, San Bernardino County Flood Control; D. Burnell Cavender, William J. Hiltgen, Peter J. Russer (deceased), and Nereus L. Richardson and Douglas Headrick both previously with the San Bernardino Valley Water Conservation District; Kevin P. Mayer, U.S. Environmental Protection Agency; Steven E. Mains, Western Municipal Water District; and Anthony W. "Butch" Araiza and Ira B. Pace, West San Bernardino County Water District.

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Hydrology

Historical hydrologic conditions in the San Bernardino area in the late 1880’s are shown in *figure 9*. Important characteristics of the hydrologic system at that time include intermittantly flowing streams, marshlands, and an extensive artesian area. These characteristics, except an intermittantly flowing Santa Ana River, represent pre-development conditions that probably existed for hundreds or thousands of years prior to human intervention.

As depicted in *figure 9*, streams emanate from the San Bernardino Mountains but do not flow continuously across the land surface, except during the largest floods. Rather they stop after a short distance, having lost all flow as recharge to the ground-water system. Then, further downstream, flow resumes as a result of ground water, restricted from flowing across the less permeable San Jacinto fault, rising to the land surface and reestablishing flow in the streams. This intermittant condition was true for all streams except for Lytle Creek and for the Santa Ana River, prior to diversions in the mid 1800s. Of equal historical importance are the extensive, but somewhat discontinous marshlands, including bogs and swamps. These areas are generally proximal to the lower stream reaches and likely also result from the rising ground water, similar to the formation of cienagas in other semiarid basins. The nearly continuous marshland between the Santa Ana River and Warm Creek limited human development in that area until ground-water levels declined during the drought of the 1950s and 1960s (*fig. 7*). During wetter periods, such as during the early 1900s and 1940s, the marshland between the two streams upstream of the San Jacinto fault had standing water, was a favorite area for bird-watching, and was used by residents for boating (*fig. 10*).

Also noteworthy is the large artesian area which covers nearly one-third of the Bunker Hill basin and extends nearly to the base of the San Bernardino Mountains in the vicinity of Perris Hill. This extensive area of upward ground-water flow results from infiltration of surface water to the ground-water system immediately adjacent to the mountain front, ponding of the ground-water system by the San Jacinto fault, and minimal development of the ground-water system by the late 1800’s.

Surface-Water System

The surface-water system in the San Bernardino area includes several natural streams and many structures designed to convey or recharge surface water (*fig. 11*). These structures include canals, ditches, and pipelines; the California Aqueduct that conveys imported water from northern California; flood-control, sewage-effluent, and debris basins; artificial-recharge basins for enhancing recharge of diverted streamflow and imported water; and the Seven Oaks Dam on the Santa Ana River. Major components of the surface-water system are identified on *plate 1*. The routing of water among these components is shown diagrammatically on *plate 2*.

Modifications to the natural surface-water system began in the early 1800s. In about 1810, the first diversion from the Santa Ana River was made to supply water for irrigation (Scott, 1977). In 1848, widespread irrigation began and prompted an equally rapid increase in water use (Scott, 1977). Many water agencies were formed to divert surface flow from the Santa Ana River and its tributaries and to organize delivery of the water throughout the surrounding area, primarily to irrigate citrus crops.

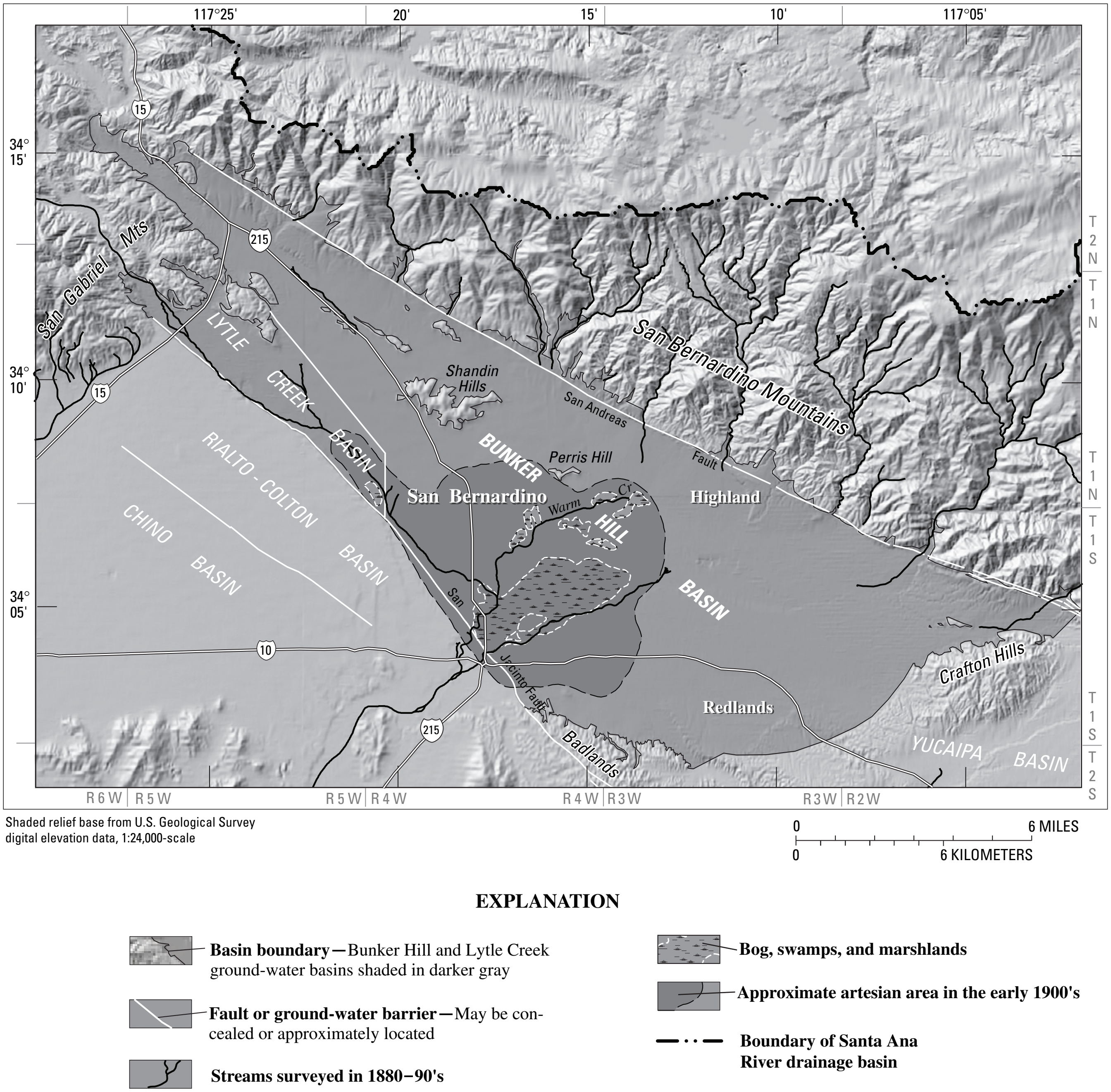


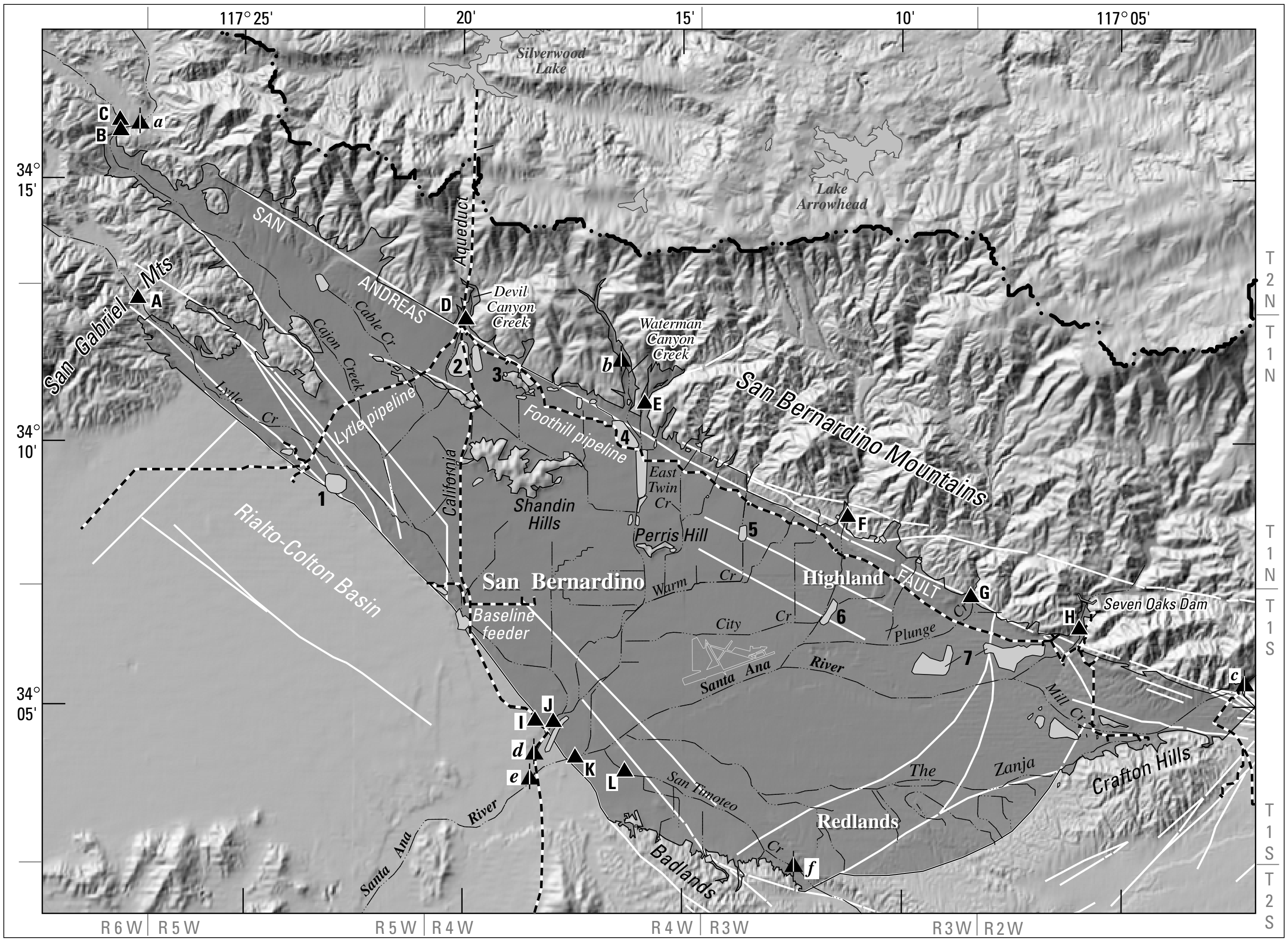
Figure 9. Historical hydrologic conditions in the San Bernardino area, California, circa 1880. Adapted from Hall (1888), Mendenhall (1905), and Fife and others (1974).

Figure 10. Urbita Springs area, 1903. An extensive marshland (cienaga) formed by ground water rising to the land surface between the Santa Ana River and Warm Creek, just upstream of the San Jacinto fault, was augmented for recreation by artesian flow from the Urbita Springs well.

As irrigation increased, the Santa Ana River flowed intermittantly and conservation measures became necessary. Artificial-recharge basins (*fig. 11*) were constructed on the major streams in order to increase the effective yield of the combined surface-water and ground-water systems. During periods of surplus runoff, the excess water was routed to the basins in order to recharge the ground-water system. During subsequent periods of low runoff and during periods of high demand, such as summer, the recharged water was extracted from wells.

After the mid-1940s, water use for irrigation decreased; however, rapid urbanization of agricultural land (*fig. 8*) caused an increase in water use for municipal purposes (Scott, 1977). This increase in water use and declining runoff prompted a lawsuit filed in 1963 by the Orange County Water District against other users of surface water in the Santa Ana River watershed. The suit, along with a related one regarding ground-water use, was settled in 1969 with a stipulated judgement that defined surface-water rights and created a watermaster to monitor surface-water use in the Santa Ana River watershed (State of California, 1969a,b). In 1972, major pipelines were completed that allowed surface water from northern California to be imported into the San Bernardino area. Although this external source provided additional flexibility for local water managers, it was not used to significantly increase the local supply of water, perhaps because of the relatively high cost of imported water. The net result of these various changes has been that total water use in San Bernardino has changed little since about 1960.

Concurrent with the development of agriculture and the increasing population of southern California, hydroelectric power production was begun on the Santa Ana River, Mill Creek, Lytle Creek, and Devil Canyon Creek. More than 50 percent of the average annual inflow from these streams is diverted from the channels into penstocks and used to generate hydroelectric power. Typically, a gaging station is present on the natural stream channel and on any upstream diversions. As a result, stream discharge is reported as uncombined discharge (flow in the natural stream channel) and combined discharge (flow in the natural stream channel plus measured discharge for all upstream diversions, most of which are used to generate hydroelectric power). The adjudication of surface-water rights has ensured that most streamflow into and out of the San Bernardino area is measured continuously and is reported annually by the local watermaster.



Shaded relief base from U.S. Geological Survey digital elevation data, 1:24,000-scale

EXPLANATION

Basin boundary—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray

Fault or ground-water barrier—May be concealed or approximately located

Artificial-recharge basin and number—Refer to table 5

Major pipeline

Stream, canal, or ditch—Bottom may be native earth or concrete

Boundary of Santa Ana River drainage basin

▲ **Gaging stations**—Continuous

A	11062001	Lytle Creek near Fontana
B	11063510	Cajon Creek below Lone Pine Creek, near Keenbrook
C	11063500	Lone Pine Creek near Keenbrook
D	11063680	Devil Canyon Creek near San Bernardino
E	11058500	East Twin Creek near Arrowhead Springs
F	11055801	City Creek near Highland
G	11055501	Plunge Creek near East Highlands
H	11051501	Santa Ana River near Mentone
I	11065000	Lytle Creek at Colton
J	11060400	Warm Creek near San Bernardino
K	11059300	Santa Ana River at E Street
L	11057500	San Timoteo Creek near Loma Linda

Gaging stations—Discontinued

a	11063000	Cajon Creek near Keenbrook
b	11058600	Waterman Canyon Creek near Arrowhead Springs
c	11054001	Mill Creek near Yucaipa
d	11065801	Warm Creek near Colton
e	11066050	Santa Ana River at Colton
f	11057000	San Timoteo Creek near Redlands

Figure 11. General features of the surface-water system in the San Bernardino area, California, 1998.

Seven Oaks Dam on the Santa Ana River was completed in 1999. The primary purpose of this massive earthen structure is to control flood water and to prevent damage to downstream agricultural and urban land such as occurred in 1938. A secondary benefit of the dam is to provide a conservation pool with about 50,000 acre-ft of storage that is used to augment local water supplies. Without the conservation pool, some of this water would flow out of the area and be unavailable for either ground-water recharge or direct delivery to a water-treatment plant.

Sources of Inflow and Outflow

Most surface-water flow in the San Bernardino area originates as runoff into streams and creeks that drain the San Gabriel and San Bernardino Mountains and as runoff from the hills bordering San Timoteo Creek (*fig. 11* and *pl. 1*). Runoff into major streams is measured at several gaging stations operated by the U.S. Geological Survey (USGS) (*fig. 11*). A lesser quantity of runoff is ungaged and enters the area as flow in small creeks and as sheetflow from the surrounding mountains. Other sources of surface-water flow include: local runoff from precipitation on the land surface of the Bunker Hill and Lytle Creek basins; water imported from northern California in the California Aqueduct; wastewater discharge from the Redlands and the San Bernardino sewage-treatment plants and from the California Edison steam power-generation plant; and ground-water seepage into Warm Creek during periods of high ground-water levels in the vicinity of the former marshland (*fig. 2*).

Gaged Runoff

As required by the adjudication of water rights in the area (State of California, 1969a,b), nearly all surface water that enters or leaves the Bunker Hill and Lytle Creek basins is measured. Between 1945 and 1998, inflow was measured routinely at eleven continuous-recording gaging stations along the base of the San Bernardino and San Gabriel Mountains (*fig. 11*). Of the total gaged inflow, almost 80 percent occurs in the three largest streams—the Santa Ana River, Lytle Creek, and Mill Creek (*fig. 12*). Outflow from the basins is measured at three continuous-recording gaging stations on the Santa Ana River, Warm Creek, and Lytle Creek.

Total gaged inflow and outflow for the San Bernardino area for 1945–98 are shown in *table 1*. During this 54-year period, total gaged inflow averaged about 146,000 acre-ft/yr, with a minimum of about 36,000 acre-ft in 1990 and a maximum of about 674,000 acre-ft in 1969. Total gaged outflow averaged about 68,000 acre-ft/yr with a minimum of about 12,000 acre-ft in 1968 and a maximum of about 370,000 acre-ft in 1980.

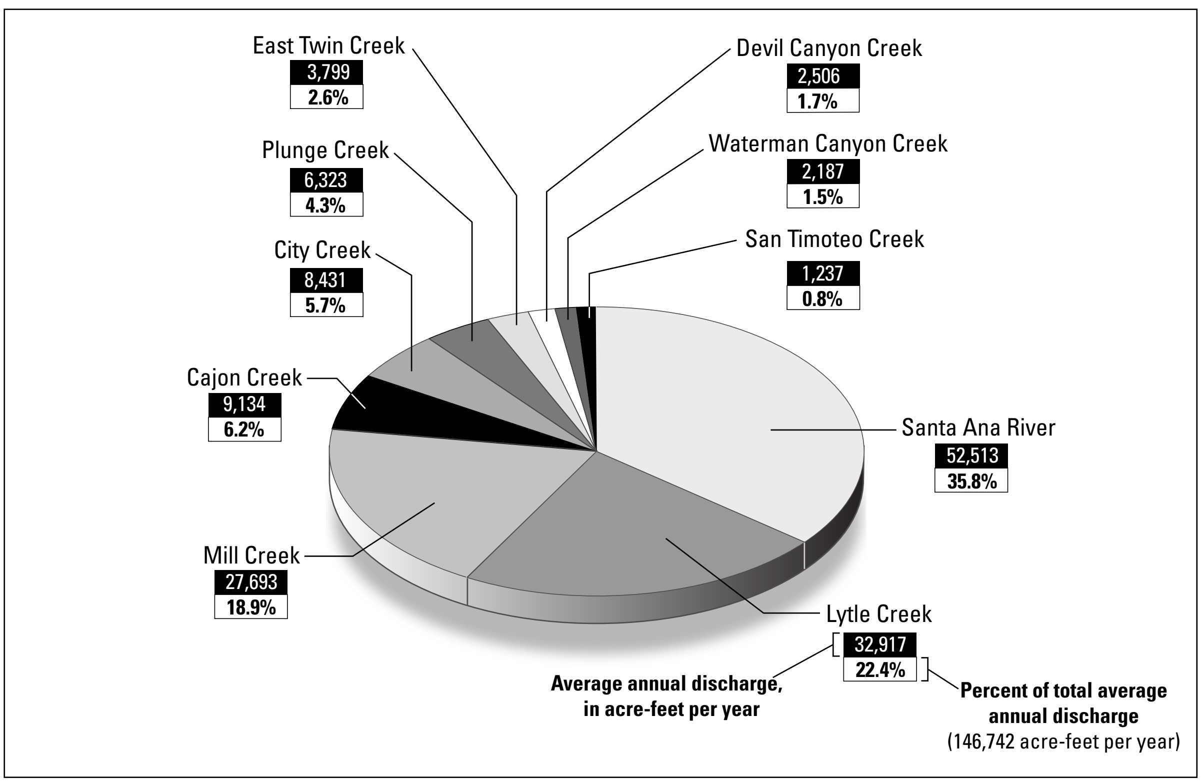


Figure 12. Average annual discharge of gaged streams flowing into the San Bernardino area, California, 1945–98.

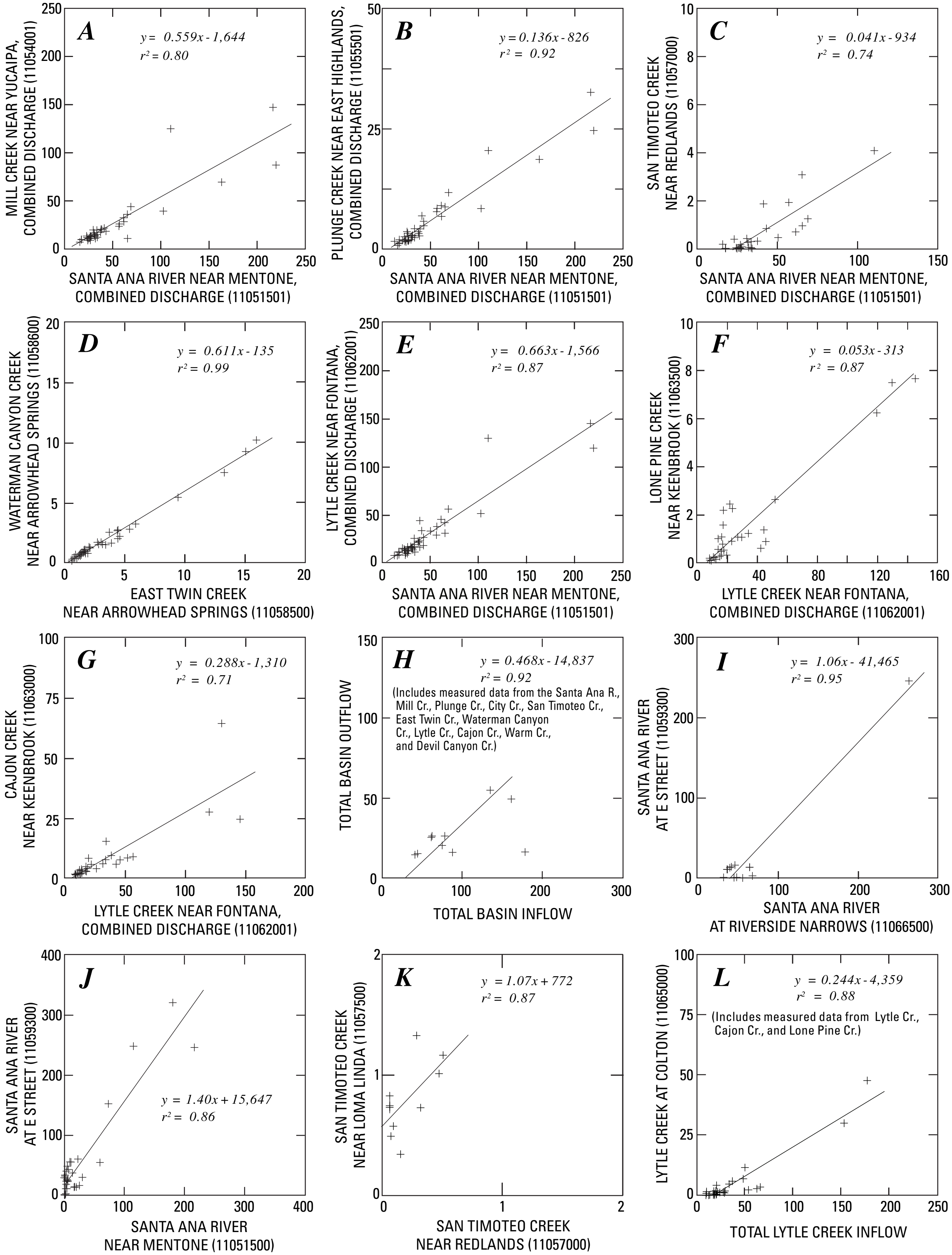


Figure 13. Linear-regression relations used to estimate annual stream discharge in the San Bernardino area, California. Annual data points are shown; values are in thousands of acre-feet per year.

Although measured inflow and outflow records for 1945–98 are nearly complete, estimation of some discharge values was required. For gages with only 11 months of data, the 12th month was estimated using qualitative comparisons with adjacent streams. In most cases, discharge during the missing months probably was zero. For longer periods of missing record, estimated annual streamflow values were calculated using linear-regression equations. Typically, discharge at gaging stations with missing record was estimated using data from the Santa Ana River gage, which has a complete record for 1945–98. Discharge at some gaging stations with missing record, such as the Waterman Canyon Creek gage (*table 1*), was estimated using data from a nearby gaging station with a similar drainage area, in this case from the East Twin Creek gage. For Cajon Creek, different combinations of gaging-station measurements and regression equations were needed to compile a complete record.

Graphs of the annual streamflow data used for the linear regressions, the equations themselves, and the related r^2 values are shown in *figure 13*. The r^2 value is a measure of the goodness-of-fit of the data to the regression equation. If the equation (line) is a good estimator of the data, the r^2 value will be near 1.0; if it is a poor estimator, the value will be near zero (Davis, 1986, p. 182).

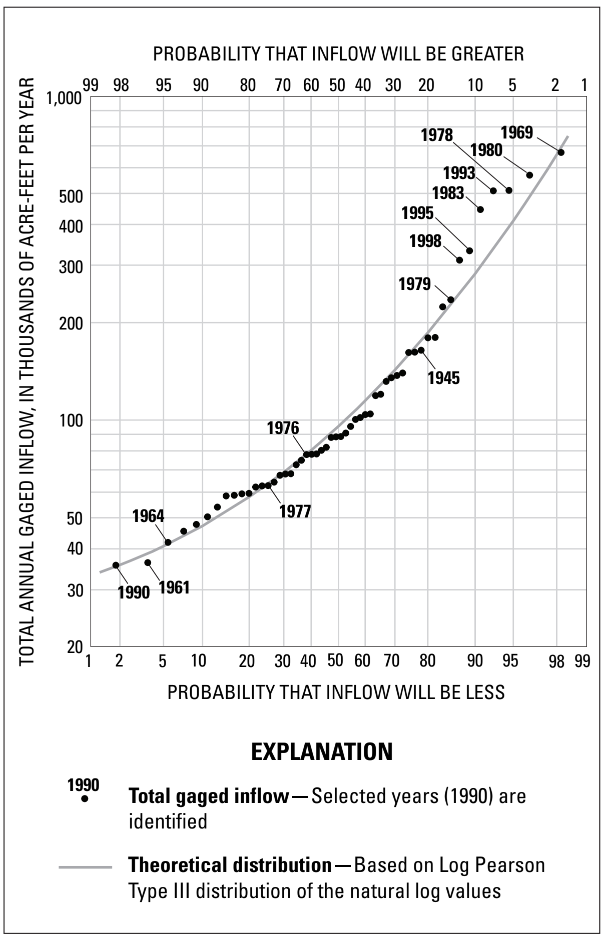
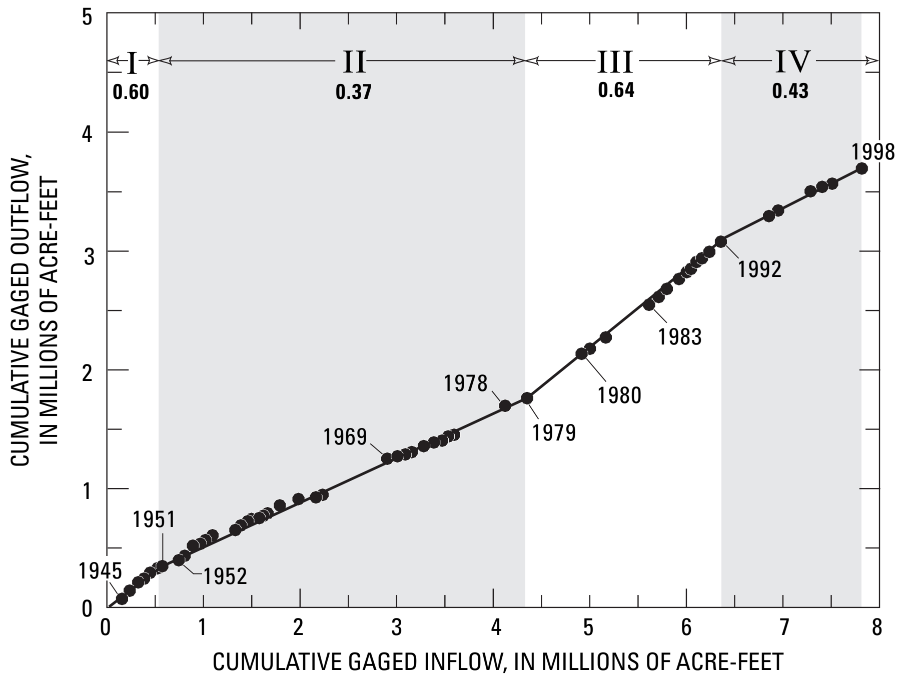


Figure 14. Cumulative probability of annual gaged surface-water inflow for the San Bernardino area, California, 1945–98. Values of inflow are listed in table 1.

Total gaged surface-water inflow (*table 1*) varies greatly from year to year, depending on local weather conditions. This variability and the uncertainty of inflow for a given year can be described in a cumulative probability graph (*fig. 14*). The graph, developed using data for total gaged surface-water inflow (*table 1*) and a Log Pearson Type III analysis (Linsley and others, 1975, p. 343), shows the likelihood of different quantities of annual inflow. For example, 50 percent of the time, total inflow will be less than about 100,000 acre-ft/yr; only 10 percent of the time will total inflow exceed about 290,000 acre-ft/yr. Selected years are labeled to show the probabilities of total inflow associated with above-average (wet) years (1969, 1983, 1998) and below-average (dry) years (1964, 1976, 1990). The deviation for very wet years between measured data and the theoretical distribution (*fig. 14*) is not well understood, but may result from either the type of precipitation (rain versus snow) or from the intensity of precipitation (many, small storms versus a few, large storms).

The relation between cumulative gaged surface-water inflow and outflow for 1945–98 is shown in *figure 15*. The slope of the double-mass curve indicates the quantity of inflow leaving the basin and the quantity retained within the basin. During four time periods between 1945 and 1998, the slope of the curve remains relatively constant. During period I (1945–51), 60 percent of the inflow left the basin and 40 percent remained; during period II (1952–79), 37 percent left and 63 percent remained; during period III (1980–92), 64 percent left and 36 percent remained; during period IV (1993–98), 43 percent left and 57 percent remained. Using a similar graphical analysis for years 1945–80, Hardt and Freckleton (1987) identified slopes for periods I and II and suggested that the marked difference in slope might be related to the height of ground-water levels in the southwestern part of the Bunker Hill basin.

When ground-water levels in the former marshland are above land surface, ground-water rises into Warm Creek and is discharged out of the basin. This additional discharge effectively decreases the percentage of inflow retained in the basin. This type of condition was present during periods I and III. During these two periods, ground-water levels were relatively high and a similar percentage of inflow was retained (40 and 36 percent, respectively). In contrast, during periods II and IV, ground-water levels declined, particularly in the former marshland between Warm Creek and the Santa Ana River near the San Jacinto fault. Ground water rising into Warm Creek decreased to zero, and the percentage of inflow retained in the basin (63 and 58 percent, respectively) was about 50 percent greater than for periods I and III.



EXPLANATION

III — **Time period**—Identified by I, 1945-51; II, 1952-79; III, 1980-92; IV, 1993-98

0.64 **Slope of line segment**—Value of 0.64 means that 64 percent of gaged inflow left the San Bernardino area as outflow during time period III

1980 **Data point**—Selected calendar year (1980), plotted at end of annual inflow

Line segment—Indicates time period when the percent of inflow retained in the area remained relatively constant

Figure 15. Cumulative gaged surface-water inflow and outflow in the San Bernardino area, California, 1945–98.

Urbanization has been suggested as a possible explanation for the change in percentage of inflow leaving the San Bernardino area. During the period II, urban acreage began increasing, rising sharply during 1957–63 (*fig. 8*). In 1962, urban acreage exceeded irrigated acreage for the first time (California Department of Water Resources, 1985). A likely consequence of increased urban acreage is an increased acreage of impervious surfaces, which in turn would increase the quantity of inflow leaving the area. The percentage of inflow leaving the area during period II, however, is less than during period I, not more as would be expected if urbanization had a major affect on basin outflow. Similarly, the percentage of outflow during period IV is less than in the immediately preceeding period III, not more.

Urbanization, however, may produce a modest increase in basin outflow for time periods with similar depths to ground water. Comparing periods I and III shows a slightly greater percentage of inflow leaving the basin during the more recent, more urbanized period (60 increased to 64). Similarly for periods II and IV, the percentage of inflow leaving the basin increased from 37 to 43.

Discharge for the Santa Ana River itself is a reliable indicator of runoff to the San Bernardino area, both for individual streams (*fig. 13*) and for total inflow (*fig. 16*) . Since 1896, the river has been gaged continously where it flows out of the mountains. Discharge data used in this report for the Santa Ana River include all upstream diversions and is referred to as the combined-flow record (USGS station number 11051501). Some discharge data, particularly during the earliest years and during the floods of 1916 and 1927, are missing. Since 1928, however, the record is complete and comprises one of the most important hydrologic datasets in the San Bernardino area.

As a predictor of total gaged inflow for the San Bernardino area, the Santa Ana River is remarkably accurate. For the period 1945–98, only 1978 appears to be anomalous, with discharge for both Lytle and Cajon Creeks being unusually high (*fig. 16*; *table 1*). In most years, total gaged inflow to the San Bernardino area is about three times discharge in the Santa Ana River at the base of the mountains.

In order to estimate other sources of gaged and ungaged inflow, the longterm average discharge for the Santa Ana River for 1928–98 was used to calculate annual values, expressed as a percent of longterm average runoff (*table 2*). The long, gaged record used to develop these percentages and the high correlation between the Santa Ana River and other surface-water inflows make the annual values in table 2 exceptionally useful both for historical analysis and for future management scenarios.

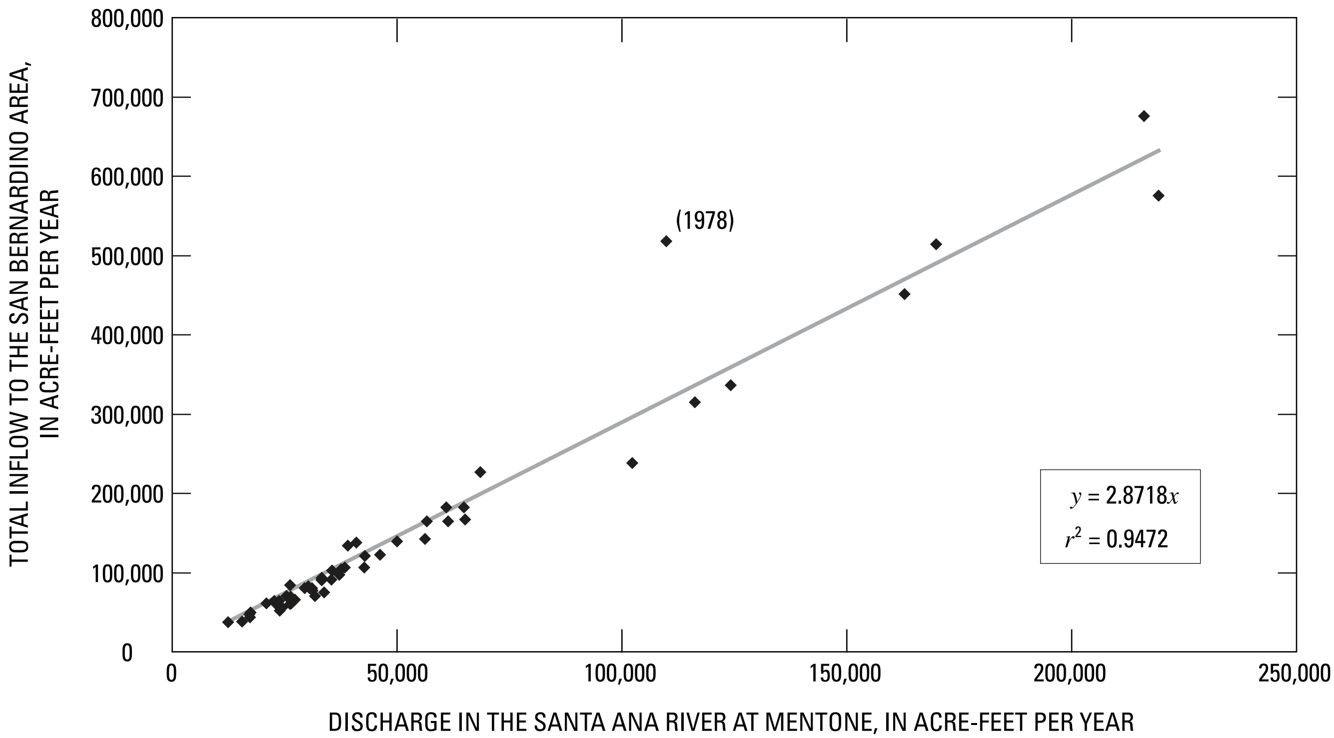


Figure 16. Relation between discharge in the Santa Ana River at the base of the mountains (USGS station 11051501) and total inflow to the San Bernardino area, 1945–98. Values are listed in table 1. During calendar year 1978, inflow for Lytle and Cajon Creeks was unusually high.

Table 2. Percent of long-term average annual runoff for the Santa Ana River, California, 1945–98.

[Annual values calculated as a percent of long-term average annual discharge (53,947 acre-feet) for calendar years 1928–98 for the Santa Ana River near Mentone (station number 11051501, table 1); average for years 1945–98]

Calendar year	Percent of long-term average runoff	Calendar year	Percent of long-term average runoff
1945	121	1973	105
1946	93	1974	69
1947	62	1975	58
1948	59	1976	57
1949	63	1977	43
1950	49	1978	204
1951	45	1979	190
1952	106	1980	407
1953	50	1981	62
1954	80	1982	114
1955	51	1983	302
1956	50	1984	72
1957	48	1985	57
1958	128	1986	80
1959	49	1987	45
1960	46	1988	39
1961	29	1989	33
1962	62	1990	24
1963	32	1991	49
1964	33	1992	73
1965	76	1993	315
1966	121	1994	66
1967	113	1995	231
1968	55	1996	86
1969	401	1997	66
1970	70	1998	216
1971	58		
1972	43	Average	97

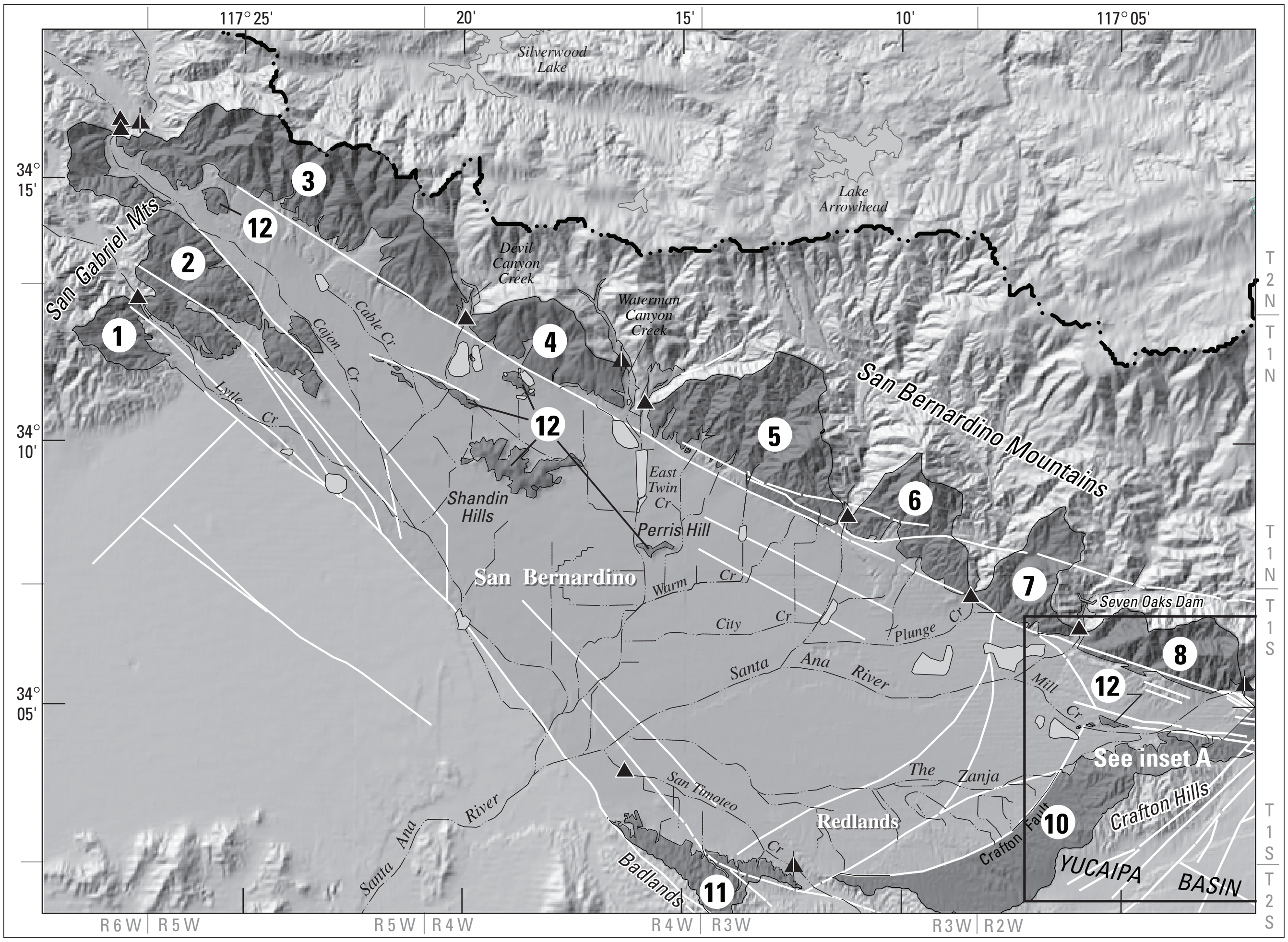
Ungaged Runoff

Ungaged runoff in the San Bernardino area is derived largely from triangular areas between gaging stations in the San Gabriel and San Bernardino Mountains (*fig. 17*). Some ungaged flow also runs off Crafton Hills, the badlands, and small bedrock outcrops within the Bunker Hill basin, such as Shandin Hills. Ungaged runoff occurs as flow in small creeks and as sheetflow from steep bedrock surfaces. Several small debris basins have been constructed at the base of the San Bernardino Mountains to capture ungaged runoff and related sediment and to prevent damage to nearby homes (*pl. 1*; California Department of Water Resources, 1970, fig. 11).

The quantity of ungaged runoff for 1928–98 was estimated for each area shown in *figure 17*. Similar ungaged areas, defined by the Western–San Bernardino Watermaster (1972, plate 26), were renumbered and modified slightly to conform to the present study-area boundaries. Acreage (*fig. 17*) and mean precipitation for 1928–98 (*fig. 6*) were calculated for each area using the GIS. The product of these two values is potential ungaged runoff (*table 3*). Evapotranspiration, however, consumes most of this potential runoff, leaving a much smaller amount that actually enters the Bunker Hill and Lytle Creek basins.

Determining this amount is much more problematic and commonly is done using a rainfall-runoff relation developed for a nearby gaged drainage. This technique was used by the California Department of Water Resources (1971, p. 133) to estimate total ungaged runoff entering the Bunker Hill, Lytle Creek, and Yucaipa basins. For water years 1935–60, total estimated ungaged runoff into these three basins averaged 21,420 acre-ft/yr. This amount is about 25 percent of the total potential runoff for this larger area. Based on these previous studies, the percentage of potential runoff for ungaged areas bordering the San Bernardino area was assumed to be 25 percent (*table 3*). A slightly lesser value of 20 percent was assumed for runoff from the more permeable deposits in or near the badlands (areas 10 and 11, *fig. 17*). Using these values, average ungaged runoff into the San Bernardino area for 1928–98 was estimated to be about 17,000 acre-ft/yr (*table 3*).

Uncertainty in estimating average annual ungaged runoff, however, can be quite high—possibly as great as 50 percent. For example, the calculations of ungaged runoff made by the California Department of Water Resources (1971, p. 133) for water years 1935–60 yielded values about 35 percent greater than those calculated by J.C. Hanson (Western–San Bernardino Watermaster, 1972, p. 545–554). Such variability can result from different estimates for ungaged area, precipitation, infiltration, or evaporation.



Shaded relief base from U.S. Geological Survey digital elevation data, 1:24,000-scale

0 6 MILES
0 6 KILOMETERS

EXPLANATION

Basin boundary—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray

Fault or ground-water barrier—May be concealed or approximately located

1 **Boundary of area of estimated ungaged runoff**—Number corresponds to table 3

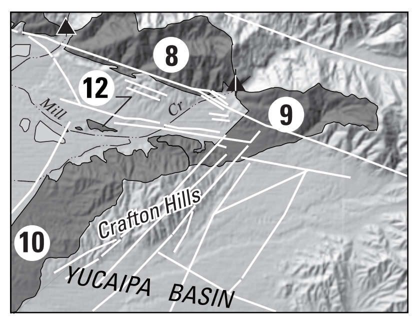
Artificial-recharge basin

Selected gaging stations

Continuous

Discontinued

Boundary of Santa Ana River drainage basin



Inset A. Ungaged runoff area 9.

Figure 17. Ungaged areas bordering the Bunker Hill and Lytle Creek basins in the San Bernardino area, California.

Average ungaged runoff for 1945–98 was calculated using mean precipitation data for San Bernardino (San Bernardino County Flood Control District, station 47723). The ratio of mean precipitation for the two periods (1945–98 and 1928–98) was 0.946 with 1928–98 being somewhat wetter. Therefore, average ungaged runoff for the 1945–98 period was about 16,000 acre-ft/year.

Annual values of ungaged runoff were calculated for each area for 1945–98 using an annual runoff index (P^{SARRO}_{k}) based on discharge in the Santa Ana River (*table 2*). This method can be summarized as:

$$Q^{UngagedRO}_{i,\,k} = Q^{UngagedRO}_{i,\,1928–98} \left(Q^{GagedRO}_{SAR,\,k} / Q^{GagedRO}_{SAR,\,1928–98}\right) \qquad (1a)$$

or more simply as

$$Q^{UngagedRO}_{i,\,k} = Q^{UngagedRO}_{i,\,1928–98} \left(P^{SARRO}_{k}\right)\ , \qquad (1b)$$

where

P^{SARRO}_{k} is average annual runoff for the Santa Ana River for calendar year k, compared to longterm average runoff for 1928–98, in percent;

$Q^{UngagedRO}_{i,\,k}$ is ungaged runoff from area i for calendar year k, in acre-ft /yr;

$Q^{UngagedRO}_{i,\,1928–98}$ is average annual runoff from ungaged area i for calendar years 1928–98, in acre-ft/yr;

$Q^{GagedRO}_{SAR,\,k}$ is gaged discharge for the Santa Ana River for calendar year k, in acre-ft/yr; and

$Q^{GagedRO}_{SAR,\,1928–98}$ is average annual discharge for the Santa Ana River for calendar years 1928–98, in acre-ft/ yr.

Annual values of ungaged runoff during 1945–98 ranged from about 4,000 acre-ft in 1990 to about 68,000 acre-ft in 1980.

In most years, it is unlikely that any ungaged runoff flows all the way through the basins and becomes part of gaged outflow (*table 1*). Rather, ungaged runoff probably follows the pattern of streamflow described by Mendenhall (1905) and illustrated in *figure 9*—water flows only a short distance from the mountain front before disappearing into the soil. Some additional evapotranspiration may occur from the unconsolidated deposits bordering the basins, but most ungaged runoff probably recharges the ground-water system.

Local Runoff

Local runoff in the San Bernardino area occurs from precipitation falling on the unconsolidated deposits and urbanized areas within the Bunker Hill and Lytle Creek basins. This runoff is in addition to the gaged and ungaged runoff that occurs as a result of precipitation in the surrounding mountains.

As in other semiarid basins, the potential evapotranspiration in the San Bernardino area is high, averaging more than 76 inches per year, nearly five times the average annual precipitation (San Bernardino County Flood Control District, 1975). As a result, most precipitation, which falls almost exclusively as rain on the basins, is evaporated or transpired before it can infiltrate or run off. However, also typical of a semiarid basin, many storms in the San Bernardino area have a short duration and high intensity, which means these events are less affected by evapotranspiration and may produce some runoff. During exceptionally wet years, such as 1969 and 1993 (*fig. 14*), precipitation greatly exceeds daily evapotranspiration, and a significant quantity of local runoff is evident—streets are flooded, small ditches are overtopped by flow, and normally dry catchment basins are full.

Estimates of local runoff were made by the California Department of Water Resources (1971) using measured precipitation at 28 stations in and adjacent to the San Bernardino area. During water years 1935–60, precipitation on the water-bearing materials, defined essentially as the nonmountainous part of the area, averaged 1.47 ft/yr (17.6 in/yr). Consumptive use was assumed to account for about 86 percent of this precipitation, and the remaining 14 percent (0.20 ft/yr) was assumed to be available for either runoff or ground-water recharge. For the San Bernardino area, this latter percentage equals about 15,000 acre-ft/yr of either local runoff or recharge from local runoff. Percolation of direct precipitation within the San Bernardino area was estimated by the California Department of Water Resources (1986, tables 17,18) to average about 8,400 acre-ft/yr for water years 1935–60. Using these estimates of precipitation and percolation, an average of about 6,600 acre-ft/yr of local runoff (15,000 minus 8,400) would be expected to leave the area as a result of direct precipitation on the unconsolidated valley-fill materials.

The quantity of local runoff, however, can vary greatly from one year to another, as a result of different quantities of annual precipitation, different intensities of precipitation for individual storms, and changes in land use. For example in 1980, when precipitation was much greater than average, percolation of direct precipitation was estimated to be about five times the average quantity, or about 42,000 acre-ft/yr (California Department of Water Resources, 1986, tables 17,18); undoubtedly, local runoff during 1980 also was much greater than average.

Local runoff is an important part of the hydrology of the San Bernardino area because it influences interpretations of surface-water inflow and outflow (*fig. 15*) and because it affects estimates of surface water retained in the area as ground-water recharge. Because no measurements of local runoff are available and because no intermediate stream gages are present within the San Bernardino area, estimates of local runoff have relied on precipitation measurements, estimated evapotranspiration, and estimated ground-water recharge.

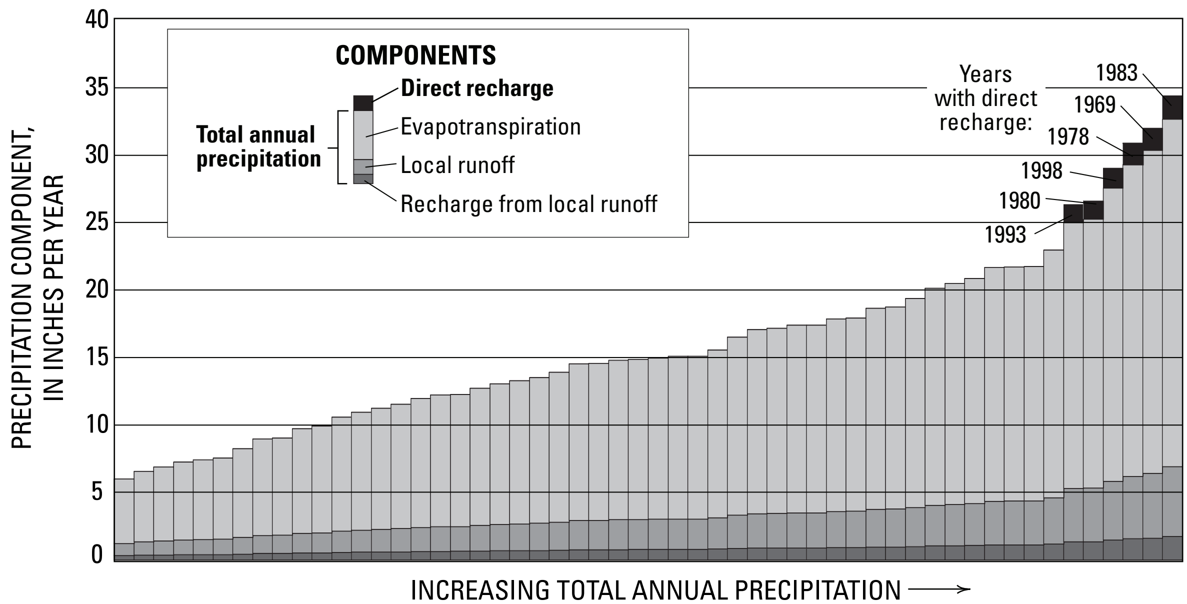


Figure 18. Components of precipitation on the Bunker Hill and Lytle Creek basins in the San Bernardino area, California, 1945–98. Precipitation measured at San Bernardino County Flood Control District station 47723. Selected years are identified.

To calculate annual values of local runoff for 1945–98, a methodology was used similar to that described above to calculate longterm average values. Annual precipitation at the San Bernardino gage (San Bernardino County Flood Control District station 47723) was divided into four components: evapotranspiration, direct recharge, local runoff, and recharge from local runoff. These components, shown in *figure 18*, are assumed to remain fairly contant as a percentage of total precipitation from one year to another. In all but the wettest years, evapotranspiration was assumed to be 80 percent of precipitation; local runoff, 15 percent; recharge from local runoff, 5 percent; and direct recharge of precipitation, 0 percent. In the wettest years, evapotranspiration dropped to 75 percent and direct recharge increased to 5 percent. Annual volumes for each component of precipitation were calculated using the areal distribution of mean annual precipitation shown in *figure 6*, scaled by annual precipitation data for station 47723. The implicit assumption that the areal distribution remains relatively constant each year seems reasonable because of the persistent direction of most storms from the northwest and the strong orographic effect of the San Gabriel and San Bernardino Mountains.

The relative percentages for each component of precipitation were based on previous investigations and a desire to create a simple tool that embodies basic hydrologic concepts for a semiarid, partly urbanized basin. A controlling assumption was that no direct recharge occurs in most years; evapotranspiration is simply too large, even in an area of highly permeable surficial deposits. In the exceptionally wet years, however, precipitation overwhelms the capacity of both vegetation and the soil to retain and evapotranspire water, and some percolation occurs.

An equally important assumption is that a small quantity of local runoff occurs in even the driest of years. High intensity rainfall is common, even in these years, and it results in runoff that is routed onto streets or into adjacent borrow ditches, then into unlined ditches and canals, and eventually into concrete-lined, flood-control channels (*fig. 11*; *pl. 1*). This routing provides the opportunity for modest precipitation to generate local runoff that leaves the area and becomes part of gaged outflow from the area. Routing of local runoff into unlined ditches and canals also provides the opportunity for some ground-water recharge, even in years with minimal precipitation.

Additional concepts used in creating *figure 18* include: more precipitation should prompt more evapotranspiration, as a quantity if not as a percentage; the change in slope of total annual precipitation occurring at about 23 in/yr seemed a reasonable threshold to begin adding direct recharge as a non-zero component; and recharge from local runoff should be a relatively small compared to local runoff.

Development of the four components of precipitation and their relative percentages, as illustrated in figure 18, was an iterative process. In addition to data from previous investigations and the hydrologic concepts described above, the ground-water model described later in this report was used to evaluate the estimated quantities of direct recharge and recharge from local runoff. Outflow simulated by the ground-water flow model was compared to gaged surface-water outflow less local runoff, less wastewater discharge. This evaluation process did not significantly alter the initial assumptions about precipitation components, but did help to confirm the reasonableness of the estimates. The final data used to calculate the values shown in *figure 18* are listed in *table 4*.

During 1945–98, total precipitation on the Bunker Hill and Lytle Creek basins averaged about 109,000 acre-ft/yr, and ranged from a minimum of about 41,000 acre-ft to a maximum of about 236,000 acre-ft (table 4). Local runoff averaged about 16,000 acre-ft/yr, and annual values ranged from about 6,000 acre-ft to about 35,000 acre-ft. Recharge from local runoff averaged about 5,000 acre-ft/yr, and annual values ranged from about 2,000 acre-ft to about 12,000 acre-ft. Evapotranspiration averaged about 86,000 acre-ft/yr, and annual values ranged from about 33,000 acre-ft to about 177,000 acre-ft. Direct recharge averaged about 1,000 acre-ft/yr, and annual values ranged from zero to about 12,000 acre-ft. Because of the method of estimating annual values, the maximum values occurred in 1983 and minimum values occurred in 1947.

Despite relatively simple assumptions, the methodology for calculating the four components of precipitation seemed to be useful and capture much of the hydrologic impact of precipitation on the basins. Undoubtedly, the real physical processes are much more complex, and, depending on antecedent conditions, actual component values may not be the same, even in years with the same annual precipitation. For example, it seems likely that increasing urbanization (*fig. 8*) is having some effect on local runoff. The California Department of Water Resources (1971, fig. 27) estimated that local runoff would double between 1965 and 1995, increasing outflow from the basin by about 10 percent. Although not visually apparent from a graphical analysis of cumulative gaged outflow (*fig. 15*), this estimated change may help to explain the increase in outflow from period II (0.37) to period IV (0.42), an increase of about 10 percent. Subsequent investigators may want to revisit the effect of urbanization on local runoff and ground-water recharge from local runoff, and consider adding a temporal adjustment to the percentages used in this report.

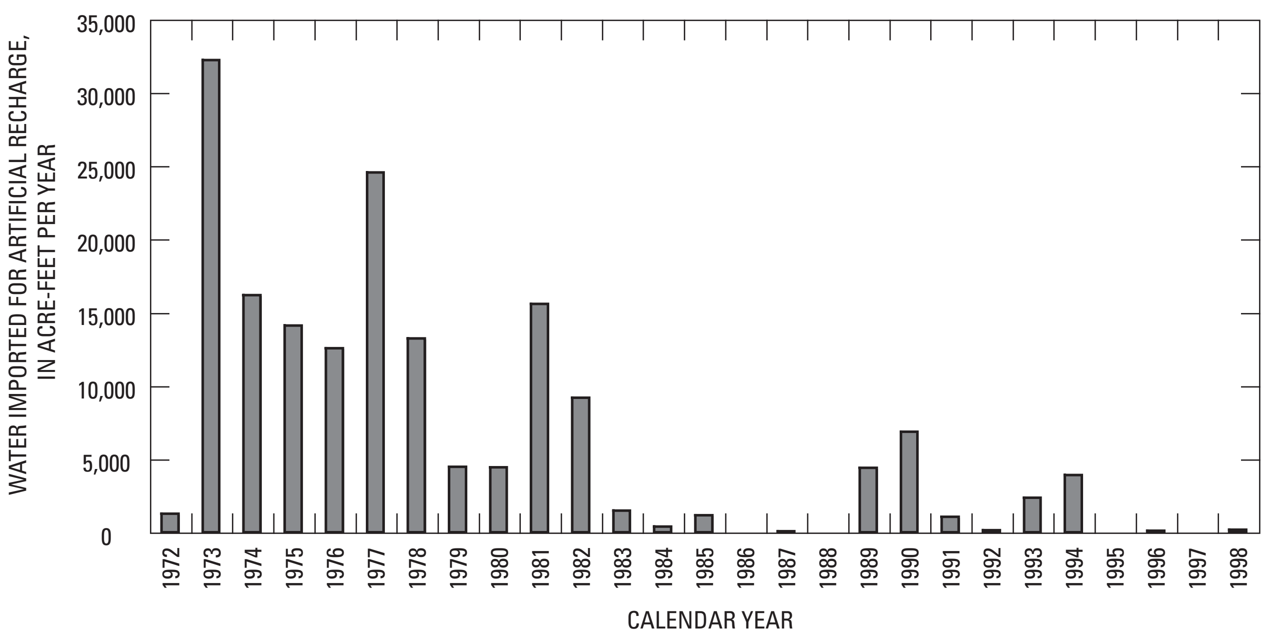


Figure 19. Water imported for artificial recharge into the San Bernardino area, California, 1972–98.

Imported Water

Beginning in 1972, water has been imported into the San Bernardino area to augment local supplies. This water, which originates as runoff into northern California streams, is collected by the State Water Project, conveyed south in the California Aqueduct, and distributed to member agencies of the State Water Project for use in central and southern California. As the local representative of the State Water Project, the San Bernardino Valley Municipal Water District has an entitlement to import as much as 102,600 acre-ft/yr (U.S. Bureau of Reclamation, 1989, table 1). In some years with abundant runoff in the Sierra Nevada, this entire quantity can be delivered. However, major facilities planned in 1963 to become part of the State Water Project were still not complete as of 1998. As a result, the longterm average quantity of imported water that can be delivered is significantly less than the maximum entitlement—probably less than 50 percent of the 102,600 acre-ft/yr (Camp Dresser and McKee, 1991, p. 2–4). By the end of the 7-year drought from 1986 to 1992, which affected water supplies throughout California, the maximum quantity of imported water available for the San Bernardino area was about 12,000 acre-ft/yr.

Water that is imported by the San Bernardino Valley Municipal Water District arrives at the Devil Canyon power plant (*fig. 11*; *pl. 1*), is kept separate from native runoff in a lined afterbay, and is distributed to local purveyors such as other water districts and nearby cities. From the afterbay, water can be diverted either east into the Foothill pipeline or west into the Lytle Creek pipeline and then can be released at any of the several artificial-recharge basins located mostly along the foot of the San Bernardino Mountains (*fig. 11*).

Prior to 1983, most imported water was used to recharge the Bunker Hill ground-water basin (*table 5*). After 1983, most of the imported water was used for agriculture in the Bunker Hill basin as part of the San Bernardino exchange plan (San Bernardino County, 1976). Under this plan, water from the Santa Ana River is diverted east to the Mill Creek area, or water from the Santa Ana River or Mill Creek is diverted out of the Bunker Hill basin to the Yucaipa basin (*fig. 2*). In exchange, the same quantity of water is imported and delivered to those, primarily in agriculture, who would have used the native runoff.

This marked change in the quantity of imported water used for artificial recharge is illustrated in *figure 19*. The reduction also was prompted by rising ground-water levels in the former marshland. By 1984, ground water had flooded the basement of the San Bernardino Valley Water District (located about equidistant between gaging stations J and K on *fig. 11*) and had damaged equipment in the post office across the street. In 1986, the city of San Bernardino sued the San Bernardino Valley Water Conservation District alleging excessive artificial recharge caused the high ground-water levels and related damage to public infrasture. Since 1983, a small quantity of imported water has been delivered within the San Bernardino area for direct municipal use. Imported water delivered outside the San Bernardino area has been used primarily for ground-water recharge in the adjacent Rialto-Colton basin (*fig. 11*; Woolfenden and Kadhim, 1997).

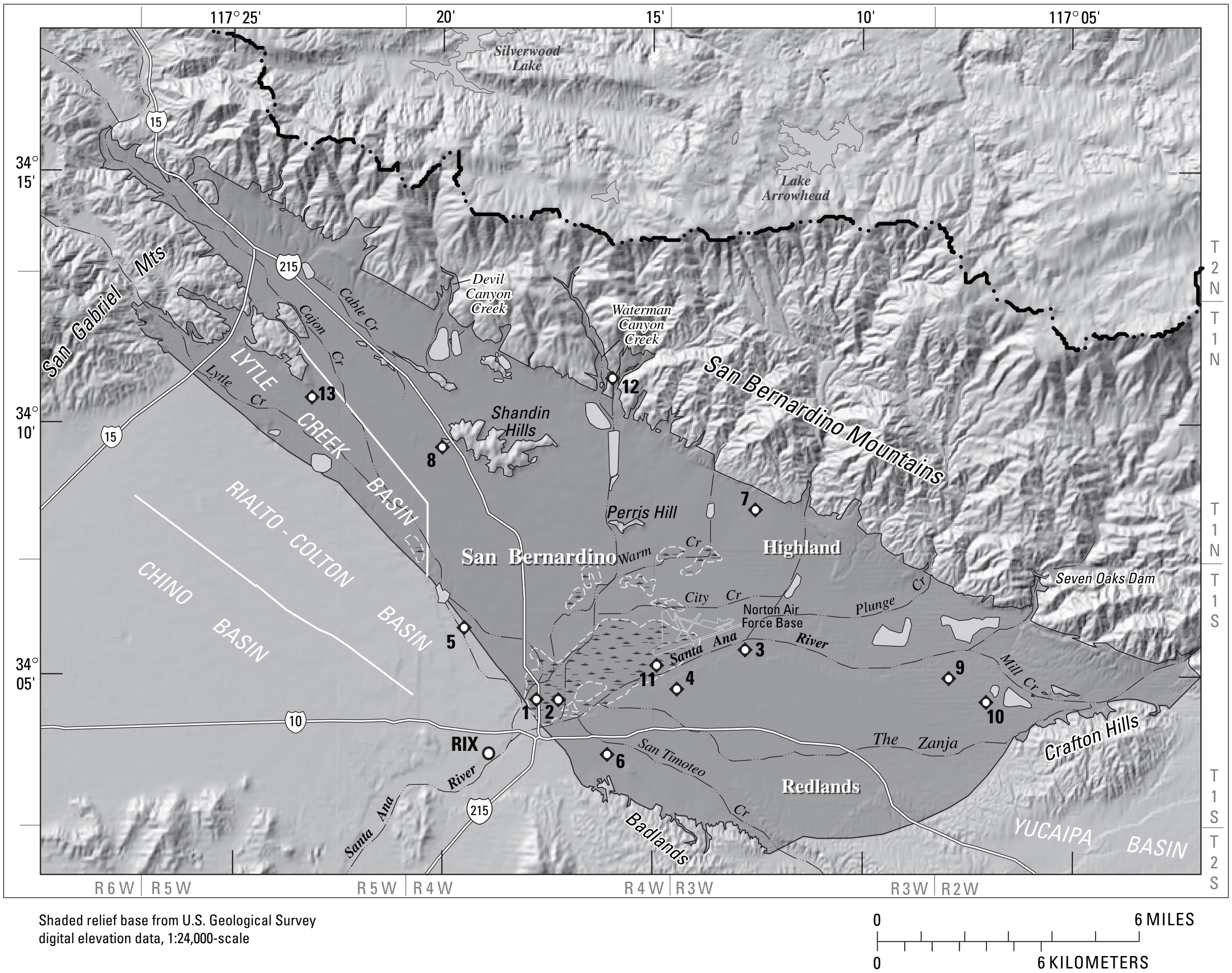
The quantities, distribution, and use of imported water for 1972–98 are summarized in *table 5*. Average annual values for imported water are listed for the period of water deliveries (1972–98) and for the primary water-budget period (1945–98). The total quantity of imported water averaged about 10,000 acre-ft/yr for 1972–98 and 5,000 acre-ft/yr for 1945–98. The quantity of water imported for artificial recharge in the Bunker Hill and Lytle Creek basins averaged about 60 percent of the total: about 6,000 acre-ft/yr for 1972–98 and about 3,000 acre-ft/yr for 1945–98.

The economic aspects of imported water are complex and highly variable, but essentially involve an annual fixed cost for the entitlement to import water from the State Water Project and a variable cost for the quantity of water imported. In 1998, the fixed cost was $14,300,000. The variable cost is based on a net energy requirement of 3,236 kilowatt-hours to bring 1 acre-ft of water from San Francisco Bay into the San Bernardino area via the Devil Canyon powerplant. In 1998, electricity purchased from the State Water Project cost $0.03 per kilowatt-hour, or about one-third the retail cost. At this reduced electrical rate, the variable cost was about $100 per acre-foot of imported water. As illustrated by this basic example, costs for importing water often are more complicated than for using native water because (1) electrical costs vary depending on when, how, and from whom the electricity is obtained; (2) reimbursements can be obtained from either local water purveyors or through increased property-tax assessments; and (3) distribution of water may involve a paper transfer of water rights, not a physical conveyance.

Wastewater Discharge

Since the 1930's, at least 4,000 acre-ft/yr of wastewater has been discharged into the San Bernardino area from 13 facilities (San Bernardino Valley Municipal Water District, c1977). Locations of these facilities are shown on *figure 20*, and their general characteristics are listed in *table 6*. Most of the smaller facilities treat industrial waste from a local business or sewage waste from a hospital or campus, and then discharge the waste to nearby evaporation or percolation ponds.

The city of Redlands also disposes of treated sewage effluent to a percolation pond, but the quantity is much greater (*table 6*). Although the pond is close to the Santa Ana River, overflow from the pond into the river has never occurred (M.L. Huffstutler, Redlands Water Department, oral commun., 1996). It is not known how much of the wastewater discharge to the pond evaporates and how much recharges the underlying ground-water system.



EXPLANATION

Basin boundary—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray

Fault or ground-water barrier—May be concealed or approximately located

Former marshland

Artificial-recharge basin

Boundary of Santa Ana River drainage basin

1 **Wastewater facility and number**—Refer to table 6. Locations approximate

RIX **Rapid infiltration and extraction (RIX) facility**—Location of City of San Bernardino wastewater discharge beginning in 1996

Figure 20. Location of major wastewater treatment facilities in the San Bernardino area, California.

Four facilities have discharged wastewater into streams or canals that are tributary to the Santa Ana River. Annual discharge from these facilities for the period 1945–98 is listed in *table 7*. The city of San Bernardino plant 1, which discharged directly into Warm Creek, closed in 1972 and all treatment was shifted to plant 2. In 1965, wastewater from the Loma Linda Sanitation District also was shifted to plant 2. As much as 25 percent of the discharge from plant 2 is used for landscape irrigation (1992); however, most of the treated wastewater is discharged directly into the Santa Ana River. Beginning in 1996, all wastewater from plant 2 was piped to a Rapid Infiltration-Extraction (RIX) facility downstream of the San Jacinto fault (*fig. 20*). Wastewater from the Southern California Edison, San Bernardino plant also is discharged into the Santa Ana River or to nearby canals that convey water to the Riverside area (*pl. 1*).

In the semiarid climate of the San Bernardino area, most wastewater discharged to small evaporation or percolation ponds probably evaporates or is transpired by vegetation. Initial pond design and subsequent siltation minimize percolation to the ground-water system. Some wastewater discharged to streams or canals is evapotranspired or recharges the ground-water system, but most probably flows out of the San Bernardino area. The relatively short distance from the largest discharge facilities (1 and 2, *fig. 20*) to the edge of the San Bernardino area limits their recharge potential. Percolation also is limited in this area by relatively high ground-water levels. During time periods when the water table is high, potential recharge is rejected; during periods when the water table falls below the stream stage, such as during the 1960's, some ground-water recharge can occur.

Since 1945, continued urbanization of the San Bernardino area has resulted in an increased volume of municipal sewage. For example, wastewater discharge from the city of San Bernardino has increased steadily from about 7,000 acre-ft/yr in 1945, to more than 48,000 acre-ft/yr in 1998 (*table 7*). Prior to 1996, most of this wastewater was discharged immediately upstream from gaging stations that measure outflow from the San Bernardino area (*fig. 11* and *table 1*).

To determine whether the increasing wastewater discharge affects the analysis of surface-water inflow and outflow presented in *figure 15*, the total quantity of wastewater discharge (*table 7*) was subtracted from total basin outflow (*table 1*), and the data in *figure 15* were replotted. Results indicate that the curve remains similar in shape, indicating that another, persistent hydrologic process controls much of the relation between surface-water inflow and outflow. Increasing urbanization, local runoff, and wastewater discharge likely affect the quantity of basin outflow, but how much surface-water inflow is retained in the basin likely is related to the depth to ground water.

Ground-Water Discharge into Warm Creek

Until about 1959 and between about 1980 and 1990, Warm Creek flowed perennially in the reach from south of Perris Hill to the San Jacinto fault (*figs. 2* and *9*). Much of the discharge resulted from ground water seeping into the creek channel whenever the water table was sufficiently high. During the period from about 1980 to 1990, ground water was observed flowing up through cracks and holes in the concrete-lined, flood-control channel of Lytle Creek near its confluence with Warm Creek (*fig. 11*; *pl. 1*). Not surprisingly, this quantity of discharge was observed to be significantly less than the discharge in the earthen channel of Warm Creek. Curiously, no ground-water discharge was observed in the Santa Ana River (J.C. Bowers, U.S. Geological Survey, oral commun., 1992).

In an effort to determine the quantity of "rising ground water," the California Department of Water Resources (1971, p. 150, table 22, column 1) measured discharge in Warm Creek and other discharges to, and withdrawals from, the creek. The resulting quantity of ground-water discharge was shown graphically to correspond to water levels in a nearby well (1S/4W–1M1) (California Department of Water Resources, 1971, fig. 7). This relation between ground-water level and discharge has been quantified by fitting a nonlinear regression equation to the discharge data and water levels in a nearby well (1S/4W–3Q1), referred to locally as the Heap well (*fig. 21*). The Heap well was used for the analysis because it has a longer period of record. As shown in *figure 21*, water-level measurements made in spring (March or April) were plotted against the corresponding annual discharge data calculated for water years 1943–59. The resulting regression equation is

$$Wl_{Heap,\,k} = 919.63 + 30.529 \log \left(Q^{RisingGW}_{WarmCrk,\,k}\right) \quad (2a)$$

or transposed

$$Q^{RisingGW}_{WarmCrk,\,k} = 10[(Wl_{Heap,\,k} - 916.63)/30.529] \quad (2b)$$

where

$Wl_{Heap,\,k}$ is average water-level altitude in the Heap well (1S/4W–3Q1) during year k, in ft, and

$Q^{RisingGW}_{WarmCrk,\,k}$ is discharge of ground water into Warm Creek during year k, in acre-ft/yr.

Equation 2b can be used to estimate likely values of annual ground-water discharge into Warm Creek for a range of ground-water levels in the Heap well. Discharge to the creek decreases to zero when ground water falls below an altitude of about 1,008 ft. When ground water rises to an altitude of about 1,065 ft, as it did in 1945, ground-water discharge rises to more than 40,000 acre-ft/yr. Estimated values of ground-water discharge for the period 1945–98 are listed in *table 8*.

Table 8. Annual ground-water discharge into Warm Creek in the San Bernardino area, California, 1945–98.

[Annual values estimated (e) using data from the California Department of Water Resources (1971, table 22, column 1, p. 150) and calculated (c) from a regression equation (fig. 21); average for 1945–98]

Calendar year	Ground-water discharge (acre-feet)	Calendar year	Ground-water discharge (acre-feet)	Calendar year	Ground-water discharge (acre-feet)	Calendar year	Ground-water discharge (acre-feet)
1945	42,000e	1959	500e	1973	0c	1987	4,300c
1946	38,200e	1960	0e	1974	100c	1988	2,500c
1947	35,600e	1961	200c	1975	100c	1989	1,200c
1948	28,500e	1962	100c	1976	100c	1990	900c
1949	24,000e	1963	100c	1977	100c	1991	300c
1950	17,800e	1964	0c	1978	100c	1992	100c
1951	13,500e	1965	0c	1979	200c	1993	100c
1952	14,300e	1966	0c	1980	1,000c	1994	100c
1953	8,700e	1967	0c	1981	2,400c	1995	100c
1954	5,600e	1968	0c	1982	5,100c	1996	200c
1955	3,800e	1969	0c	1983	8,500c	1997	200c
1956	1,700e	1970	0c	1984	10,100c	1998	200c
1957	1,200e	1971	0c	1985	9,400c		
1958	1,300e	1972	0c	1986	6,700c	Average	5,393

Diversions

Since the 1800s, canals and pipelines have been constructed to divert water from streams in the San Bernardino area. Typically, diversions are located near the mountain front, and flow is distributed to lower altitudes for agricultural and municipal use (*pl. 1*). For example, flow from the Santa Ana River is diverted to the city of Redlands and to farmlands between the river and the San Bernardino Mountains. Much of the flow in streams emanating from the San Bernardino Mountains is diverted into nearby spreading ponds to control floods and to enhance recharge of the ground-water system (*fig. 11*).

The Foothill pipeline follows the edge of the San Bernardino Mountains and typically conveys water east from the California Aqueduct (*fig. 11*). Because of its dual-end, gravity design, however, the Foothill pipeline can convey water west in the opposite direction. Capacity of the pipeline to convey water east is about 290 ft^3/s; capacity to convey water west is about 100 ft^3/s, or about the average flow in the Santa Ana River. As of 1998, water from the Santa Ana River can be transported west to artificial-recharge basins (*pls. 1* and *2*), though it rarely is. When Seven Oaks Dam was completed in 1999, the quantity of water available to transport to the west was increased, subject to legal decisions, as a result of storage behind the dam.

The numerous diversions in the San Bernardino area are shown planemetrically on *plate 1* and schematically on *plate 2*. These illustrations were prepared from detailed information obtained during 1990–94. Except for Seven Oaks Dam, which was being planned at the time of the research, the information on *plates 1* and *2* is believed to accurately represent conditions in 1998. The two plates document the surface-water operation of the basin, and provide detailed information for constructing a numerical model that routes surface-water through the San Bernardino area.

Ground-Water System

Ground water in the San Bernardino area occurs primarily in the valley-fill sediment. The small quantity of ground water that is found in, or moves through, consolidated and crystalline rocks surrounding and underlying the valley fill was assumed to be negligible for purposes of this report.

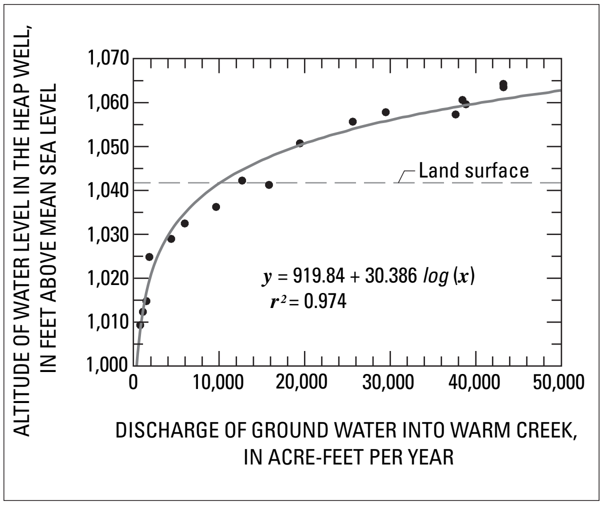


Figure 21. Relation between discharge in Warm Creek and ground-water level in the San Bernardino area, California. Measured discharge data for water years 1943–59 from California Department of Water Resources (1971, table 22, column 1, p. 150). Measured water levels for the Heap well (1S/4W-3Q1) from the USGS.

Description of Valley-Fill Aquifer

The valley-fill aquifer in the San Bernardino area was defined initially by Dutcher and Garrett (1963) and redefined for purposes of numerical simulation by Hardt and Hutchinson (1980). The extent of the valley-fill aquifer, as defined in this study and shown in *figure 22*, includes the Bunker Hill and Lytle Creek basins as defined by Dutcher and Garrett (1963, pl. 4). Boundary conditions for the valley-fill aquifer and the general direction of ground-water flow also are shown in *figure 22*.

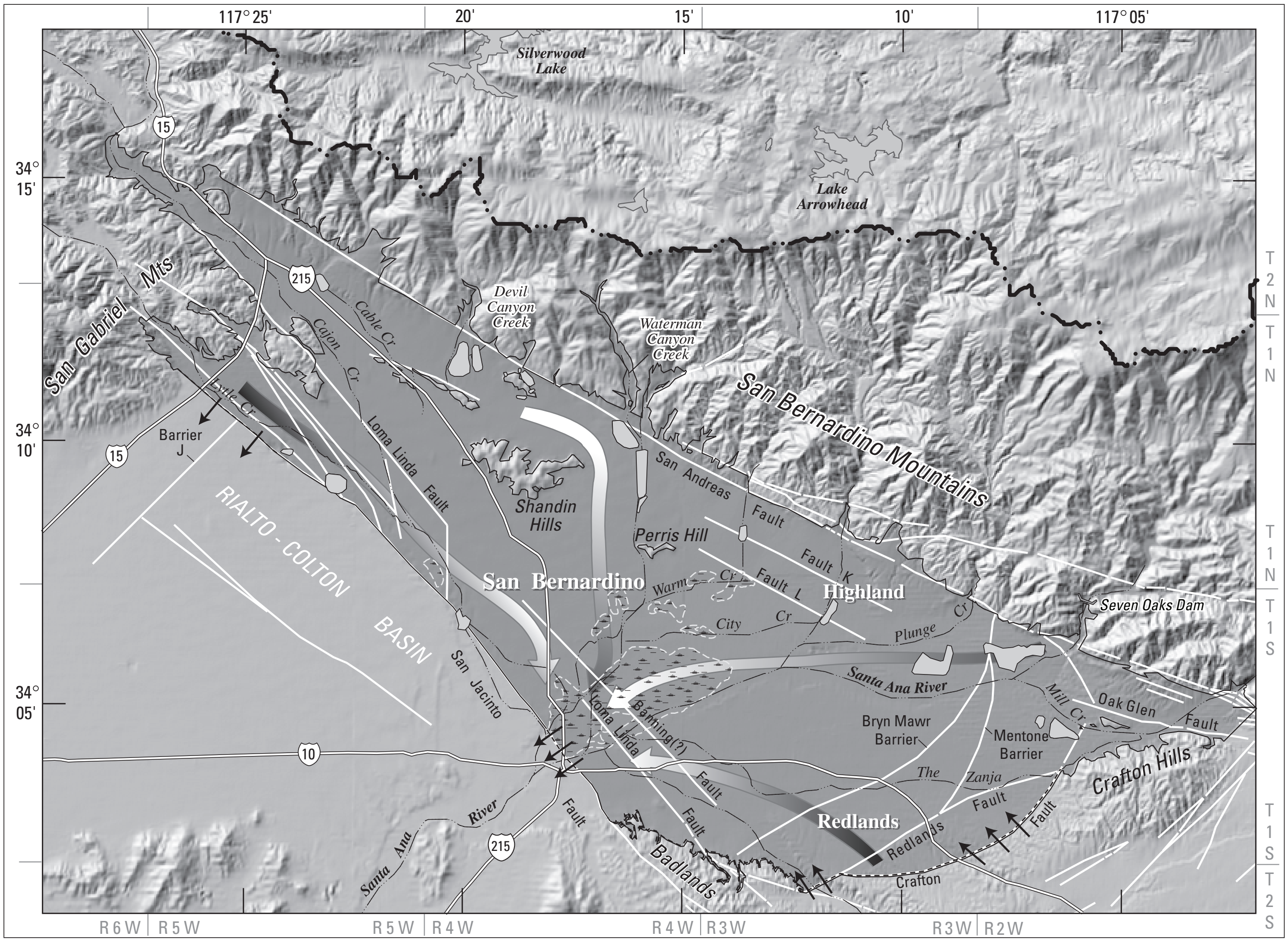
The valley-fill aquifer includes both unconsolidated deposits and sedimentary rocks. The unconsolidated deposits, which constitute the primary reservoir for storing large quantities of water, are composed of gravel, sand, silt, and clay. This sediment was formed mostly by alluvial fans coalescing along the mountain front, and by the Santa Ana River and Lytle Creek reworking and redepositing these materials. Near the mountain front, the unconsolidated deposits tend to be coarse grained and poorly sorted, becoming finer grained and better sorted downstream from the mountains. Zones of well sorted sand and gravel, where saturated, yield copious quantities of water to wells. Yields of 4 ft^3/s are common from municipal wells ranging in diameter from 14 to 20 inches.

The unconsolidated deposits have been divided further by Dutcher and Garrett (1963, pl. 1) into older (Pleistocene) and younger (Holocene) alluvium and Holocene river-channel deposits (*fig. 5*). The older alluvium (Qoal) consists of continental, fluvial deposits ranging in thickness from a few tens of ft to more than 800 ft. The younger alluvium (Qyal) is about 100-ft thick, composed mainly of flood-plain deposits. The thin river-channel deposits (Qrc) are among the most permeable sediments in the San Bernardino area and cause large seepage losses from streams. Hydraulic conductivity for these deposits ranges from about 40 to 100 ft/d (Dutcher and Garrett, 1963, p. 51–56). These stratigraphic units are described more fully in *figure 23*, which includes a representative geologic section and stratigraphic column for the San Bernardino area.

The sedimentary rocks (QTc) crop out mainly in the southern part of the San Bernardino area between the San Jacinto fault and Crafton Hills and underlie unconsolidated deposits in the vicinity of Redlands (*fig. 5*). In the badlands, these sedimentary rocks are referred to as the San Timoteo Formation and are composed of partly lithified, non-marine, alluvial and lacustrine sediment, ranging in age from late Tertiary to early Quaternary (*fig. 23*). Well yields are moderate from the more permeable layers—generally less than 1 ft^3/s; hydraulic conductivity ranges from 7 to 29 ft/d (Dutcher and Garrett, 1963; Dutcher and Fenzel, 1972). Small areas of sedimentary rocks also are present along the edge of the San Bernardino Mountains, but in these areas the deposits are not considered to be part of the aquifer system because they probably are mostly unsaturated and are underlain by less permeable consolidated rock.

The greatest thickness of water-bearing, unconsolidated deposits and sedimentary rocks in the valley-fill aquifer is more than 1,200 ft and occurs adjacent to the northeast side of the San Jacinto fault between the city of San Bernardino and the Santa Ana River (Fife and others, 1976). This area coincides with the former marshland (*fig. 2*). Upslope from the former marshland, the valley-fill deposits become progressively thinner northwest toward the San Gabriel Mountains, north toward the San Bernardino Mountains (*fig. 23*), and northeast toward Crafton Hills (Hardt and Hutchinson, 1980).

The valley-fill aquifer has been divided by Dutcher and Garrett (1963, pl. 7) into six hydrogeologic units: an upper confining member (UCM), an upper water-bearing zone (UWB), a middle confining member (MCM), a middle water-bearing zone (MWB), a lower confining member (LCM), and a lower water-bearing zone (LWB). These hydrogeologic units are shown in *figure 24*, using the same section (*A–A'*) as in *figure 23*. The hydrogeologic interpretation in this report is similar to that suggested by Dutcher and Garrett (1963), but relies on new data from lithologic logs (*fig. 23*), geophysical logs (*fig. 24*), and multiple-depth piezometers. This additional data gives increased accuracy and reliability to the hydrogeologic interpretation of the valley-fill aquifer compared to the work of Dutcher and Garrett (1963), who had to rely almost exclusively on lithologic data from driller's logs and water-level data from production wells.



Shaded relief base from U.S. Geological Survey digital elevation data, 1:24,000-scale

EXPLANATION

Boundary of the valley-fill aquifer—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray. Solid line indicates no underflow. Dashed line indicates area of minor underflow; arrows indicate area with more than 1,000 acre-feet per year of underflow and point in the direction of ground-water flow to or from adjacent permeable materials

Fault or ground-water barrier—May be concealed or approximately located

Former marshland

Artificial-recharge basin

Generalized direction of ground-water flow—Within the valley-fill aquifer

Boundary of Santa Ana River drainage basin

Figure 22. Areal extent of the valley-fill aquifer, boundary conditions, and direction of ground-water flow in the San Bernardino area, California.

The upper confining member is a near-surface deposit with low hydraulic conductivity. The member is thin and discontinuous, and may be absent, thinner, or much more permeable near Warm Creek (Dutcher and Garrett, 1963). In the area between Warm Creek and the Santa Ana River, the upper confining member acts to restrict vertical flow, causing semi-confined conditions in the upper 50 to 100 ft of saturated materials. As shown in *figure 24*, the upper confining member is effectively at land surface between the San Jacinto fault and the Banning (?) fault.

In places, the upper confining member appears to have been eroded by streamflow and replaced with coarse sand and gravel. Boreholes drilled to a depth of about 50 ft below land surface in the vicinity of the Santa Ana River and the San Jacinto fault indicate a predominance of coarse sand and gravel, not fine-grained silt and clay. In these locations, the coarse material is essentially part of the upper water-bearing unit, vertical ground-water flow is less restricted, and unconfined conditions are likely to be present throughout the upper 100 to 200 ft of valley-fill sediment.

North of the Banning (?) fault, the slope of the land surface increases and a more permeable deposit, considered to be part of the upper water-bearing zone, overlies the upper confining member. This overlying deposit appears to be the result of aggrading alluvial fans being deposited over the finer-grained upper confining member (*fig. 24*).

The upper and middle water-bearing zones provide most of the water to municipal and agricultural wells. In the central part of the San Bernardino area, these zones are separated by as much as 300 ft of interbedded silt, clay, and sand (the middle confining member). This middle confining member produces confined conditions over the central part of the basin, but thins and becomes less effective toward the margins of the basin (Dutcher and Garrett, 1963). In the area where the middle confining layer is effective, it is referred to locally as the confined area (Mendenhall, 1905; Dutcher and Garrett, 1963; *fig. 1*). The areal extent of the confined area is approximately the same as the areal extent of flowing wells mapped by Mendenhall (1905) and also about the same as the areal extent of the upper confining member (*fig. 24*). The similarity among these three areas has caused several previous publications to cite them incorrectly or to use them interchangeably. Analysis of most hydrologic conditions, particularly those described in this report, requires that each area be considered separately.

Although not as permeable as the adjacent water-bearing zones, the middle confining member does yield significant quantities of water to wells. As a result, most production wells have casing that is open opposite one or both of the water-bearing zones, and open opposite the middle confining member. As suggested by well yields and geophysical logs (*fig. 24*), relatively continuous zones of silt and sand are present in the middle confining member. Perhaps because of its name, some previous studies have tended to characterize the middle confining member as a impermeable mass of clay and silt, which it is not.

The lower confining member and lower water-bearing zone are not penetrated by most production wells and play a lesser role in the valley-fill aquifer. When penetrated by a production well, the lower confining member is used to increase the yield of the well. Measurements of vertical flow within three such production wells distributed along section *A–A′*, however, showed that very little water was contributed to the any of the three wells from the lower confining member (Izbicki and others, 1998, fig. 3).

The lower water-bearing zone rarely is tapped by production wells, mainly because it is much slower to drill through than the overlying deposits. The lower water-bearing zone may be composed of poorly consolidated or partly cemented older alluvium (Qoal), or may be composed solely of even older sedimentary rocks (QTc). In either case, the top of the lower water-bearing zone forms the effective bottom of the ground-water flow system within the valley-fill aquifer, at least in the vicinity of section *A–A′*.

Recharge and Natural Discharge

Sources of recharge in the San Bernardino area are seepage from gaged streams, seepage from ungaged runoff, direct infiltration of precipitation, recharge from local runoff that originates as precipitation on the Bunker Hill and Lytle Creek basins, artificial recharge of imported water, return flow from pumpage, and underflow from adjacent ground-water areas. Ground water is discharged naturally by subsurface underflow out of the area, by ground water flowing into the lower reaches of Warm Creek, and by evapotranspiration.

Recharge from Gaged Streamflow

Seepage from gaged streams is the major source of recharge in the San Bernardino area. Recharge occurs both in the stream channels and in nearby artificial-recharge basins (*fig. 11*). The exceptionally coarse and highly permeable materials in the stream channels increase the percentage of streamflow that recharges the valley-fill aquifer. Equally important for enhancing recharge is the standard operation of routing streamflow through each artificial-recharge basin in a serpentine pattern designed to maximize residence time in the basin (K.J. Mashburn, San Bernardino County Flood Control, oral commun., 1990). Commonly, large earthen dikes are used to form the sides of a basin and effectively contain streamflow within the basin. Smaller earthen dikes, locally referred to as "sugar" dikes, are bulldozed into a series of rows that impede low-to-average streamflow and create the serpentine pattern of flow. During higher flows when stream stage reaches a certain altitude, the sugar dikes are designed to break apart, "dissolving" like sugar into the higher flow. Flood water then is conducted safely straight through the artificial-recharge basin and back into the main stream channel.

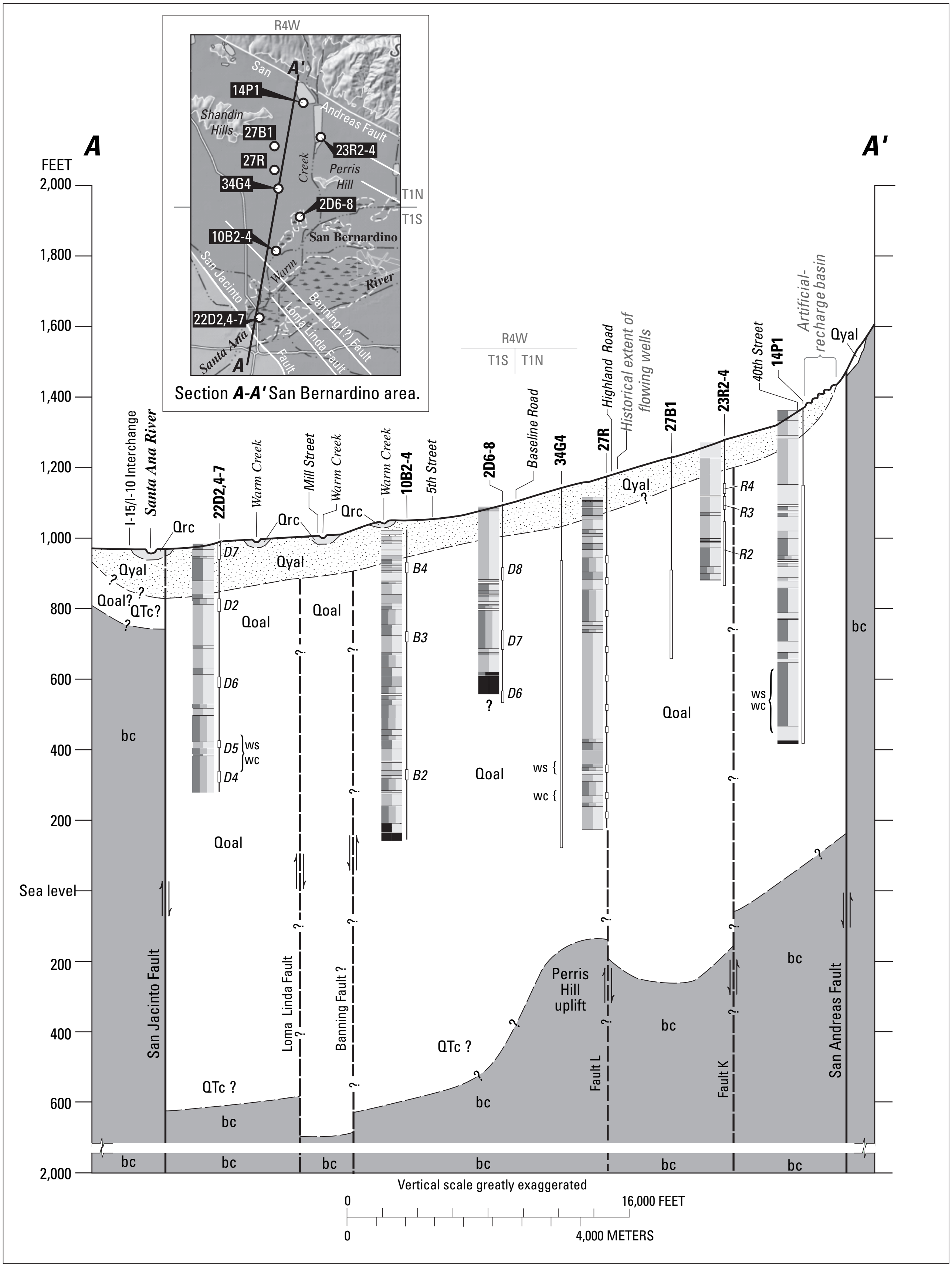


Figure 23. Representative geologic section *A–A′* and stratigraphic units in the San Bernardino area, California.

EXPLANATION FOR FIGURE 23

Stratigraphic units and their water-bearing characteristics

QUATERNARY

RECENT

Qrc **River-channel deposits**
Unconsolidated coarse gravel, sand, and silt in major river channels, generally becoming finer grained at greater distance from the San Bernardino and San Gabriel Mountains. Highly permeable, but largely unsaturated except near the San Jacinto Fault. Large quantities of water seep into these deposits from streams during runoff.

LOCAL UNCONFORMITY

Qyal **Younger alluvium**
Unconsolidated coarse gravel, sand, and clay, forming of a fine-grained and coarse-grained member, which in places are indistinguishable. Largely unsaturated near the mountains. Where present, the fine-grained member, which is 60 to 90 feet thick and composed mostly of silt and clay, overlies and confines the coarse-grained member. The coarse-grained member is composed mostly of gravel and sand, is highly permeable, and yields water copiously to wells.

LOCAL UNCONFORMITY

PLEISTOCENE

Qoal **Older alluvium**
Unconsolidated coarse gravel, sand, silt, and clay of continential, largely fluvial origin. In places, deeply weathered with red and yellow soil zones; often contains easily broken pebbles of dioritic and granitic gneiss; marker beds include a white clay and white sand at about 800 feet below land surface. Locally unconformably overlies crystalline bedrock; fractured by numerous faults and in places slightly folded. Contains thick, fine-grained silt and clay beds which confine underlying permeable members; some lower members are partly cemented. More permeable members yield water copiously to wells.

LOCAL UNCONFORMITY

TERTIARY

PLIOCENE

QTc **Quaternary and Tertiary sedimentary rocks**
Upper members are poorly consolidated to compacted gravel, sand, silt, and clay, locally containing a mammalian fauna and broken by numerous faults. Lower members are predominately brown to blue-green calcareous, indurated clays with locally discontinuous lenses of sand. Tertiary rocks, also non-marine, include compacted, cemented calcareous units, in places laminated with beds of conglomerate. Upper members yield water in moderate quantities to wells that penetrate permeable members; lower members are essentially non-water-bearing.

UNCONFORMITY

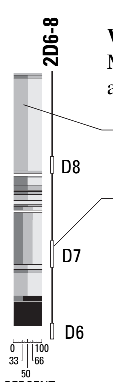
PRE-TERTIARY

bc **Basement complex**
Consolidated metamorphic and igneous rocks. Essentially non-water-bearing, though small quantities of water, including geothermal fluids, flow in fractures.

? **Fault—**Dashed where inferred; queried where uncertain. Arrows indicate relative movement

Contact between stratigraphic units—
Dashed where approximately located, queried where uncertain

Well design and lithologic units



Well and abbreviated state well number—
Multiple numbers (D6-8) represent piezometers at selected depths

Lithologic log composition—Queried where uncertain

- Gravel
- Sand
- Clay or silt
- Hard rock or hard drilling

ws White sand
wc White clay

Figure 23.—Continued.

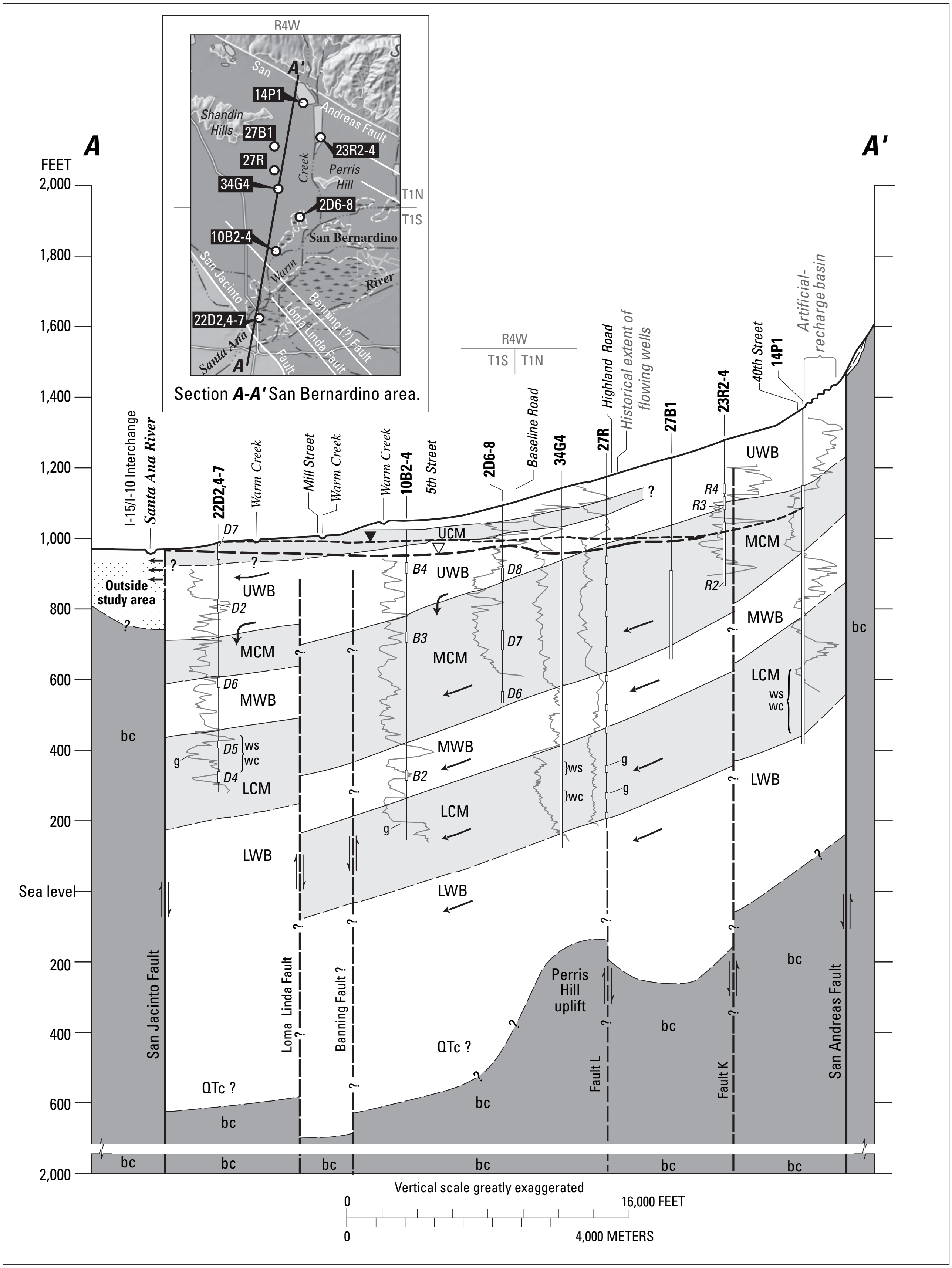


Figure 24. Section *A–A'* showing major hydrogeologic units, geophysical logs, and direction of ground-water flow in the San Bernardino area, California.

EXPLANATION FOR FIGURE 24

Unconsolidated deposits—Outside study area

bc **Bedrock basement complex of igneous and metamorphic rocks**—Composed of virtually impermeable rocks

Fault—Dashed where inferred; queried where uncertain. Arrows indicate relative movement

Contact between hydrogeologic units—Dashed where approximately located, queried where uncertain

Hydrogeologic units—Determined by lithology, geophysical logs, and hydraulic-head measurements. Naming convention follows Dutcher and Garrett (1963)

Hydrogeologic unit

- UCM Upper confining member
- UWB Upper water bearing
- MCM Middle confining member
- MWB Middle water bearing
- LCM Lower confining member
- LWB Lower water bearing
- QTc Sedimentary rocks

Potentiometric surface—Spring 1992

- Water table
- Composite hydraulic head for hydrogeologic units beneath UWB

Ground-water movement—

- Arrow indicates general direction of ground-water flow
- Underflow to adjacent permeable deposits

2D6-8 **Well and abbreviated state well number**—Multiple numbers (D6-8) represent piezometers at selected depths

D8

Perforated casing

D7 Geophysical log—Single point electrical resisitivity, increasing to the right; g, indicates a high reading from the natural gamma log

D6

Increasing resistivity

- ws White sand
- wc White clay

Figure 24.—Continued.

As a result of the highly permeable river-channel deposits (Qrc, fig. 5) and the artificial-recharge operations (*fig. 11*), nearly all flow in the smaller gaged streams (Devil Canyon, Waterman, East Twin, Plunge, and San Timoteo Creeks) is recharged to the aquifer close to the mountain front (*fig. 9*; Mendenhall, 1905, p. 48). During floods, the major streams (Santa Ana River, Mill Creek, and Lytle Creek) transmit large volumes of water in a short time, and some flow leaves the San Bernardino area, passing the outflow gages (*fig. 11*).

Recharge from all gaged streamflow can be estimated by subtracting total gaged surface-water outflow from total gaged surface-water inflow (*table 1*), then adding rising ground water (*table 8*), estimated local runoff (*table 4*), and wastewater discharge (*table 7*). This is not an ideal calculation because some of the rising ground water and wastewater discharge originated as seepage from gaged streams, but it does give a reasonably good estimate, especially for longer periods. Equally important, the calculation recognizes that gaged outflow is composed of at least four components: through-flowing gaged inflow, rising ground water, local runoff, and wastewater discharge. This means that if any of the components change over time, such as is likely for both local runoff and wastewater discharge, the perceived change in gaged surface-water outflow or in net gaged inflow minus outflow (table 1) can be misleading. An apparent increase in surface-water outflow may not result from a decrease in seepage of gaged surface-water inflow, or a decrease in ground-water recharge.

For 1945–98, estimated recharge from gaged inflow, as calculated above, averages about 116,000 acre-ft/yr (146,452 – 67,931 + 5,393 + 16,377 + 15,873). During this period estimated recharge from gaged inflow ranges from a minimum of about 27,000 acre-ft in 1961 to about 423,000 acre-ft in 1969.

Recharge from Ungaged Runoff

Recharge from ungaged runoff is of lesser importance because the total quantity of ungaged runoff is estimated to be only about 10 percent of gaged runoff (*tables 1* and *3*). However, virtually all ungaged runoff that flows into the San Bernardino area is assumed to recharge the valley-fill aquifer. Both the hard, impermeable surface of the surrounding bedrock and the short distance between the bedrock and the valley-fill aquifer minimize potential losses of ungaged runoff to evapotranspiration or to percolation into the bedrock.

For 1945–98, recharge from estimated ungaged runoff averaged about 16,000 acre-ft/yr (16,647 times 0.946; *table 3*). Because annual runoff from each ungaged area is assumed to vary linearly with gaged flow in the Santa Ana River (table 2), ungaged recharge also varies linearly with gaged flow in the Santa Ana River.

A significant quantity of ungaged recharge occurs as a result of ungaged runoff from the San Gabriel and San Bernardino Mountains, from Crafton Hills, and from the badlands (*fig. 17*; *table 3*). A much lesser quantity of recharge occurs as a result of ungaged runoff from bedrock outcrops within the ground-water basins, such as from Shandin Hills. Some ungaged runoff, such as from Crafton Hills and the badlands, may actually enter the valley-fill aquifer as underflow from adjacent ground-water areas. Although some previous investigators have lumped recharge from ungaged runoff with underflow (California Department of Water Resources, 1971; Dutcher and Fenzel, 1972; Hardt and Hutchinson, 1980), the two components are considered separately in this report.

Recharge from Direct Precipitation

Recharge from precipitation falling directly on the Bunker Hill and Lytle Creek basins probably is minimal in this semiarid region. In years of average and below-average precipitation, little or no recharge of this type occurs because of high potential evapotranspiration (Young and Blaney, 1942; Hardt and Hutchinson, 1980, p. 34). Even the small amount of precipitation that may infiltrate to the unsaturated zone is evaporated or transpired by vegetation. In exceptionally wet years, such as 1969 and 1983 (*fig. 14*), some direct recharge from precipitation probably does occur, although the quantity is unknown.

In other semiarid basins, researchers have estimated that infiltration of direct precipitation ranges from zero to about 0.05 ft/yr (Eychaner, 1983, p. 10; Danskin, 1988, 1998; Hollett and others, 1991, p. B59; Hanson and others, 1994, p. 41). For the San Bernardino area, a value of 0.05 ft/yr would be equivalent to about 4,000 acre-ft/yr. Nearly twice that value (8,400 acre-ft/yr) was estimated by the California Department of Water Resources (1986, tables 17,18). These values, particularly that by the California Department of Water Resources, likely include recharge from local runoff that was generated by direct precipitation.

As part of the process of determining local runoff for the San Bernardino area (refer to this report, pages 26–28), direct recharge from precipitation was estimated as a separate component of precipitation (*table 4*). During 1945–98, direct recharge is assumed to occur in only 6 years (1969, 1978, 1980, 1983, 1993, 1998). For 1945–98, this infrequent recharge equates to an average rate of about 1,000 acre-ft/yr. On the one hand, this amount is nearly inconsequential for the 54–year period; on the other hand, it accounts for about 30,000 acre-ft of recharge during 1978–83. If purchased from the State Water Project in 1998, this amount of water would have cost more than $3 million.

Recharge from Local Runoff

Recharge also occurs from local runoff generated from precipitation falling on the Bunker Hill and Lytle Creek basins. Precipitation, particularly that falling on impermeable urban surfaces, runs off, forming small creeks, pools, and ponds. Several small catchment basins have been constructed to collect this type of runoff from streets, railroad corridors, and university lands (San Bernardino County Flood Control District, 1987). This accumulation of precipitation into localized areas offers a greater potential for recharge than from direct precipitation. It seems likely that much of the recharge from direct precipitation, estimated by the California Department of Water Resources (1986, tables 17,18) to average 8,400 acre-ft/yr, actually is recharge from local runoff resulting from direct precipitation. Similarly, the significant recharge from direct precipitation during above-average runoff years, estimated by the California Department of Water Resources (1986, tables 17,18) to exceed 42,000 acre-ft/yr in 1980, probably is mostly recharge from local runoff.

These estimates of recharge were based on all precipitation falling on the land surface within the Bunker Hill and Lytle Creek basins (*fig. 1*) and on the ratio of pervious and impervious surfaces (California Department of Water Resources, 1971, p. 291). As urbanization of the San Bernardino area continues, the quantity of local runoff is estimated to double as a result of an increase in impervious surfaces, but the quantity of recharge from local runoff is expected to remain relatively constant (California Department of Water Resources, 1971, fig. 27; California Department of Water Resources, 1986, tables 17,18).

As described on pages 26–28, of this report, recharge from local runoff was estimated as a one of the components of precipitation (*fig. 18* and *table 4*). This methodology of separating different physical processes makes comparisons to previous estimates more problematic, but it facilitates simulating the processes as part of a surface-water and (or) ground-water model.

For 1945–98, recharge from local runoff averaged about 5,000 acre-ft/yr. Annual values ranged from a minimum of about 2,000 acre-ft in 1947 to a maximum of about 12,000 acre-ft in 1983. Based on these estimates, total recharge from precipitation during 1945–98 averaged about 6,000 acre-ft/yr (1,000 acre-ft/yr of direct recharge from precipitation plus 5,000 acre-ft/yr of recharge from local runoff). This combined value compares well with the estimate (8,400 acre-ft/yr) by the California Department of Water Resources (1986, tables 17,18) for a slightly larger area that includes the Yucaipa basin (*fig. 2*). The combined value also compares well with the estimate (4,000 acre-ft/yr) based on research in other semiarid basins, which uses an equivalent rate of 0.05 ft/yr (refer to "Recharge from Direct Precipitation" earlier on this page).

Artifical Recharge from Imported Water

Artificial recharge of imported water in the San Bernardino area began in 1972 and has occurred sporadically since then (*fig. 19*; *table 5*). Because of the extremely permeable sand and gravel deposits beneath the artificial-recharge basins, the maximum instantaneous recharge rates are high, generally 1 to 2 ft^3/s per acre of water surface, or about 2 to 4 ft/d. As a result, nearly all of the imported water probably percolates to the valley-fill aquifer. The percentage of percolation was estimated from previous recharge studies by Moreland (1972), Schaefer and Warner (1975), and Warner and Moreland (1972) and from personal observations by G.L. Fletcher (San Bernardino Valley Municipal Water District, oral commun., 1994). Based on this information, a recharge rate of 90 percent was assumed for all artificial recharge of imported water—relatively high compared to recharge rates for native runoff. This higher rate results from having a more controllable supply with lower turbidity. Based on the assumed recharge efficiency of 90 percent, the total quantity of artificial recharge from imported water in the Bunker Hill and Lytle Creek basins averaged about 6,000 acre-ft/yr for 1972–98 and about 3,000 acre-ft/yr for 1945–98.

In 1973, total recharge was much greater, exceeding 30,000 acre-ft/yr. An even greater quantity of imported water could be recharged along the base of San Bernardino Mountains because of the sizeable acreage of the several artificial-recharge basins and the exceptionally permeable materials. For example, the maximum recharge rate for the two large artificial-recharge basins on the Santa Ana River (*fig. 11*) exceeds 1,200 ft^3/s (California Department of Water Resources, 1971, table 46).

Whether these high recharge rates could be maintained for an extended period, particularly along the Santa Ana River, is not known. Nearby gravel pits likely would become flooded and might prompt a change in recharge operations. Also, it is not known whether large volumes of water recharged in the artificial-recharge basins would simply resurface as flow in the nearby Santa Ana River. Despite these caveats, the San Bernardino area is unusual compared to other semiarid basins: a large quantity of water can be imported from outside the area (*table 5*) and can be recharged in highly permeable artificial-recharge basins scattered across the valley-fill aquifer (*fig. 22*). This capability makes imported water a much more valuable source of recharge than its meager historical average of 3,000 acre-ft/yr might imply.

Return Flow from Pumpage

Return flow from pumpage is the quantity of total pumpage that is returned to the ground-water system. Water is returned as a result of percolation from agricultural irrigation and from some domestic and municipal uses. Although ground water is extracted from many different zones within the valley-fill aquifer depending on the construction of individual wells, return flow occurs only to the top of the aquifer via percolation from the land surface. The downward movement of return flow is restricted by the presence of fine-grained deposits, such as the upper confining member (UCM, *fig. 24*). These conditions near a production well can create a hydrologic paradox where water is pumped from lower zones of the aquifer at the same time that net recharge is occurring to the uppermost zone.

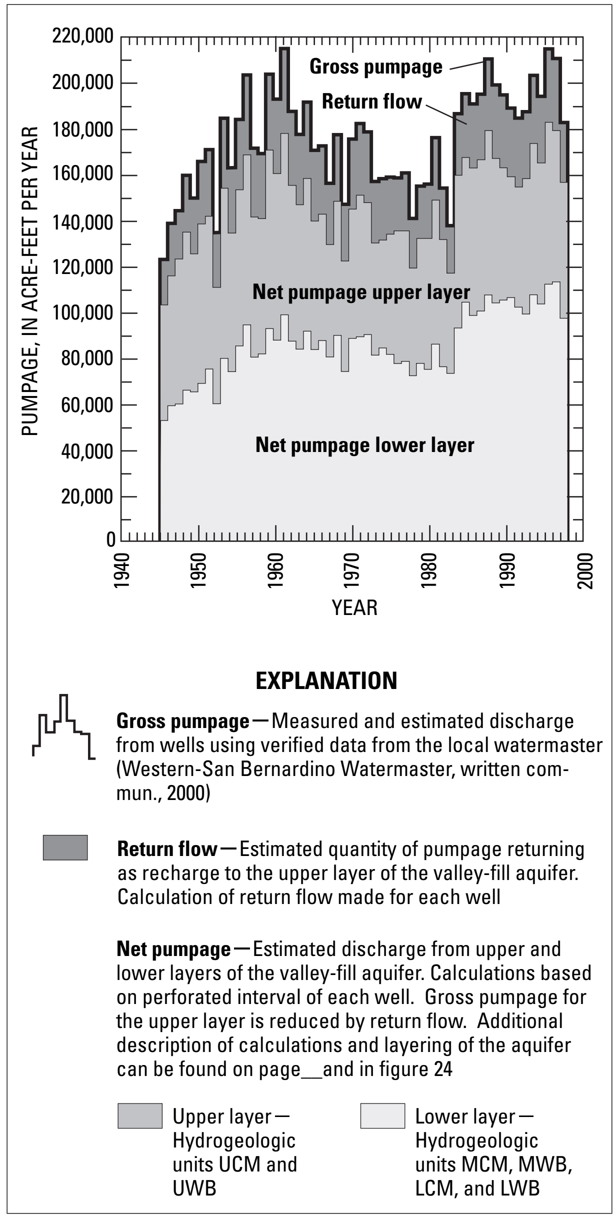


Figure 25. Annual ground-water pumpage for the San Bernardino area, California, 1945–98.

Return flow was estimated by Hardt and Hutchinson (1980, p. 35) to be 30 percent of total pumpage except for wells that export ground water directly out of the San Bernardino area, such as wells along Gage Canal near the confluence of the Santa Ana River and San Timoteo Creek (*pl. 1*). Wells used for export were assumed to have zero return flow. Annual values of return flow for 1945–98 were calculated for each well in the valley-fill aquifer using measured or estimated pumpage and an estimated percentage of return flow based on the work of Hardt and Hutchinson (1980) and on advice from the local watermaster (R.L. Reiter, San Bernardino Valley Municipal Water District, written commun., 2000).

Total values of gross pumpage, return flow, and net pumpage for each year for the San Bernardino area are shown in *figure 25*. For 1945–98, return flow averaged about 28,000 acre-ft/yr. Annual values ranged from about 20,000 acre-ft/yr to about 37,000 acre-ft/yr. Estimation of net pumpage, including net pumpage from upper and lower layers of the valley-fill aquifer, is described in greater detail in the following section entitled “Pumpage.”

Underflow

Underflow into the San Bernardino area occurs across the Crafton fault and through the poorly transmissive materials comprising the badlands (*fig. 22*). The short section of permeable deposits between the Crafton Hills and the San Bernardino Mountains is assumed to be a ground-water divide and to transmit essentially no underflow to or from the adjacent Yucaipa basin. Any underflow into the area from the San Bernardino or San Gabriel Mountains, such as that estimated by the California Department of Water Resources (1971), is assumed to be accounted for in recharge from either gaged or ungaged runoff. Underflow from the surrounding and underlying consolidated rock (basement complex, *fig. 23*) is assumed to be negligible.

Underflow across the Crafton fault and through the badlands as defined by Dutcher and Fenzel (1972, p. 29) averaged approximately 6,000 acre-ft/yr for 1945–65. As a result of increased pumpage east of the San Bernardino area in the Yucaipa basin (*fig. 2*), underflow across the Crafton fault decreased progressively from 8,150 acre-ft/yr in 1927 to 5,350 acre-ft/yr in 1967. More than half this underflow was assumed to occur in the vicinity of San Timoteo Creek. Underflow from the badlands was less than 300 acre-ft/yr. No underflow was estimated to occur north of Crafton Hills.

To develop annual values of underflow for use in the present study, prior estimates of underflow in the vicinity of the Crafton fault were re-evaluated and extended in time. Included in this re-evaluation were estimates done by the San Bernardino Valley Municipal Water District (1970, vol. IV, task 10–SY–6), by the California Department of Water Resources (1971), and by Dutcher and Fenzel (1972). As an aid in calculating underflow in a geologically complex area, Dutcher and Fenzel (1972, fig. 2) defined areas of underflow that represent eroded canyons and faulted subbasins between the Bunker Hill and Yucaipa basins. During the re-evaluation, these areas were used substantially as originally defined, but the actual location of underflow was moved to coincide with the boundary of the valley-fill aquifer as defined in this present study (*fig. 26*).

As described by Dutcher and Fenzel (1972, fig. 7), underflow for three areas (Reservoir Canyon, Redlands Heights, and the badlands) did not appear to change from 1927 to 1967. Average underflow was 450 acre-ft/yr for the Reservoir Canyon area, 300 acre-ft/yr for the Redlands Heights area, and 280 acre-ft/yr for the badlands. The assumption of constant underflow was based on Dutcher and Fenzel (1972) who observed virtually no change in ground-water levels in a few wells. The re-evaluation assumed that no further change in ground-water levels occurred after 1967; if it did, then some modification to the assumption of constant underflow is warranted. The quantity of underflow from the three areas is minimal, however, and any increase or decrease is unlikely to significantly affect ground-water flow in the Bunker Hill basin.

Underflow in two areas, San Timoteo Canyon and Sand Canyon, was assumed by Dutcher and Fenzel (1972) to decrease from 1927 to 1967. For San Timoteo Canyon, the decrease was nearly 50 percent; for Sand Canyon, the decrease was about 25 percent. For both areas, the decreased underflow was attributed to a steady decline in ground-water levels in the Yucaipa basin caused by pumpage. To account for continually changing underflow, prior estimates for San Timoteo Canyon and Sand Canyon (Dutcher and Fenzel, 1972; San Bernardino Valley Municipal Water District (1970, vol. IV, task 10–SY–6) were replotted and regression equations were fit to the values. These prior estimates and newly fitted equations are shown in *figure 27*. The equations are described below. Annual values of underflow calculated using these equations are listed in *table 9*.

In the vicinity of San Timoteo Creek, underflow can be estimated as

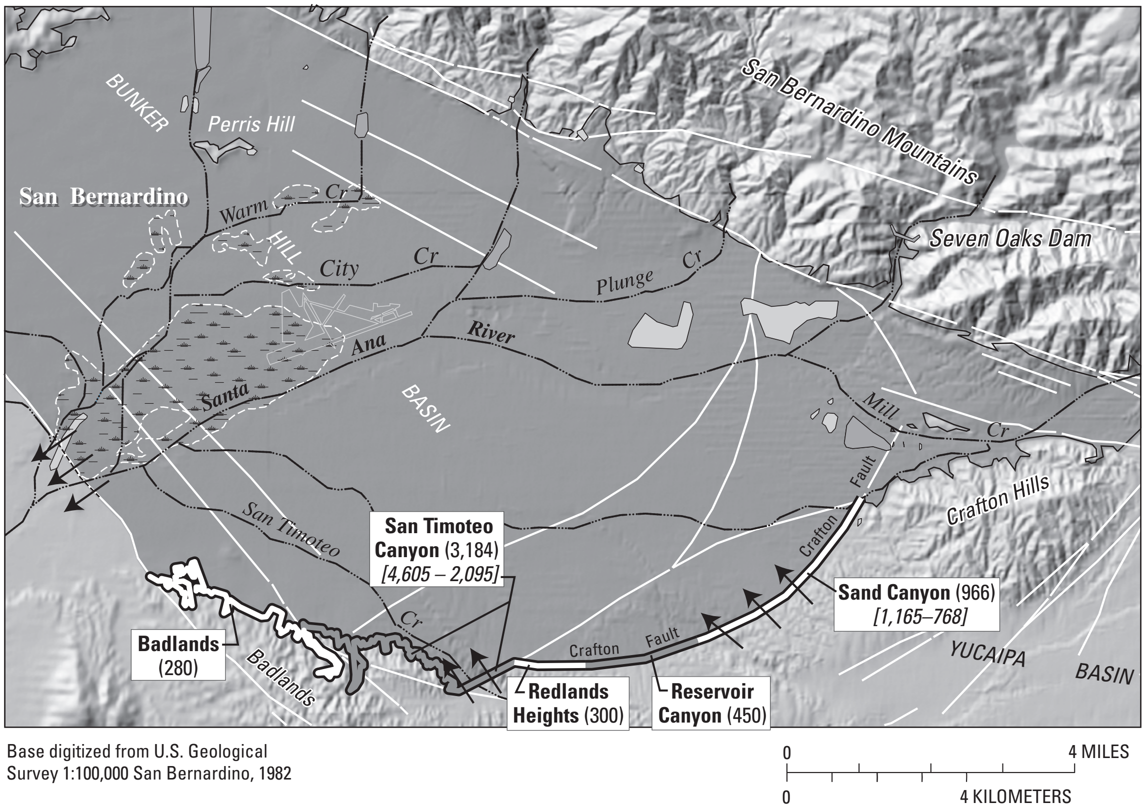
$$Q^{Underflow}_{SanTimoteo,\,k} = e^{[-29.309 \ln(k) + 230.4]} \qquad (3)$$

where

$Q^{Underflow}_{SanTimoteo,\,k}$ is underflow in acre-ft/yr in the vicinity of San Timoteo Canyon for calendar year k.

Underflow in the vicinity of San Timoteo Creek calculated from equation 3 averaged about 3,000 acre-ft/yr for 1945–98, and decreased from a maximum of almost 5,000 acre-ft/yr in 1945 to about 2,000 acre-ft/yr in 1998.

In the vicinity of Sand Canyon, underflow can be estimated as



Base digitized from U.S. Geological Survey 1:100,000 San Bernardino, 1982

EXPLANATION

Boundary of the valley-fill aquifer—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray. Solid line indicates no underflow. Dashed line indicates area of minor underflow; arrows indicate area with more than 1,000 acre-feet per year of underflow and point in the direction of ground-water flow to or from adjacent permeable materials

Fault or ground-water barrier—May be concealed or approximately located.

Former marshland

Artificial-recharge basin

Underflow—Estimated average underflow into the San Bernardino area for 1945–98 shown in parentheses. Bracketed values show decline in annual underflow from 1945 to 1998; refer table 9. Values in acre-feet per year

Sand Canyon

Reservoir Canyon

Redland Heights

San Timoteo Canyon

Badlands

Figure 26. Average underflow from permeable sediment near the Crafton fault into the San Bernardino area, California, 1945–98.

$$Q^{Underflow}_{SandCanyon,\,k} = -7.5k + 15{,}756 \qquad (4)$$

where

$Q^{Underflow}_{SandCanyon,\,k}$ is underflow in acre-ft/yr in the vicinity of Sand Canyon for calendar year k.

Underflow in the vicinity of Sand Canyon calculated from equation 4 averaged about 1,000 acre-ft/yr for 1945–98. Total decrease from 1945 to 1998 was about 400 acre-ft. Both approximations of underflow defined by equations 3–4 will remain valid for future years only if ground-water levels in the Yucaipa basin continue to decline.

Total underflow into the San Bernardino area for 1945–98 averaged about 5,000 acre-ft/yr (*table 9*). Annual values declined from a maximum of about 7,000 acre-ft in 1945 to about 4,000 acre-ft in 1998.

Underflow out of the San Bernardino area occurs across the San Jacinto fault and Barrier E in two locations—near the Santa Ana River and near Barrier J where Lytle Creek emerges from the San Gabriel Mountains (*figs. 5* and *22*) (Lu and Danskin, 2001).

Underflow near the Santa Ana River occurs only in the younger alluvium, which is about 100 ft thick (*fig. 23*). The river has eroded and redeposited these materials, removing most of the restriction to ground-water flow caused by movement of the San Jacinto fault (Dutcher and Garrett, 1963, p. 101). In the older, deeper deposits, fault gouge and offset of permeable zones restrict ground-water flow. For the period 1936–49, underflow ranged from 14,300 to 23,700 acre-ft/yr (Dutcher and Garrett, 1963, table 5). During water years 1935–60, underflow estimated using a mathematical model decreased from 16,900 acre-ft/yr in water year 1935 to 3,900 acre-ft/yr in water year 1960 (California Department of Water Resources, 1971, table 4). Average underflow for the period 1945–74 was about 15,000 acre-ft/yr (Hardt and Hutchinson, 1980, table 5).

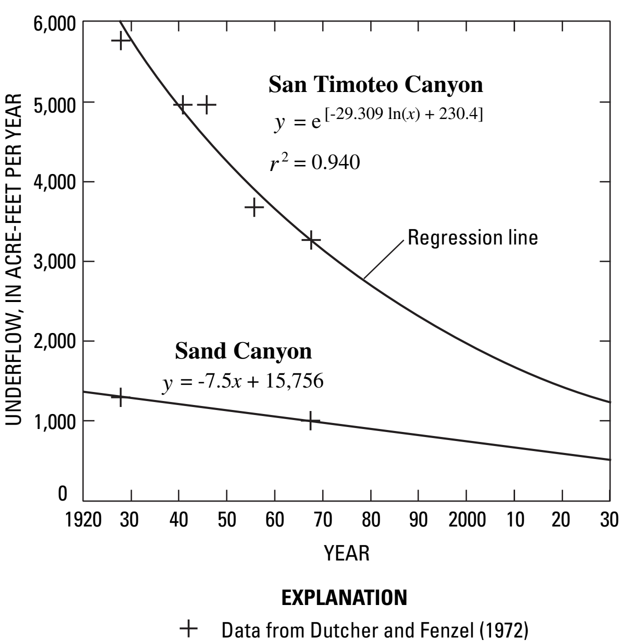


Figure 27. Estimated underflow from San Timoteo Canyon and Sand Canyon into the San Bernardino area, California.

Underflow near the Santa Ana River is mostly dependent on ground-water levels in the Bunker Hill basin. As ground-water levels in the Bunker Hill basin rise, more ground water is forced out of the basin as underflow. As ground-water levels fall, less ground water leaves the basin as underflow. Changes in ground-water levels in the Rialto-Colton basin appear to have a much lesser effect. Attempts in this study to correlate estimated values of underflow, such as those by the California Department of Water Resources (1971, table 4), to ground-water levels near the Santa Ana River in the Rialto-Colton basin, were largely unsuccessful. Instead, the estimated underflow correlates well with ground-water levels in the Bunker Hill basin, as illustrated in *figure 28*.

The Heap well (1S/4W–3Q1; refer p. 31) has a long period of record and is relatively shallow (*fig. 29*). The water-level record extends from 1942 to 1998 and the well is reported to be 200 ft deep. Most likely, this well senses only ground-water levels in the UCM and UWB hydrogeologic units (*fig. 24*), not in lower confined units. The well also does not appear to be affected by nearby pumpage or recharge. These characteristics make the Heap well a good predictor of underflow from the UCM and UWB units in the Bunker Hill basin across the San Jacinto fault.

In order to estimate underflow for 1945–98, a nonlinear regression equation was fitted to measured ground-water levels in the Heap well and estimated underflow from the California Department of Water Resources (1971, table 4). This equation is

$$Q^{Underflow}_{SAR,\,k} = 96.876 \log(Wl_{Heap,\,k}) + 663.136 \qquad (5)$$

where

$Q^{Underflow}_{SAR,\,k}$ is underflow across the San Jacinto fault near the Santa Ana River during year k, in acre-ft/yr; and

Wl_{Heap} is average ground-water level measured in the Heap well during year k, in ft above sea level.

Equation 5 does a good job ($r^2 = 0.937$) of predicting historical underflow, as simulated by the regional ground-water flow model developed by the California Department of Water Resources (1971). Using equation 5, estimated underflow near the Santa Ana River averaged about 4,000 acre-ft/yr for 1945–98, and ranged from about 12,000 acre-ft in 1945 to about 1,000 acre-ft in the mid 1960's (*table 9*).

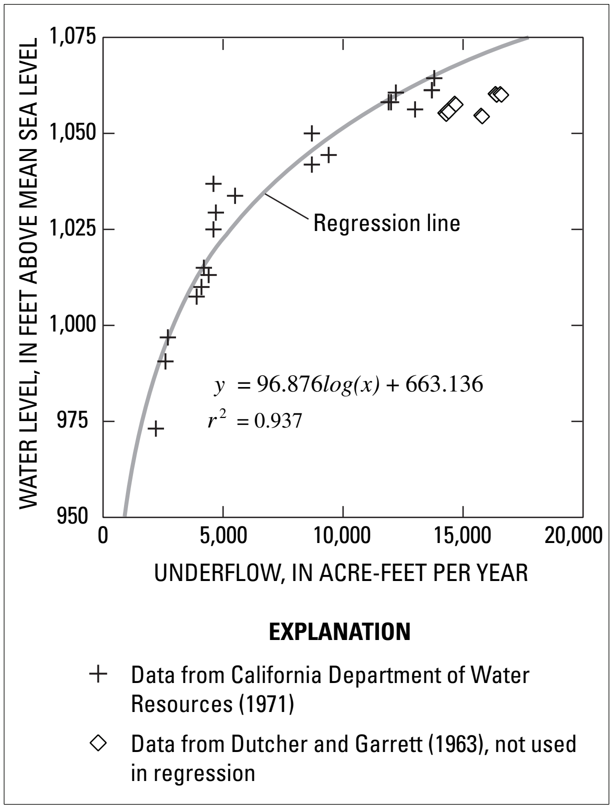


Figure 28. Relation between water level in the Heap well (1S/4W–3Q1) and underflow across the San Jacinto fault near the Santa Ana River.

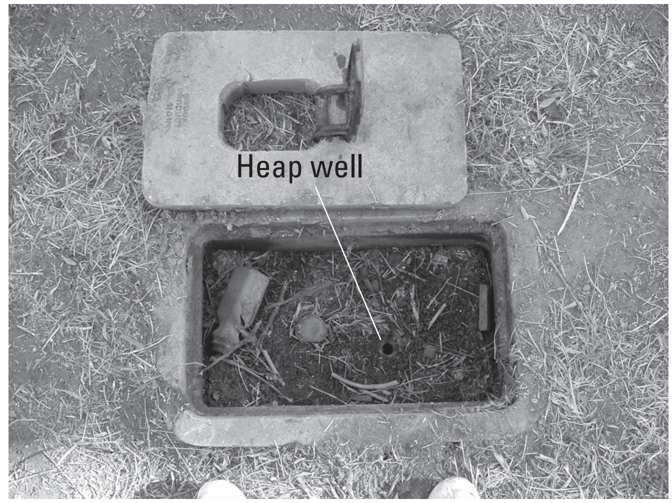
Underflow estimated by Dutcher and Garrett (1963) using Darcy's law is not as well predicted by equation 5 (*fig. 28*). In general, the estimates by Dutcher and Garrett are slightly greater and are tightly grouped, falling within a narrow range of ground-water levels and underflow. It is possible that the California Department of Water Resources (1971) underestimated the true underflow, or that Dutcher and Garrett (1963) overestimated the true underflow. A ground-water flow model will tend to produce smoothly varying values of underflow as shown in *figure 28*, but how well these values match real underflow is rarely, if ever, known. Using Darcy's law to estimate underflow can be equally problematic. Both hydraulic conductivity and cross-sectional area of underflow must be estimated. Uncertainty in these individual estimates can result in estimated underflow varying by a factor of two or more. Despite these caveats, equation 5 produces hydrologically reasonable values of underflow that vary in a predictable way within the general range of underflow values estimated by other investigators.

Underflow near Barrier J and across Barrier E, which is often mapped as part of the San Jacinto fault, was estimated by Dutcher and Garrett (1963, p. 107). Although limited data were available, estimated underflow was about 4,000 acre-ft/yr during 1952. Underflow estimated using a mathematical model ranged from 2,700 to 4,200 acre-ft/yr during water years 1935–60 (California Department of Water Resources, 1971, table 4). Discharge in Lytle Creek and nearby ground-water levels were examined by D.E. Williams (Geoscience Support Services Inc., 1992, 1994), who concluded that underflow to the Rialto-Colton basin averaged about 6,800 acre-ft/yr for 1978–93.

Underflow near Barrier J was investigated by Woolfenden and Kadhim (1997) using ground-water chemistry, though no new estimates of underflow were provided. Subsequent numerical simulation of ground-water flow in the Rialto-Colton basin, however, involved estimating underflow from the Lytle Creek basin to the Rialto-Colton basin near Barrier J (L.R. Woolfenden, U.S. Geological Survey, written commun., 2000). These estimates were calculated from discharge measurements of Lytle Creek where it flows out of the San Gabriel Mountains (*fig. 11*). Estimated underflow was 35 percent of discharge in Lytle Creek for most years, and 15 percent during very wet years. Using these assumptions, underflow near Barrier J averaged about 8,000 acre-ft /yr, and ranged from about 2,000 acre-ft/yr to about 19,000 acre-ft/yr (*table 9*). Final calibration of the ground-water flow model for the Rialto-Colton basin (Woolfenden and Koczot, 2001) slightly altered these initial estimates, but generally substantiated their reasonableness.

An important characteristic of the estimated values of underflow near Barrier J is that they vary greatly from one year to the next (*table 9*). More commonly, underflow varies somewhat smoothly, increasing and decreasing based on fluctuations in nearby ground-water levels (*figs. 27* and *28*). Ground-water levels do fluctuate markedly in the Lytle Creek and Rialto-Colton basins (Geoscience Support Services, Inc. 1992, 1994; Woolfenden and Kadhim, 1997). Whether these fluctuations are sufficient to cause the variations in underflow is not known.

One possible mechanism that could lend crediblity to large annual variations in underflow would be the uplifted bedrock shelf north of Barrier J. If it is sufficiently dissected and extends past Barrier E, then much of what is attributed to be underflow from the Lytle Creek basin, may actually be recharge from Lytle Creek routed via a subsurface drainage directly into the Rialto-Colton basin. Underflow south of Barrier J, estimated to be about 3,000 acre-ft/yr (Geoscience Support Services, Inc., 1994), would be more traditional underflow, fluctuating minimally from one year to the next depending on relative ground-water levels in the Lytle Creek and Rialto-Colton basins. Seismic or ground-penetrating-radar studies might be useful to test this theory.



Photographs by W.R. Danskin, 2005

Figure 29. Heap well (1S/4W–3Q1) in the San Bernardino area, California. Heap well, being measured by USGS Hydrologic Technician Kimball Stumpf and located on the edge of the former marshland, is used to estimate ground water rising into Warm Creek (fig. 21) and underflow from the Bunker Hill basin (fig. 28).

Total estimated underflow into and out of the San Bernardino area for 1945–98 is listed in *table 9*. Total estimated underflow into the San Bernardino area averaged about 5,000 acre-ft/yr for 1945–98, and ranged from about 4,000 acre-ft/yr to about 7,000 acre-ft/yr. Total estimated underflow out of the San Bernardino area averaged about 13,000 acre-ft/yr for 1945–98, and ranged from about 4,000 acre-ft/yr to about 25,000 acre-ft/yr. If a constant value of 3,000 acre-ft/yr as estimated by Geoscience Support Services, Inc. is assumed for outflow across the San Jacinto fault in the vicinity of Barrier J, then total underflow out of the San Bernardino area would have averaged about 7,000 acre-ft/yr for 1945–98, and would have ranged from about 4,000 acre-ft/yr to about 15,000 acre-ft/yr. With this assumption, net underflow for 1945–98 would have averaged about 2,000 acre-ft/yr flowing out of the San Bernardino area.

Ground-Water Discharge into Warm Creek

Ground water discharges into the lower reaches of Warm Creek when nearby ground water rises above the bottom of the stream channel (refer p. 31 in this report). As ground-water levels rise higher than this threshold, discharge increases. For 1945–98, annual values of ground-water discharge into Warm Creek were determined in two ways. Annual values for 1945–59 were interpolated from water-year data calculated by the California Department of Water Resources (1971, table 22) using measured streamflow. Annual values for 1960–98 were estimated using a nonlinear regression equation derived from that measured data (*fig. 21*). As shown in *figure 30*, estimated ground-water discharge varies considerably. Maximum discharge for 1945–98 exceeded 40,000 acre-ft/yr, and minimum discharge was zero for 16 consecutive years, from 1963 to 1978. Average discharge was about 5,000 acre-ft/yr.

Evapotranspiration

Evapotranspiration from the valley-fill aquifer occurs whenever ground water approaches land surface. As depth to the water table decreases, the evapotranspiration rate increases. When the water table is more than about 15 ft below land surface, the evapotranspiration rate is essentially zero. No detailed evapotranspiration studies have been done in the San Bernardino area in order to correlate evapotranspiration rates and depth to ground water. However, a maximum evapotranspiration rate of 1.0×10^{-7} ft/s (38 in/yr) was used by Hardt and Hutchinson (1980) in order to simulate evapotranspiration from the valley-fill aquifer in areas where the water table occasionally coincides with land surface, such as between Warm Creek and the Santa Ana River (*fig. 22*). The maximum rate of 38 in/yr is approximately 50 percent of the evaporation rate observed from a class-A evaporation pan in the San Bernardino area (San Bernardino County Flood Control District, 1975).

Simulated results from Hardt and Hutchinson (1980, figs. 16 and 18, table 3) show that as ground-water levels in the former marshland declined, evaporation declined from a maximum of about 30,000 acre-ft/yr in 1945, to zero from 1954 until the model simulation ended in 1974. Using the same maximum evapotranspiration rate and an estimated depth to ground water, evapotranspiration was estimated to average about 7,000 acre-ft/yr for 1945–98. Evapotranspiration ranged from a maximum of about 26,000 acre-ft in 1983 to about 1,000 acre-ft/yr in the mid 1960's and late 1980's.

Additional evapotranspiration occurs from the unsaturated zone as a result of moisture that does not percolate to the saturated ground-water system. This moisture originates chiefly as direct precipitation on the valley-fill deposits (*fig. 22*) and was estimated to average about 86,000 acre-ft/yr (*table 4*). This additional evapotranspiration is important for the health and growth of vegetation, but it is not discussed further in this report because the unsaturated zone is not considered to be part of the valley-fill aquifer. Similarly, evapotranspiration resulting from native or imported water applied to agricultural fields or to urban areas is not quantified in this report, nor included in any of the reported evapotranspiration values.

Pumpage

Ground-water pumpage in the San Bernardino area is used for agricultural, municipal, and industrial purposes. As the area has become urbanized, the quantity of agricultural pumpage has declined considerably, particularly since about 1957 (California Department of Water Resources, 1971, table 49; Camp Dresser and McKee, 1991, p. 6). By 1992, agriculture accounted for less than 20 percent of total pumpage.

Gross pumpage, defined as total pumped water without accounting for any consumptive use losses or return flow, increased markedly between the mid-1940's and the early 1960's (*fig. 25*). Prior to 1940, gross pumpage in the San Bernardino area typically was less than 110,000 acre-ft/yr (California Department of Water Resources, 1971, table 58). After 1960, gross pumpage commonly exceeded 170,000 acre-ft/yr, and occasionally exceeded 200,000 acre-ft/yr. For 1945–98, gross pumpage averaged about 175,000 acre-ft/yr, and ranged from about 123,000 acre-ft in 1945 to about 215,000 acre-ft in 1996 (*table 10*).

Gross pumpage has remained relatively constant since about 1960 primarily because of a lawsuit filed in 1963. In it, the Western Water District, representing residents generally west and downstream of the San Bernardino area, claimed that their extractions were in jeopardy because of unregulated extraction by others including the city of San Bernardino. The suit, which originally had five plantiffs and more than 4,000 defendants, was settled in 1969 with a stipulated judgement that defined ground-water rights and created a watermaster to monitor ground-water use in the San Bernardino area (State of California, 1969b).

Settlement of this lawsuit, referred to as the Western Judgement, was contemporaneous with settlement of a related lawsuit by the Orange County Water District that focused on similar surface-water issues (State of California, 1969a). Cooperative settlement of both lawsuits involved extensive technical investigations of water availability and use (California Department of Water Resources, 1970, 1971; San Bernardino Valley Municipal Water District, 1970). Much of these historical data, especially ground-water pumpage, was critically important in preparation of this present study.

Typically, more than one-fourth of the gross annual ground-water pumpage is exported out of the San Bernardino area, mostly to the nearby city of Riverside (*fig. 1*). As part of the Western Judgement, the city of Riverside is entitled to pump and export as much as 52,199 acre-ft/yr of ground water from the San Bernardino area. Since 1981, this pumpage-export limit has been increased by about an additional 10,000 acre-ft/yr in an attempt to reduce high ground-water levels in the former marshland (San Bernardino Valley Municipal Water District, 1981b, 1985).

Most pumpage, as shown in *figure 31*, is located near the Santa Ana River, Lytle Creek, and the smaller tributary streams. This areal distribution of pumpage probably results from the exceptionally permeable deposits that underlie the stream channels and from the abundant nearby recharge. As the San Bernardino area has urbanized, some water purveyors have begun installing new wells higher on the alluvial fans, closer to the mountains and closer to the new urban demand (*fig. 8*). This new location permits the extraction of ground water at a higher head in the valley-fill aquifer. As a result, less additional lift is required for distribution, and pumping costs are reduced. The city of San Bernardino, which pumps nearly 20 percent of the ground water extracted in the San Bernardino area, has followed this philosophy in siting new wells since about 1975 (J.F. Stejskal, city of San Bernardino Water Department, oral commun., 1992).

Centroids of gross pumpage were calculated for each decade between 1945 and 1998 to quantify any changes in the areal distribution of pumpage. The centroids, shown in *figure 31*, indicate a small shift to the northwest of about 2 mi as would be expected by the increased pumpage by the city of San Bernardino near Shandin Hills. This shift occurred in about 1970, but probably has not had a significant effect on the overall ground-water flow system. The shift, however, may have had some effect on the migration of ground-water contamination, particularly any contamination flowing near the centroids of pumpage.

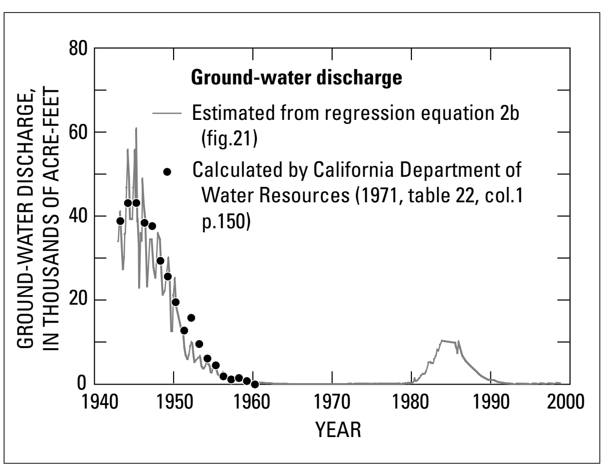


Figure 30. Ground-water discharge into Warm Creek in the San Bernardino area California, 1943–98.

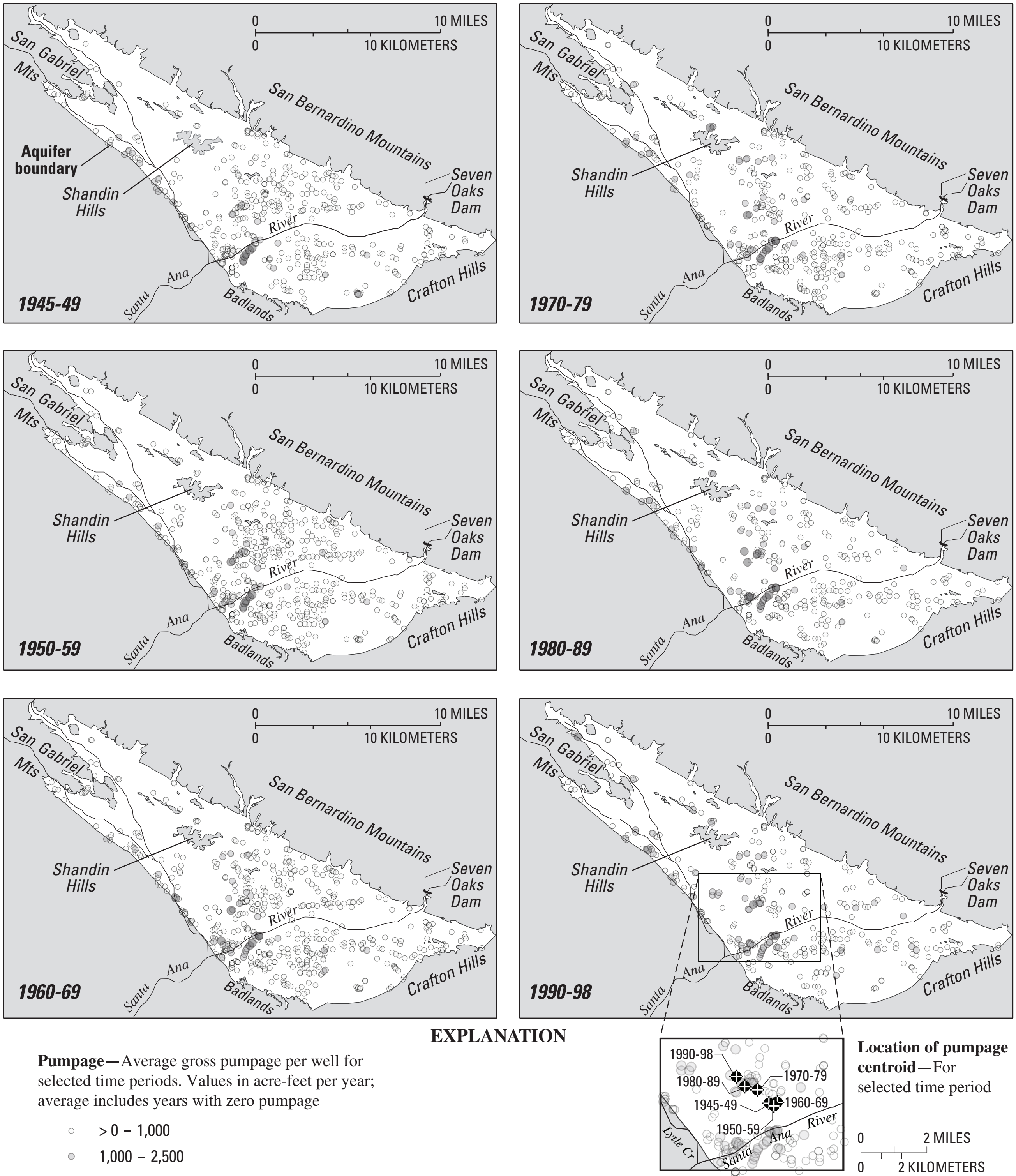


Figure 31. Areal distribution of average pumpage in the San Bernardino area, 1945–98.

The gradual change from many small wells in the San Bernardino area, to fewer large wells also is illustrated in *figure 31*. Part of this change is caused by the installation of new, large-capacity wells, but part of the change is more hours of pumping per year when agricultural wells are converted to municipal use. Note, values of average gross pumpage per well shown in *figure 31* were calculated as if the well were in operation each year. Zero pumpage was assigned for any years that the well did not pump or did not exist.

Compilation and estimation of pumpage values used to prepare *table 10* and figure 31 was a long, detailed procedure involving the Western–San Bernardino watermaster, staff from the San Bernardino Valley Municipal Water District, and cooperation of many other water purveyors in the San Bernardino area. The objective was to create a computerized pumpage database that includes all wells in the San Bernardino area for 1945–98. The database needed to be consistent with data maintained by the watermaster, match watermaster recordation numbers with state well numbers, have all wells located with a global positioning system (GPS), and incorporate selected characteristics necessary for development of the ground-water flow model, such as return-flow percentage.

The end-result of this 3-year effort is a database maintained and annually updated by the Western–San Bernardino watermaster (State of California, 1969b). Compilation of prior watermaster data was done by R.L. Reiter (San Bernardino Valley Muncipal Water District) and S.E. Mains (Western Municipal Water District). Correlation of data maintained by the watermaster using a recordation number, with data maintained by the State of California using a state well number, was done by staff of the San Bernardino Valley Municipal Water District, in particular, S.H Fuller and R.W. Peterson. Prior to this effort, recordation numbers were not necessarily correlated to specific wells. In some cases, a single recordation number referred to different wells in different years. And a few recordation numbers actually referred to surface-water diversions.

Individual wells were identified in the field by a person who was familiar with that specific well, typically someone from the water district or municipality maintaining the well and using the pumped water. During that site visit, latitude and longitude of the well were determined using global positioning equipment. Error in locating each well is believed to less than about 20 ft.

Return-flow percentage for each well was defined using estimates from Hardt and Hutchinson (1980) and knowledge from the Western–San Bernardino watermaster (R.L. Reiter, oral commun., 2000) about the fate of water pumped from individual wells. If water from a well was exported from the San Bernardino area in either a pipe or concrete-lined channel, then a zero return-flow percentage was assigned. For all other wells, a return-flow percentage of 30 was assigned. These percentages were held constant for 1945–98 because wells used for export rarely were used for local supply.

The vertical distribution of gross pumpage between layers of the aquifer (model) was calculated for individual wells using original data sheets developed by W.F. Hardt (U.S. Geological Survey, written commun., 1980). The original estimates from Hardt were based on the perforated interval of a well and the hydraulic conductivity of deposits adjacent to the perforated interval. For new wells, a similar calculation was made to define the ratio of extraction from the upper and lower layer. Since 1980, when extensive ground-water contamination by volatile organics was discovered, many new wells have been installed with perforations only in the lower layer. As shown in *figure 25*, an approximately equal quantity of pumped water is withdrawn from the upper and lower layers of the valley-fill aquifer.

The upper layer is defined as hydrogeologic units UCM and UWB; the lower layer is defined as hydrogeologic units MCM, MWB, LCM, LWB, and QTc (*fig. 24*). These are exactly the same definitions used for the upper and lower layers of the ground-water flow model. Estimating the percentage extraction from upper and lower layers has a significant limitation. For nearly all wells, the hydraulic conductivity of the deposits adjacent to the perforated interval is not known and must be estimated from lithologic descriptions found in driller's logs. Typically, these descriptions do not have detailed information about the compaction, cementation, or sorting of the deposits. Especially for unconsolidated deposits, this technique can over-estimate the amount of extraction from lower deposits which may be described as being very similar to overlying, younger deposits. But in reality the lower deposits are much older, more consolidated, possibly partly cemented, may be deeply weathered, and are likely much less permeable than when they were younger and higher in the stratigraphic column, despite having very similar lithologic descriptions on driller's logs.

Flowmeter testing of three production wells using a spinner tool showed that most of the extraction came from the shallow, younger deposits (Izbicki and others, 1998). In some cases, virtually no water was contributed to the well from the deepest perforated intervals. These measurements suggest that the estimated percentages of extraction from upper and lower layers may be in error, and may overestimate the amount of water extracted from the lower layer. What effects this error has on understanding the ground-water system are difficult to determine, but the effects are likely to be important. For example, the relative quantity of extraction from different hydrogeologic units would change interpretations of where and how fast a contaminate is transported. Flowmetering of as many wells as possible in the San Bernardino area would aid in re-evaluating the estimates of the vertical distribution of extraction used in this report, and likely would aid in better understanding of critical ground-water issues.

Although most pumpage data was provided by the watermaster, some additional processing was necessary to prepare a complete dataset for 1945–98 (*table 10*). To understand this processing, some background information is needed. Each calendar year, anyone who pumps more than 25 acre-ft in the adjudicated area, which includes the entire Bunker Hill and Lytle Creek basins, is required to report this pumpage to the State of California. The watermaster then obtains this *filed* data and *verifies* that it is correct. As an official officer of the court, the watermaster is legally entitled to obtain electrical records, crop use, tax data, and other information needed to verify that the reported pumpage is reasonable. If it is not, then the watermaster adjusts the data for a specific well.

These *verified* data are summarized for plaintiffs and non-plaintiffs in the Western Judgement. Because there are only seven plaintiffs (Western Municipal Water District, city of Riverside, Gage Canal Company, Aqua Mansa Water Company, Meeks and Daley Water Company, Riverside-Highland Water Company, and the Regents of the University of California), most adjustments are for non-plaintiffs. Typically, the total value of pumpage for the seven plaintiffs changes by less than 1 or 2 percent. The total value of pumpage for the hundreds of non-plaintiffs changes by less than about 5 percent, and is usually an increase.

If the watermaster is aware of any pumpage that has not been filed, then the amount is estimated and is added to the dataset. These values are referred to as *non-filed* pumpage, and occur only for non-plaintiffs. At the end of this entire process, the pumpage data for a year is deemed verified and is published.

Verified pumpage data has been available since 1970, the year following the Western Judgement. Verified data also are available for a 5-year period (1959–63) that was used for a safe-yield analysis done as a result of the adjudication (Western–San Bernardino Watermaster, 1972). Records of reported, but non-verified pumpage began in 1947, although non-filer pumpage was not recorded until 1958. No pumpage values per well were available prior to 1947. In order to create a complete pumpage dataset for 1945–98, some pumpage values per well needed to be estimated for selected time periods. These estimates are described below and are referenced in *table 10* as E1 through E5.

Use of non-verified, plaintiff and non-plaintiff pumpage for 1947–58 and 1964–69.—Pumpage data for both plaintiffs and non-plaintiffs for 1947–58 and 1964–69 are available only as non-verified pumpage. These two time periods preceded adjudication of the San Bernardino area, and although pumpage data were reported, it was never verified. Pumpage for the interim period, 1959–63, was verified only because it was analyzed as part of adjudication. Because pumpage for 1947–58 and 1964–69 will never be verified, the non-verified pumpage was deemed adequate for this study. Non-plaintiff non-filer pumpage was estimated separately for the two periods, as described below.

Analysis of verified and non-verified pumpage for both plaintiffs and non-plaintiffs for 1970–97 showed that the non-verified pumpage was generally less than verified pumpage. This difference is less than about 1 percent for plaintiff pumpage and less than about 3 percent for non-plaintiff pumpage. No reliable method was identified to modify non-verified pumpage for 1947–58 and 1964–69 to account for this likely difference; therefore, no adjustment was made.

Estimation of plaintiff pumpage, 1945–46 (E1).—Plaintiff pumpage for 1945–46 was estimated using a set of average values scaled by a linear regression. The set of average values was calculated using plaintiff pumpage for 1947–52, averaged for each well. Zero pumpage was assumed for any well for any year with no pumpage, or if the well may have been destroyed or may not yet have been installed. The linear regression equation,

$$Q^{PumpP}_{k} = -166.38\ P^{SARRO}_{k} + 59{,}244 \qquad (6)$$

where

Q^{PumpP}_{k} is plaintiff pumpage for calendar year k, in acre-ft/yr; and

P^{SARRO}_{k} is average annual runoff for the Santa Ana River measured at USGS station 11051501 for calendar year k, compared to longterm average runoff at the same station for 1928–98, in percent,

was determined from total annual plaintiff pumpage for 1947–52 (*fig. 32*). The plaintiff pumpage for 1945–46 was calculated by multiplying the set of average pumpage values determined for 1947–52 by the total annual pumpage determined from equation 6.

Estimation non-plaintiff pumpage, 1945–46 (E1).— Non-plaintiff pumpage for 1945–46 was estimated in the same way as plaintiff pumpage for 1945–45, using a different, but similarly derived regression equation,

$$Q^{PumpNP}_{k} = -388\ P^{SARRO}_{k} + 117{,}891 \qquad (7)$$

where

Q^{PumpNP}_{k} is total non-plaintiff pumpage for calendar year k, in acre-ft/yr; and

P^{SARRO}_{k} is average annual runoff for the Santa Ana River measured at USGS station 11051501 for calendar year k, compared to longterm average runoff at the same station for 1928–98, in percent.

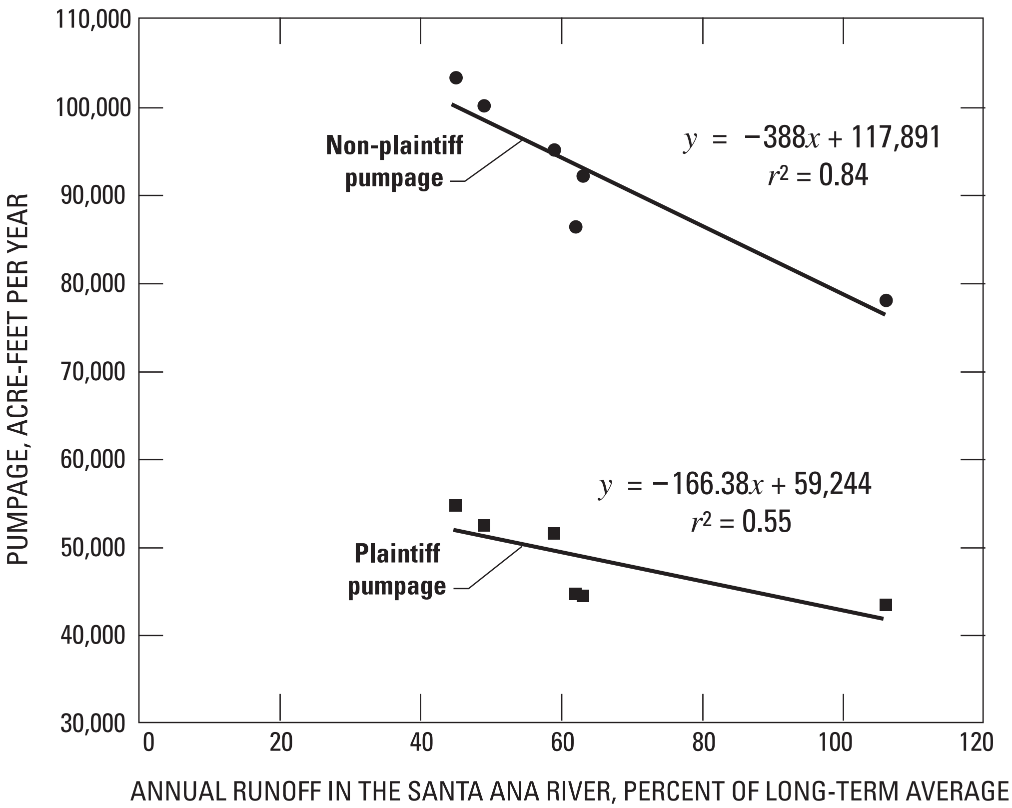


Figure 32. Regression equations used to estimate pumpage in the San Bernardino area, California, 1945–46. All data for 1947–52; refer tables 2 and 10. Runoff data for U.S. Geological Survey gaging station 11051501; long-term average runoff for 1945–98. Plaintiff and non-plaintiff (filed) pumpage data from the Western–San Bernardino Watermaster.

Note, the annual values used to develop equation (7) are slightly different from those in *table 10*. The initial pumpage data included a few wells, whose total pumpage was less than about 500 acre-ft/yr. These wells ultimately were excluded from the analysis because they were not located in the valley-fill, but the regression equation and values for 1945–46 were not redone. The net effect of this decision was deemed to be insignificant.

Estimation of non-plaintiff, non-filer pumpage, 1945–58 (E3).—No data were available for non-plaintiff non-filer pumpage for 1945–58. Therefore, non-filer pumpage for the subsequent safe–yield period (1959–63) was analyzed to detect any changes in the areal distribution of pumpage or any temporal trend. The areal distribution of non-filer pumpage was approximately the same for 1959, 1960 and 1961–63, implying the same wells or nearly the same wells were used during each time frame. No new non-filer pumping centers had been created or destroyed. The average non-filer pumpage for 1959–63 (12,825 acre-ft/yr) was almost exactly the same as for 1959 (12,824 acre-ft/yr). Therefore, it seemed reasonable to use the exact values of non-filer pumpage for 1959 for each year during 1945–58.

A caveat in this assumption is that non-filer pumpage generally decreased from 1959 to 1998. It is possible that non-filer pumpage was greater during 1945–58 than in subsequent periods, but no corroborating data were found. Attempts to correlate non-filer pumpage with annual values of precipitation, runoff in the Santa Ana River, or total reported pumpage were unsuccessful.

Estimation of non-plaintiff, non-filer pumpage, 1964–69 (E4).—No data were available for plaintiff non-filer pumpage for 1964–69. Therefore, annual non-filer pumpage for the prior safe–yield period (1959–63) and for the subsequent verified period (1970–97) were analyzed to detect any changes in the areal distribution of pumpage or any temporal trend. This analysis showed that annual non-filer pumpage before 1964–69 averaged about 12,000 acre-ft, and after 1964–69, decreased from about 8,400 acre-ft to less than 3,500 acre-ft (table 10). Attempts to correlate this decline with annual values of precipitation, runoff in the Santa Ana River, or total reported watermaster pumpage failed to identify a reliable relation. The simplest and seemingly best method of estimating annual non-filer pumpage for 1964–69 was to linearly decrease total non-filer pumpage from the value in 1963 (12,599 acre-ft/yr) to the value in 1970 (8,416 acre-ft/yr).

Analysis of the areal distribution of annual non-filer pumpage showed noticeable differences in the 2 years before 1964–69 (1962–63) and the 2 years after 1964–69 (1970–71). In general, more non-filer wells were present in the former marshland and along Warm and City Creeks. Differences in the areal distribution of non-filer wells between 1962 and 1963 were minor, as were differences between 1970 and 1971.

To determine a representative areal distribution of non-filer wells and pumpage for 1964–69, the 4 proximal years (1962–63, 1970–71) were selected. For this 4-year period, pumpage was summed, then averaged for each non-filer well. Zero pumpage was assumed for any well for any year with no pumpage, even if the well may have been destroyed or may not yet have been installed. Next, these average values of non-filer pumpage were summed for the San Bernardino area, and the total (10,071 acre-ft/yr) was used to calculate annual values of non-filer pumpage for each well for 1964–69. This calculation multiplied the 4-year average pumpage at each non-filer well by NFP^{i}_{Total}/10,071 acre-ft, where NFP^{i}_{Total} is the total non-filer pumpage, in acre-ft, for a specific year i, which was estimated by a linear decrease in total non-filer pumpage from 12,599 acre-ft in 1963 to 8,416 acre-ft in 1970 (*table 10*).

Estimation of non-plaintiff, non-filer pumpage, 1998 (E5).—Plaintiff non-filer pumpage for 1998 was unavailable at the time of preparation of this report; therefore, non-filer pumpage for 1997 was used verbatim. This assumption is unlikely to have a significant effect on total pumpage from the San Bernardino area because recent non-filer pumpage is small. Since 1961, non-filer pumpage had decreased by a factor of three to less than 4,000 acre-ft/yr in 1997 (*table 10*). Any error associated with using 1997 data for 1998 is likely to be minimal, probably less than 500 acre-ft.

Use of non-verified, plaintiff and non-plaintiff pumpage for 1998.—Pumpage data for both plaintiffs and non-plaintiffs for 1998 were available only as non-verified pumpage at the time of this study; therefore, the non-verified data were used. Since 1970, the difference between verified and non-verified pumpage has become progressively smaller. The difference for 1998 is likely to be insignificant, although any update of this study should consider revising *table 10* and the ground-water flow model with verified pumpage for 1998 and subsequent years.

Ground-Water Storage

The first comprehensive change-in-storage calculations for the greater San Bernardino area were done by the California Department of Water Resources (1971, table 61) for water years 1935–60. Two methods were used to calculate change in storage. The first relied on a detailed accounting of recharge and discharge. The second used the technique of multiplying the annual change in the water table by specific yield. The two methods were used together to estimate an average decrease in storage of about 24,000 acre-ft/yr for the 26-year period. The study area for this analysis included the Yucaipa basin; therefore, the decrease in storage for the Bunker Hill and Lytle Creek basins was likely somewhat less.

A few years later, Hardt and Hutchinson (1980, p. 38) calculated the change in storage for calendar years 1945–74. Partly with the aid of a ground-water model of the Bunker Hill and Lytle Creek basins, these investigators estimated an average decrease in storage of about 33,000 acre-ft/yr for the 30-year period. This rate equates to a total storage depletion of almost 1 million acre-ft. For the same period, but only for the Bunker Hill basin, the San Bernardino Valley Municipal Water District (1977) estimated a total storage depletion of about 700,000 acre-ft, or about 23,000 acre-ft/yr. This analysis used the technique of multiplying the annual change in the water table by specific yield. Computational limitations restricted the calculation to nine subareas of the Bunker Hill basin; each subarea was assumed to have a single value of specific yield and change in water table.

More recently, annual change in storage for 1934–98 was calculated by the San Bernardino Valley Municipal Water District (2000) for an area slightly larger than the Bunker Hill and Lytle Creek basins. The technique used a GIS with annual measurements of ground-water levels and areally distributed estimates of specific yield. Ground-water-level measurements were chosen to represent the lowest levels observed during fall of each year and were selected from wells that were believed to represent the unconfined part of the valley-fill aquifer. Contours of specific yield were digitized from Eckis (1934, map E). A GIS gridding program was used to interpolate ground-water levels and specific yield for thousands of small grid cells. Interpolation was done independently for nine subareas bounded by faults in order to avoid interpolation errors associated with abrupt changes in ground-water levels or specific yield across a fault.

Annual change in storage for 1934–98 was calculated for each grid cell as the annual change in ground-water level multiplied by specific yield. Values for individual cells were summed specifically for the area of the valley-fill aquifer (*fig. 22*), which is a subset of the area analyzed by the San Bernardino Valley Municipal Water District (2000).

Based on these data, significant changes in ground-water storage occurred during 1945–98. Although the total quantity of ground water in storage is not known, the maximum cumulative change in storage was greater than 900,000 acre-ft from 1945 to 1966 and greater than 500,000 acre-ft for several 10-year periods. The annual change in storage commonly ranged from 50,000 to 100,000 acre-ft. The average change in storage for 1945–98 was about 4,000 acre-ft/yr. This decrease is significantly less than for other estimates described above, but may result from greater recharge from local runoff during recent years, as well as recharge from imported water.

Potential errors in this calculation of storage are likely to be minor, but may include: (1) inaccurately estimating specific yield, (2) selecting of a well that includes a confined response rather than only the desired water-table response, and (3) not calculating change in storage for the confined part of the aquifer. Errors associated with item 1 were mitigated somewhat by using the careful mapping of specific yield by Eckis (1934, map E). Errors associated with item 2 were avoided as much as possible by reviewing construction data and hydrographs for all wells used for ground-water levels. The review helped ensure that the wells sensed only the water table. Commingling of a water-table and confined response would overestimate changes in storage. Because of large ground-water-level changes from 1934 to 1998, the goal of sensing only the water table may not have been achieved perfectly.

Errors associated with item 3 can be addressed by estimating the additional change in storage that occurred in the confined part of the valley-fill aquifer. The difference between specific yield and confined storage coefficient in the San Bernardino area is about a factor of 1,000—an average of about 0.10 for specific yield compared to about 0.0001 for confined storage coefficient. Change in water-level for a confined system is probably less than 10 times the change in water-level for an unconfined system. These assumptions mean that the approximate change in storage for the confined part of the San Bernardino valley-fill aquifer is about 100 times less than for the unconfined system, or about 40 acre-ft/yr.

Ground-Water Budget

A ground-water budget for the valley-fill aquifer (*fig. 22*) for 1945–98 is listed in *table 11*, and a piechart of these values is shown in *figure 33*. The budget is derived mostly from measured or estimated values of the various components of recharge and discharge. Detailed descriptions are included in two previous sections of this report, "Recharge and natural discharge" and "Pumpage." Evapotranspiration values for the budget were derived with the aid of the ground-water flow model, described in a later section of this report, because no annual valleywide estimates of evapotranspiration have been made.

Minimum and maximum annual values for 1945–98 also are listed in *table 11*. These values, many of which are much different from the average value, give an indication of the possible uncertainty in the average values, and demonstrate the wide range in recharge and discharge that can be expected to occur absent a major change in either climatic conditions or human water-management decisions.

The relatively large annual fluctuations in ground-water storage—indicated by a 400,000 acre-ft/yr difference between minimum and maximum values—mirror the large fluctuations in runoff from the surrounding mountains. Recharge from gaged streams ranges from less than 30,000 acre-ft/yr to more than 400,000 acre-ft/yr (*table 11*). By comparison, gross pumpage is relatively static, varying less than about 50 percent of the average value, or about 50,000 acre-ft/yr. The maximum variation in recharge of imported water is only 30,000 acre-ft/yr. Values in *table 11* illustrate that although some water has been imported for artificial recharge and pumpage varies somewhat, replenishment of the valley-fill aquifer depends mostly on recharge from local sources.

A residual term, which reflects the cumulative error in estimating recharge, discharge, and change in storage, is included explicitly in the ground-water budget. This approach is rare in water-budget analyses, but was chosen for this study in order to maintain the veracity of each of the components of the water budget. Many of the components were calculated from a rare abundance of measured data, or were estimated with great care. How the residual term should be distributed amongst the various components is not known, but an estimate is provided as part of *table 11*.

The largest component of the residual term is underflow across the San Jacinto fault near Barrier J. Underflow in this area was the subject of lengthy investigation and debate during this study, but no clear understanding emerged. Increased recharge from gaged streams and a greater change in storage for the unconfined part of the valley-fill aquifer also are relatively large components of the residual term. The reasonableness of values chosen for these three components of the residual resulted from review of a simulated water budget for the ground-water flow model. Seepage from the bedrock aquifer surrounding and underlying the valley-fill aquifer commonly is assumed to be zero, as it was in the conceptual model used for this study. But a heat-transport model suggested that as much as 15,000 acre-ft/yr of water could be contributed to the valley-fill aquifer from the bedrock aquifer (Hughes, 1992). A value of 6,000 acre-ft/yr was used to parse the residual and recognize that the seepage is certainly greater than zero, though how much greater is unknown.

Table 11. Ground-water budget for the San Bernardino area, California, 1945–98.

[Values in acre-feet per year; –, indicates a decrease in ground-water storage; na, not applicable; average values are well researched from measured and estimated data; values to compensate for calculated residual are speculative]

Component	Minimum	Average value	Maximum	Comment
Recharge				
Direct precipitation	0	1,000	12,000	
Gaged runoff	27,000	116,000	423,000	
Ungaged runoff	4,000	16,000	68,000	
Local runoff	2,000	5,000	12,000	
Imported water	0	3,000	30,000	
Underflow	4,000	5,000	7,000	
Return flow from pumpage	20,000	28,000	37,000	
Total	57,000	174,000	589,000	
Discharge				
Pumpage	123,000	175,000	215,000	
Underflow	4,000	13,000	25,000	
Evapotranspiration	1,000	7,000	26,000	
Rising ground water	0	5,000	42,000	
Total	128,000	200,000	308,000	
Change in storage	–143,000	–4,000	289,000	
Residual	na	–22,000	na	
Sources of water to compensate for residual				
Recharge from gaged runoff	0	4,000	5,500	Simulated values are 5,500 acre-feet per year greater than original estimate, which required many assumptions.
Recharge from ungaged runoff	0	500	500	Original estimate is highly uncertain.
Recharge from local runoff	0	500	500	Roundoff error of original estimate is 500 acre-feet per year.
Seepage from bedrock aquifer	0	6,000	15,000	Some underflow from bedrock is likely, and has been estimated using a heat-transport model to be as much as 15,000 acre-feet per year.
Change in storage, unconfined part of the valley-fill aquifer	0	3,000	7,500	Ground-water flow model suggests a greater change in storage occurred.
Change in storage, confined part of the valley-fill aquifer	0	100	500	Original estimate for change in storage did not account for the confined aquifer.
Water released during land subsidence	0	500	1,000	Some inelastic release of water from storage likely occurred, but the quantity is unknown.
Reduced evapotranspiration	0	1,000	2,000	Model may overestimate evapotranspiration.
Reduced underflow out of aquifer	0	6,400	6,400	Simulated value for underflow near Barrier J is 6,400 acre-ft/yr less than original estimate.
Total	na	22,000	na	

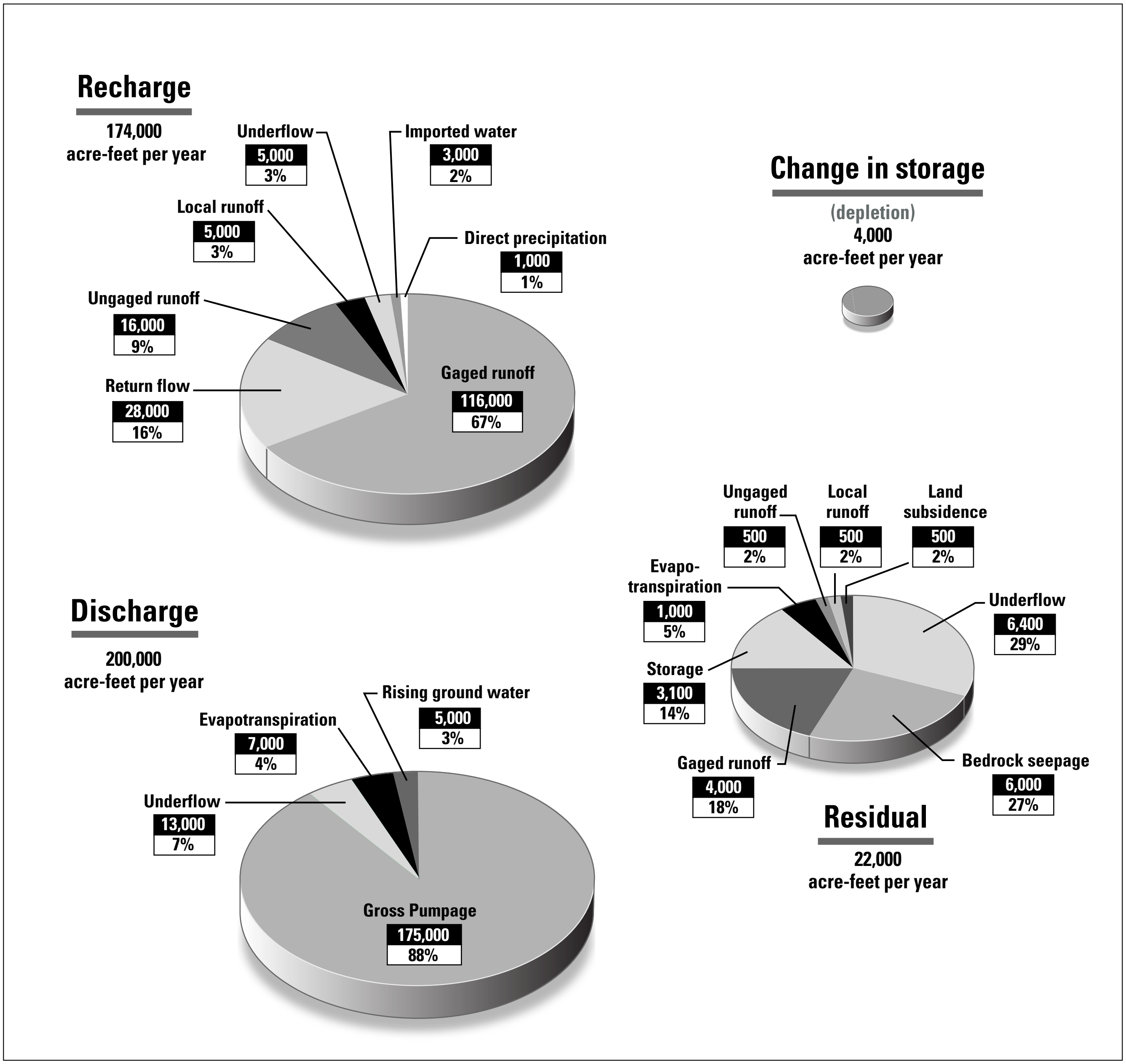


Figure 33. Average recharge, discharge, and change in storage for the valley-fill aquifer in the San Bernardino area, California, 1945–98. Values in acre-feet per year; individual components also shown as approximate percent of total recharge or discharge. Annual values listed in table 8. Residual represents difference between recharge, discharge, and change in storage; components of the residual are rough estimates.

Ground-Water Movement

In all aquifers, ground water flows from areas of recharge to areas of discharge. In the San Bernardino area, the overall pattern of ground-water flow is controlled primarily by the relatively large areas of recharge, and to a lesser degree, by the more localized areas of discharge and by the location of faults that impede ground-water movement. The areal pattern of ground-water movement—from areas of recharge along the base of the San Bernardino Mountains, south toward areas of discharge where the Santa Ana River crosses the San Jacinto fault—has remained similar from historical times prior to ground-water development (fig. 9; Mendenhall, 1905, pl. 8) to the present (fig. 22; Duell and Schroeder, 1989, fig. 5).

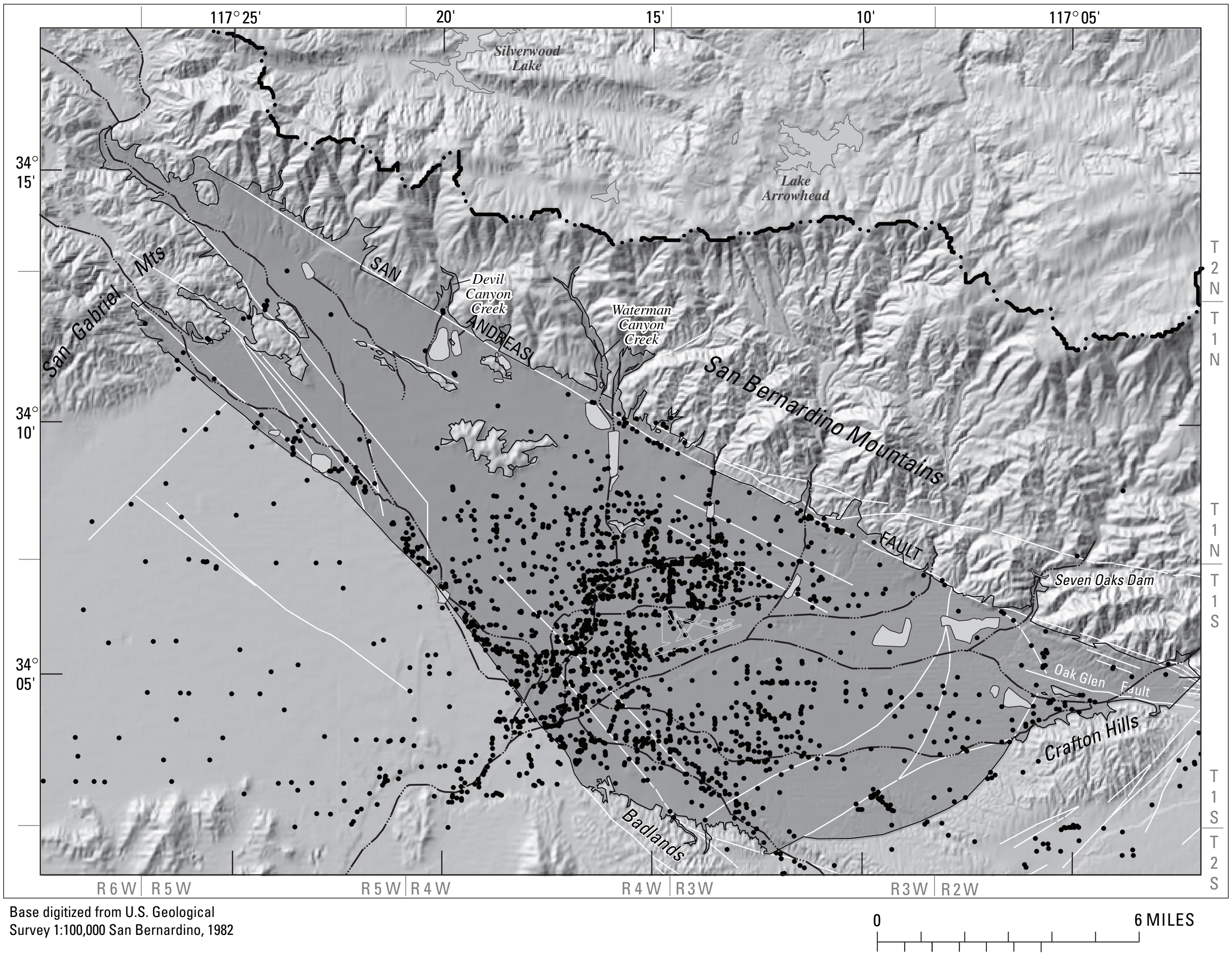
The vertical pattern of ground-water flow, however, has been changed significantly by ground-water development. Historically, ground water moved vertically down through the aquifer materials in recharge areas, then horizontally through the more permeable layers of the valley-fill aquifer, and eventually vertically up through fine-grained materials to be discharged as underflow across the San Jacinto fault, as evapotranspiration from the marshland, and as rising water into Warm Creek. A downward vertical gradient was present in the recharge areas, and an upward vertical gradient was present in the discharge area. This pattern of flow and vertical distribution of ground-water head is typical of an undeveloped ground-water basin (Freeze and Cherry, 1979, p. 196). Withdrawal of water from the earliest hand-dug wells, which were less than 100 ft deep, did not significantly alter ground-water levels or the pattern of ground-water flow.

As ground-water production increased, first for agricultural and then for urban uses, ground water was withdrawn from increasingly deeper parts of the valley-fill aquifer. Natural discharge to the land surface was replaced by discharge to pumping wells within the aquifer. Hydraulic head within the aquifer changed to reflect the change in ground-water flow patterns, and the upward vertical gradient was reduced or reversed. The extent of this change is shown in figure 24 for a representative section of the valley-fill aquifer. By 1992, the entire extent of historically flowing wells from the San Jacinto fault to Highland Road showed a downward, not upward, vertical gradient.

The abundance of wells, in particular abandoned wells, in the San Bernardino area also has affected vertical ground-water flow. Between 1939 and 1945, Horace Hinckley inventoried and mapped more than 2,500 wells in the San Bernardino area; over 800 of these wells were identified as being destroyed (*fig. 34*). Most likely, some of the wells identified by Hinckley as destroyed, in addition to some of the wells abandoned in the succeeding 55 years, were perforated, or have been corroded opposite permeable layers of the valley-fill aquifer. Many of these wells would have been filled in, at least partly, with dirt or debris, but some lengths of open hole or casing probably remain open. In addition, gravel packing in the annulus of a well may span less permeable zones of the aquifer. Together the well casing and adjacent gravel pack can act as an exceptionally permeable vertical conduit, shunting ground water either up or down through the valley-fill aquifer, at many times the rate that is possible through the native aquifer materials. The same effect occurs in non-pumping production wells. Video logging of such wells has shown ground water flowing vertically at several feet per minute even during unpumped conditions (J.F. Stejskal, City of San Bernardino, written commun., 1992). The rate and direction of vertical flow through aquifer materials or abandoned wells is governed by vertical gradients in the aquifer.

Ground-water-level data from a multiple-depth monitoring site (1S/4W–22D2, 4–7; *fig. 24*) show the complexity of vertical gradients and how the gradients between hydrogeologic units change throughout the year (*fig. 35*). At this monitoring site, ground water during summer flows up and down to a major pumping zone at 160–200 ft below land surface. During winter, when pumpage is reduced and recharge to the valley-fill aquifer increases, ground water flows from this zone to underlying zones. During the entire year, ground water flows from the uppermost zone (10–45 ft) to the 160–200 ft zone. This complexity of vertical ground-water movement probably is typical of conditions in the valley-fill aquifer at distances of as much as 1 mile from significant ground-water pumping (*fig. 31*).

In 1990, a large-scale aquifer test was conducted using a newly installed, high-capacity production well (Ninth Street well, 1S/4W–4E8) and four multiple-depth monitoring wells (*figs. 24* and *35*). Additional monitoring wells with a single screened interval also were used. The Ninth Street well was pumped continuously at about 7 ft^3/s for about 7 days. Response in ground-water levels occurred much further away and in a much more predictable way than was expected. At the Meadowbrook site (1S/4W–10B3), more than 8,000 ft from the pumping well, the ground-water-level decline was about 4 ft in the middle confining member (MCM; *fig. 24*) after 7 days of pumping. Even a very small response (0.05 ft) was observed after 1 day more than 15,000 ft away in the water-table piezometer (1S/4W–22D7) at the SBVMWD site. Subsequent aquifer-test analyses (Theis, 1935) confirmed the continuity of the hydrogeologic units and their hydraulic properties in the center of the Bunker Hill basin. Approximate values of transmissivity and storage coefficient from this test were 17,000 ft^2/d and 0.001, respectively.



EXPLANATION

Basin boundary—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray

Fault or ground-water barrier—May be concealed or approximately located

Artificial-recharge basin

Boundary of Santa Ana River drainage basin

• **Water well**—One of about 2,000 wells visited and mapped between 1939 and 1945 by H.P. Hinckley (San Bernardino Valley Water Conservation District, written commun., 1945). About 800 of these wells were identified as having been destroyed by 1945. Another 434 wells, listed in U.S. Geological Survey Bulletin 142, were noted by Hinckley as having been destroyed, but are not included on either his map or this illustration

Figure 34. Location of wells in the San Bernardino area, California, 1945.

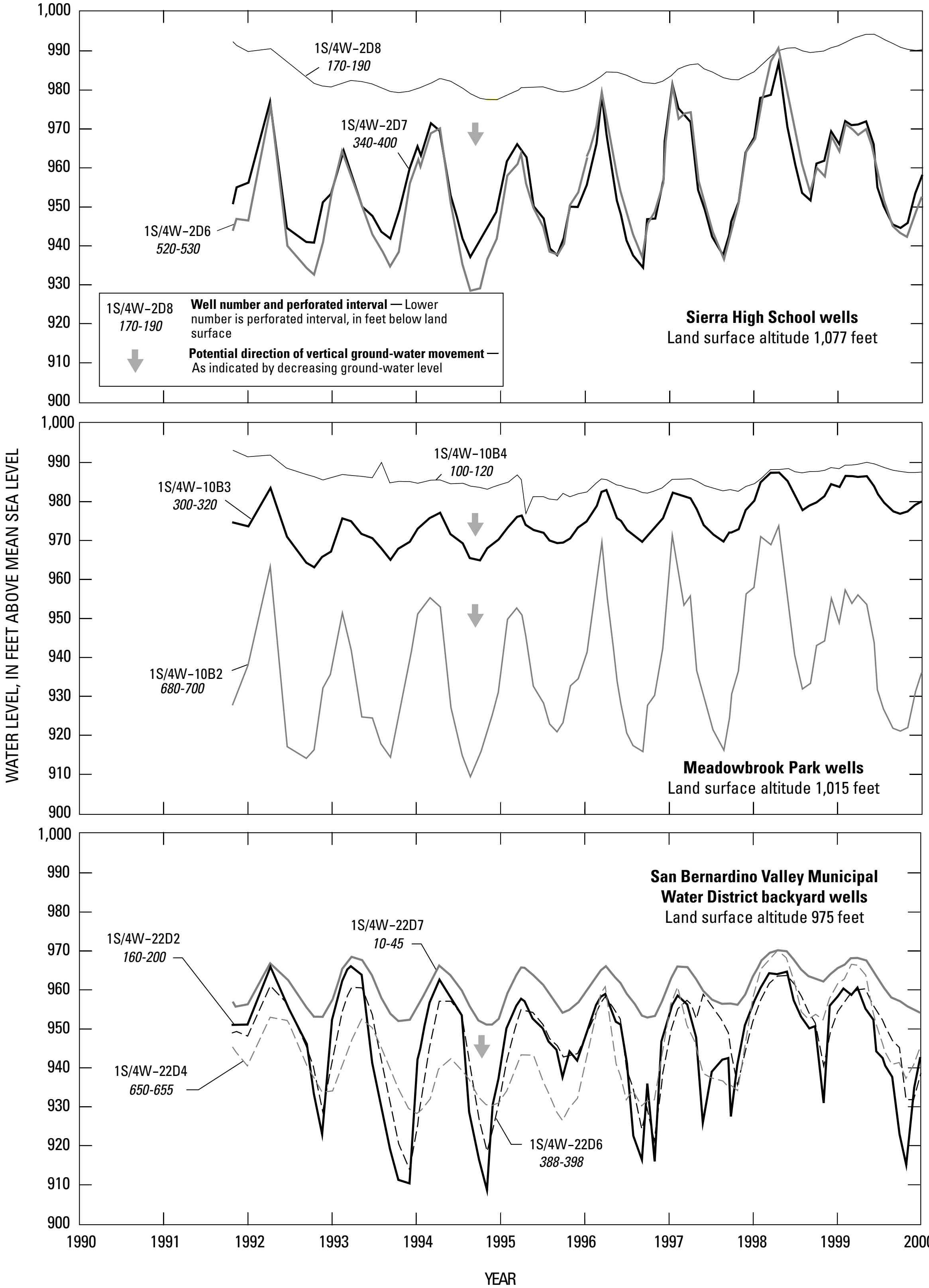


Figure 35. Measured water level in multiple-depth cluster wells in the San Bernardino area, California, 1990–2000.

As of 1998, the longterm trend in ground-water development continues; ground water is preferentially withdrawn from hydrogeologic units deeper than UWB (*figs. 24–25*). Many large municipal production wells have perforations only below a depth of 200 to 300 ft below land surface. This change in construction, done largely to avoid ground-water-quality problems near the land surface, has further altered the vertical movement of ground water. With additional deeper extractions, the hydraulic head in the deeper hydrogeologic units (MWB, LCM) will decline. If this decline is significant, compared to historical declines, then land subsidence, which occurred from 1950 to 1970, may resume (Miller and Singer, 1971; California Department of Water Resources, 1986). In addition, a decline of hydraulic head in the deeper hydrogeologic units will induce some ground-water flow to the pumped zones from the poorly permeable LWB unit, through faults and fractures, and possibly from the surrounding and underlying bedrock aquifer.

Ground-Water Quality

Ground water in the San Bernardino area generally is a sodium-calcium-bicarbonate type, containing equal amounts (on an equivalents basis) of sodium and calcium in shallow ground water and an increasing predominance of sodium in water from deeper parts of the valley-fill aquifer. Concentrations of both sodium and chloride are higher in the lower confining member (LCM) and lower water-bearing unit (LWB) (*fig. 24*). Mean dissolved-solids concentration was about 400 milligrams per liter (mg/L) in the upper part the valley-fill aquifer and about 200 mg/L in the deeper parts of the valley-fill aquifer where confined conditions are present (*fig. 24*) (Duell and Schroeder, 1989, p. 56).

The inorganic composition of ground water varies areally in the valley-fill aquifer depending on the part of the watershed contributing runoff and recharge (Dutcher and Garrett, 1963). Runoff from igneous and metamorphic rocks tends to have a lower dissolved-solids concentration than runoff from sedimentary rocks or unconsolidated deposits (*fig. 5*). The largest sources of runoff and recharge—the Santa Ana River, Lytle Creek, and Mill Creek (*figs. 11* and *12*)—have calcium-bicarbonate water. The smaller creeks in the middle of the San Bernardino area—East-Twin, City, and Plunge Creeks— have higher equivalent concentrations of sodium (Dutcher and Garrett, 1963, fig. 2).

The inorganic composition of ground water also may be affected by small quantities of geothermal water emanating from faults and fractures in the bedrock surface underlying the valley-fill aquifer (Eccles and Klein, 1978). Geothermal water typically has high concentrations of metal ions (Hem, 1985, p. 31) and an elevated temperature. Ground water suggesting a geothermal origin has been found near Faults L and K (Eccles and Klein, 1978), near the westernmost extension of the Banning (?) fault (Geoscience Support Services, Inc., 1990), and near the Loma Linda fault just west of the Santa Ana River (Young and others, 1981). This last area, which is within about 2,000 ft of the Loma Linda fault, has been identified as a localized geothermal zone, and as of 1996 is being used to provide geothermal energy for the city of San Bernardino (Cuniff and Gross, 1984).

The inorganic quality of most native ground water in the San Bernardino area is suitable for both agricultural and municipal uses (Duell and Schroeder, 1989). Concentrations of fluoride that exceed the public drinking-water standard, however, have limited the use of ground water extracted near some faults in the area and from some deeper parts of the valley-fill aquifer. Ground water with an unacceptably high concentration of fluoride or an elevated temperature generally can be blended with other ground water in order to achieve an acceptable quality for either agricultural or municipal use.

Agricultural and urban development have caused additional water-quality problems, primarily contamination of the native ground water by nitrogen species, pesticides, and volatile (purgeable) organic compounds. Since about 1980, detection of these ground-water contaminants has become widespread (fig. 36), and more than 40 public-supply wells have been closed. As more wells are contaminated and as the legally acceptable concentrations of contaminants are lowered (California Department of Water Resources, 1995a), local water-supply agencies have become increasingly concerned about the source and movement of ground-water contaminants. To ensure continued use of the valley-fill aquifer, the agencies have funded additional data collection, enlisted the aid of the state and federal governments to provide ground-water treatment facilities, and sought improved methods for managing ground water.

Nitrate concentrations, measured as nitrogen (NO_3–N), have equaled or exceeded the public drinking-water standard of 10 mg/L in some parts of the valley-fill aquifer for much of the past 20 years (Duell and Schroeder, 1989, fig. 12). Closure of public-supply wells prompted several detailed field investigations (Eccles and Bradford, 1977; Eccles and Klein, 1978; Eccles, 1979; Klein and Bradford, 1979, 1980; Peter Martin, U.S. Geological Survey, written commun., 1980). Early investigations attempted to correlate the presence of nitrate to land use and the observed increase in nitrate concentrations to rising ground-water levels. Rising ground-water levels were believed to remobilize nitrogen species in the unsaturated zone. A followup investigation by Duell and Schroeder (1989, p. 1) failed to discern any consistent relation between land use and nitrate concentration and did not detect any areal trend in nitrate concentrations since 1955. As found by previous investigators (Eccles and Bradford, 1977, p. 25), nitrate concentration was observed to generally decrease with increasing depth below land surface (Duell and Schroeder, 1989, fig. 11). For example, in the central part of the Bunker Hill basin, mean concentrations of nitrate as nitrogen in the upper and lower layers of the valley-fill aquifer were 14.0 and 3.2 mg/L, respectively (Duell and Schroeder, 1989, table 5).

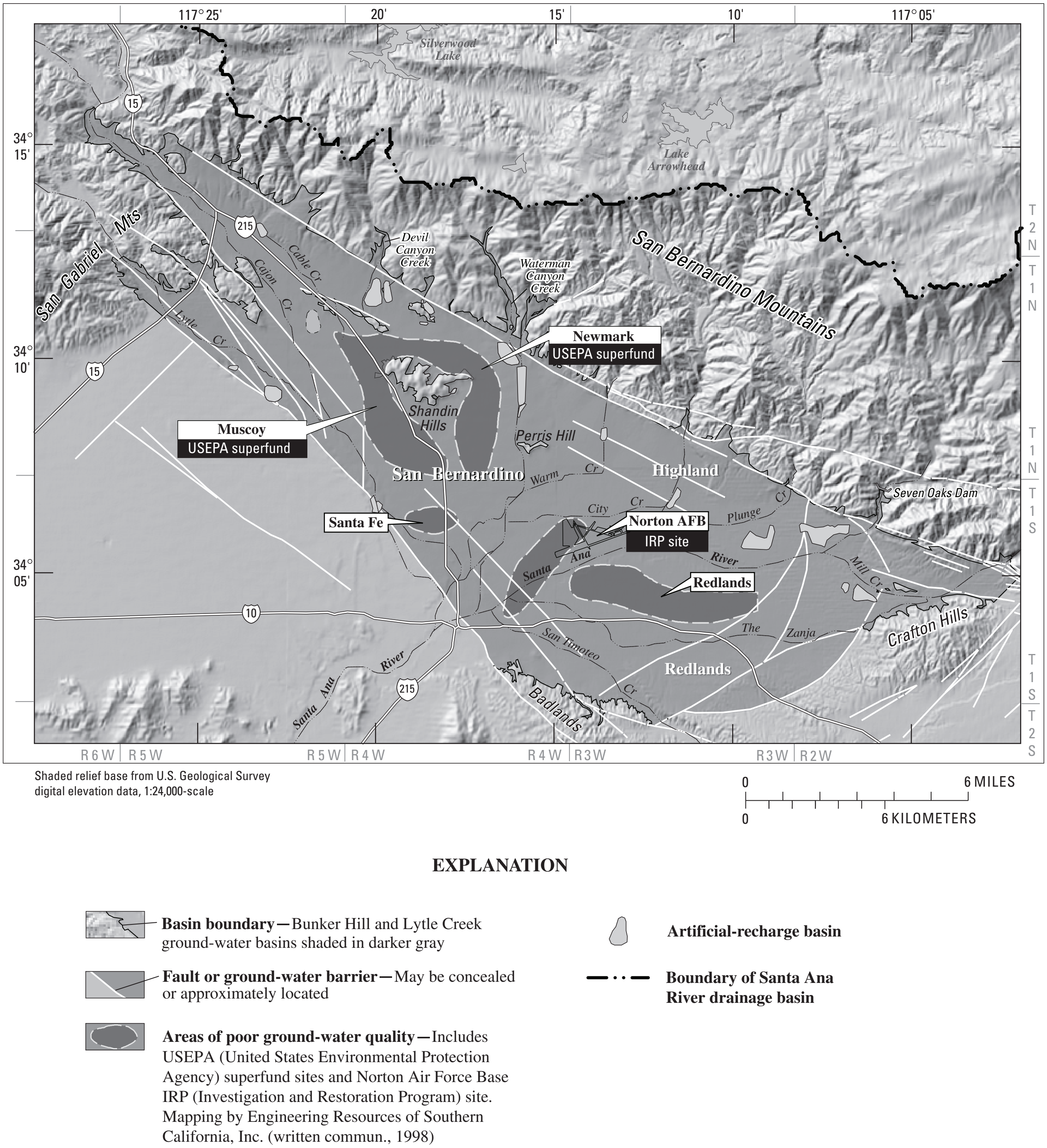


Figure 36. Areas with poor ground-water quality in the San Bernardino area, California, 1997.

The most prevalent pesticide that contaminates ground water in the San Bernardino area is dibromochloropropane (DBCP), a soil fumigant no longer used in the area. As with nitrate contamination, little is known about the occurrence and transport of DBCP in the valley-fill aquifer. The extensive agricultural lands, particularly near Redlands, were used for growing citrus crops (Duell and Schroeder, 1989, p. 17). Application of DBCP was a routine part of citrus production for more than 30 years. As land was converted from agricultural to urban use, wells were converted from agricultural to municipal supply, and additional wells were drilled. Testing of ground water from many of the wells converted to municipal supply revealed the presence of contaminants, such as nitrate and DBCP. These contaminants may have been present previously, but would not have been considered a problem in ground water used solely for agricultural purposes.

Contamination of ground water in the San Bernardino area by volatile (purgeable) organic priority pollutants was first discovered in 1980 (Duell and Schroeder, 1989, p. 6). The most commonly found organic contaminants in the area are trichloroethylene (TCE) and tetrachloroethylene (PCE). As a result of this contamination, pumping from 14 municipal water-supply wells was discontinued as early as 1981. Since that time several additional wells have been closed and many more are threatened with closure (*fig. 36*). Continued ground-water-quality sampling has been done by local water purveyors, the California Regional Water Quality Control Board, the California Department of Health Services, the U.S. Environmental Protection Agency, Ecology and Environment, Inc. (1987), URS Corporation, and the U.S. Geological Survey.

As a result of the extensive data collection, two areas of ground-water contamination have been identified and designated by the U.S. Environmental Protection Agency as operable units (Newmark and Muscoy on *fig. 36*) of a federal superfund site. The source or sources of contamination for the Newmark and Muscoy operable units are not known, and the extent of contamination for the Muscoy operable unit has been identified only schematically (URS Corporation, 1994). A third area of ground-water contamination by volatile organic compounds is referred to locally as the Redlands plume (fig. 36). In 1994, the California Regional Water Quality Control Board (1994) required Lockheed Corporation to begin investigations to quantify the areal extent of ground-water contamination and to design a remediation plan. A fourth area of significant contamination by volatile organic compounds is on and adjacent to Norton Air Force Base (*fig. 36*). Contamination in this area involves radionucleides and metals in addition to volatile organic contaminants. Investigations of ground-water contamination on and near Norton Air Force Base were begun in 1984 as part of the federal base closure program. The cleanup part of the base closure is referred to as the Investigation Remediation Program (IRP). As of 1996, ground-water monitoring was continuing and contamination by TCE and PCE was being cleaned up using a combination of extraction and injection wells (CDM Federal Programs Corporation, 1997).

Water that is imported into the area is generally of good quality, but has a higher concentration of dissolved solids than the native ground water. The export of native ground water with a low concentration of dissolved solids and the use and reuse of imported water with a higher concentration of dissolved solids has prompted concern by local water purveyors that dissolved solids (salts) in ground water will increase (San Gorgonio Pass Water Agency, 1993). Desalting facilities have been installed in nearby ground-water basins in an attempt to reduce the increasing concentration of dissolved solids (Santa Ana Watershed Project Authority, 1980), and construction of a similar facility for the San Bernardino area has been a topic of ongoing discussions by local water purveyors.

Since about 1980, the increased concern about various types of ground-water contamination and ground-water degradation has prompted several changes in ground-water use and management. Some municipal wells are no longer used or no longer used as much because of recurring contamination problems. Commonly, water from some municipal wells must be blended to achieve an acceptable quality. New or refurbished wells often are designed to extract ground water preferentially from the lower hydrogeologic units in the valley-fill aquifer. Ground water extracted from the upper water-bearing zone (UWB, *fig. 24*) typically has a higher concentration of dissolved solids than the middle water-bearing zone (MWB), particularly in the vicinity of the former marshland (*fig. 2*). Concentrations of nitrate, pesticides, and volatile organic compounds also tend to be higher in the upper water-bearing zone, sometimes in excess of safe drinking-water standards (Duell and Schroeder, 1989).

Some multiple-depth well sites, such as those shown in *figures 24* and *33*, have been installed near the Newmark, Muscoy, and Norton Air Force Base areas of contamination. Ground-water-level measurements and ground-water-quality samples from these monitoring sites are being used to identify the source and movement of ground-water contaminants. Installation and monitoring of additional multiple-depth well sites throughout the San Bernardino area would facilitate extending the three-dimensional knowledge gained at the existing sites to the rest of the valley-fill aquifer.

Computer Models

Ground-Water Flow Model

A ground-water flow model of the valley-fill aquifer was developed to provide quantitative information to aid in managing water resources in the San Bernardino area. This information can be used independently or can be combined with the constrained optimization model, described later in this report, to address more comprehensive questions of water use.

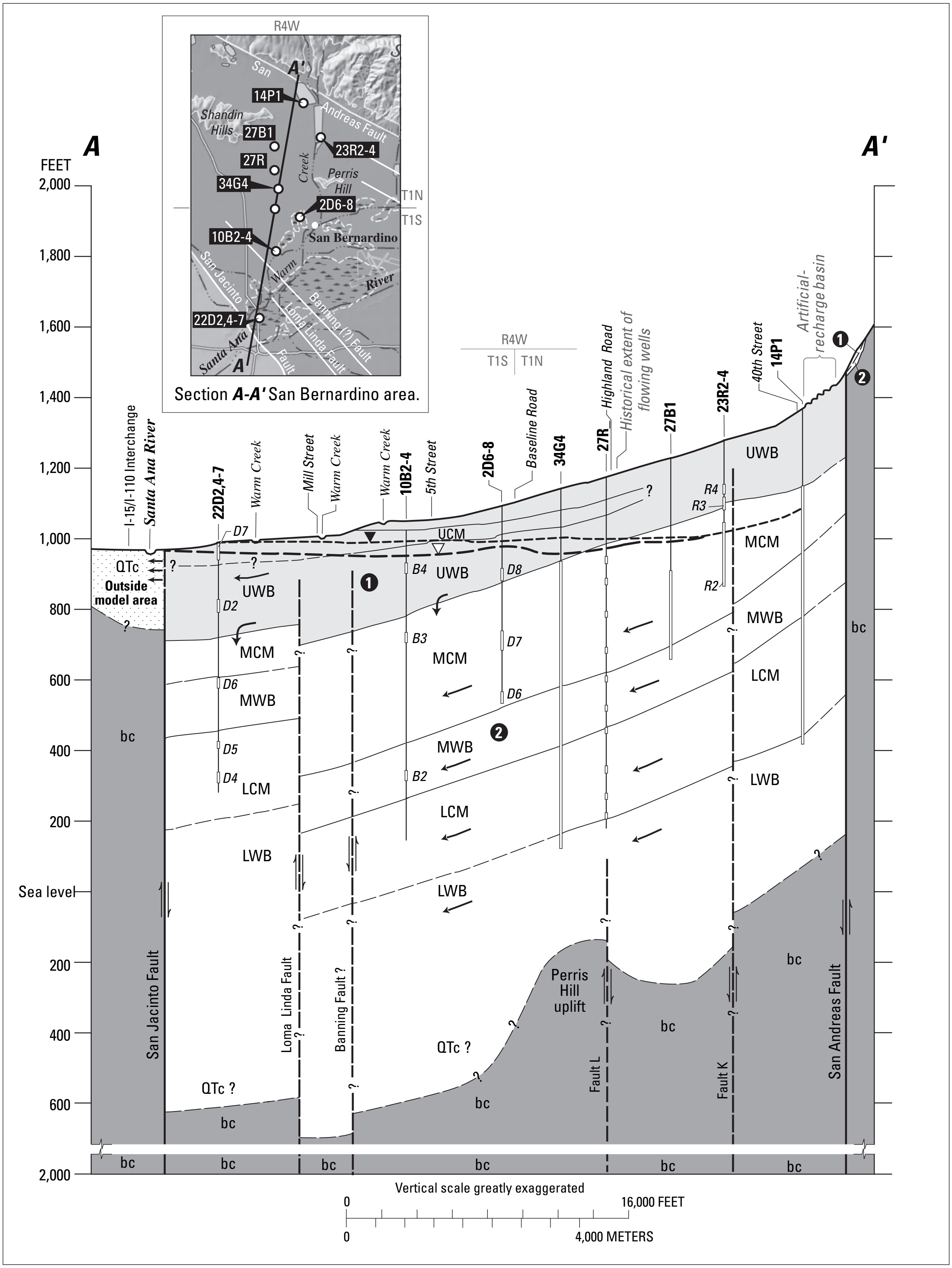


Figure 37. Section *A–A′* showing relation between hydrogeologic units and layers of the ground-water flow model in the San Bernardino area, California.

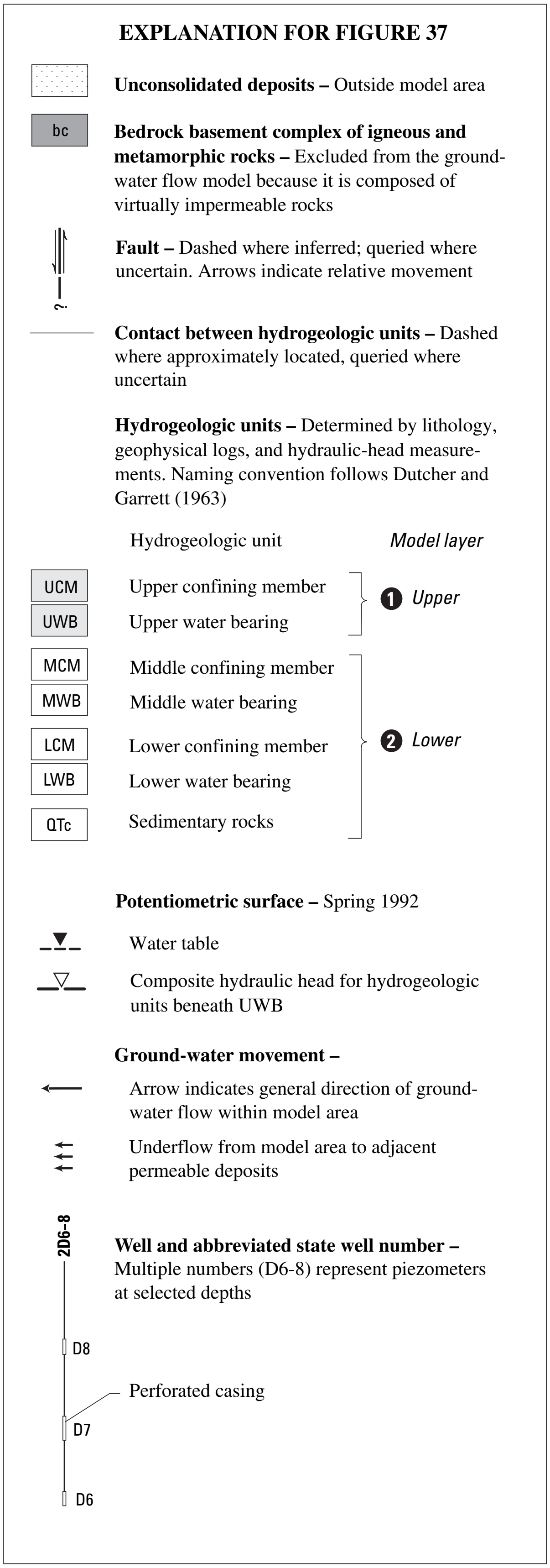


Figure 37.—Continued.

The ground-water flow model is a mathematical representation of ground-water flow through the valley-fill aquifer. In order to solve the equations that constitute the model, it is necessary to make simplifying assumptions about the valley-fill aquifer and the physical processes governing ground-water flow. The most important of these assumptions are embodied in the conceptual model of the valley-fill aquifer. Although the ground-water flow model cannot be as detailed or as complex as the real system, the model is useful in at least four ways: (1) the model integrates and assures consistency among aquifer properties, recharge, discharge, and ground-water levels; (2) the model can be used to estimate flows and aquifer characteristics for which direct measurements are not available; (3) the model can be used to simulate response of the valley-fill aquifer under hypothetical conditions; and (4) the model can identify sensitive areas where additional hydrologic information could improve understanding.

Conceptual Model

The conceptual model, which is the basis for the numerical ground-water flow model, is derived mostly from the hydrogeologic setting and hydrogeologic units described by Dutcher and Garrett (1963). Essentially the same conceptual model was used by Hardt and Hutchinson (1980) to develop a previous ground-water flow model of the San Bernardino area.

The unconsolidated and poorly-consolidated sediments filling the Bunker Hill and Lytle Creek basins compose the valley-fill aquifer and are considered to be the permeable part of the ground-water system. Igneous and metamorphic rocks underlying and surrounding the valley-fill aquifer are assumed to be impermeable (*fig. 22*). Sedimentary rocks that bound the southwestern edge of the valley-fill aquifer are assumed to be poorly permeable and to transmit only small amounts of water to the valley-fill aquifer. Part of the perimeter of the valley-fill aquifer is defined by faults, each with a somewhat different capability for transmitting ground water. The transmissive character of the bounding and internal faults also varies with depth (*fig. 24*).

The valley-fill aquifer is conceptualized as having two highly transmissive layers: an upper layer composed of hydrogeologic units UCM and UWB; and a lower layer composed of hydrogeologic units MCM, MWB, LCM, LWB, and QTc. This conceptualization of hydrogeologic units and model layers is shown in *figure 37*. Most ground-water flow in the upper and lower layers occurs in the UWB and MWB units, respectively. Flow between the two layers is restricted by numerous, fine-grained deposits, found mostly in the MCM unit, that act as a confining bed. Near the mountain front, the fine-grained deposits thin to extinction, and the two layers act as one. The transmissivity and storage coefficient of each layer are assumed to remain constant over time.

Hydrogeologic unit UCM, which is a fine-grained and discontinuous deposit, can create locally semi-confined or perched conditions near the land surface. This unit is particularly important in water management because it retains water at the land surface and is susceptible to liquefaction during an earthquake. An alternate conceptualization of the valley-fill aquifer could have identified UCM as a separate layer; however, this approach would have required using two layers for UWB because it vertically spans UCM (*fig. 37*). Also, three-dimensional mapping of UCM is insufficient to adequately characterize UCM as a separate layer.

The primary source of recharge to the aquifer is runoff from the surrounding mountains; the primary discharge is to pumped wells. Important, but lesser quantities of ground water flow into and out of the valley-fill aquifer through some sections of the bounding faults (*fig. 22*). Evapotranspiration and ground-water discharge to Warm Creek (*fig. 21*) also affect ground-water flow when ground-water levels are near land surface.

The conceptual model is the basis for formulating the numerical ground-water flow model. Although the conceptual model is a simplification of the real system, additional hydrogeologic information not included in the conceptual model can be combined with results from the ground-water flow model to achieve an improved, more quantitative understanding of the ground-water system (*fig. 3*). Relations among field data, the conceptual model, and the numerical model are shown schematically in *figure 38*.

Relation to Previous Flow Model

The ground-water flow model documented in this report is a revision and an update of the ground-water flow model described by Hardt and Hutchinson (1980). General characteristics of the two models are compared in *table 12*. Both models simulate ground-water flow in essentially the same area surrounding the city of San Bernardino, and both models were designed using similar hydrogeologic concepts of how ground water flows through the valley-fill aquifer. Both models also use the same vertical discretization of the aquifer.

The primary reason for revising the previous model was to improve simulation of recharge and discharge. The computer code used for the previous model required that most recharge and discharge components be combined into a single dataset (Durbin, 1978). This approach made identifying or modifying specific recharge or discharge components difficult or impossible. The present model uses a modular computer code (McDonald and Harbaugh, 1988) specifically designed to overcome these limitations. Each module or package is a set of computer subroutines designed to calculate a separate part of the model. Recharge and discharge components can be separated into different packages and readily critiqued, modified, or updated. In addition, simulation of some recharge and discharge components was improved by using a more realistic approximation of the actual physical process. For example, a streamflow-routing package was used to simulate the interaction between streams and ground water (Prudic, 1989). This package explicitly calculates not only the quantity of water exchanged, but also the quantity of water remaining in each stream—an important feature in an area of intermittently flowing streams. Use of this package can aid in developing linked water budgets for both the surface-water and ground-water systems.

Other reasons for modifying the previous model were to increase spatial resolution by using a finer areal discretization, correct an inappropriate use of specific yield in the lower model layer, and update the model to more recent (1998) conditions. To improve simulation of ground-water flow near the numerous faults in the area, a horizontal-flow-barrier package was included in the revised model. This package by Hsieh and Freckleton (1993) does not require reductions in the transmissivity of selected cells in order to simulate a fault, as was required in the previous model. Finally, the revised model grid was registered to latitude and longitude in order to take advantage of GIS databases.

Design and Discretization

The conceptual model of ground-water flow in the valley-fill aquifer was converted into a numerical model in the following way. The aquifer is approximated by an upper, unconfined model layer and a lower, confined model layer (*figs. 37* and *38*). Transmissivity and storage coefficients within each layer are assumed to vary spatially, but not temporally. Horizontal flow within the layers is described by

$$S\frac{\delta h}{\delta t} = \frac{\delta}{\delta x}\left(T_x\frac{\delta h}{\delta x}\right) + \frac{\delta}{\delta y}\left(T_y\frac{\delta h}{\delta y}\right) + W + Q_Z \qquad (8)$$

where

h	is hydraulic head (L);
Q_z	is vertical leakance (L/T);
S	is storage coefficient (dimensionless);
t	is time (T);
T	is transmissivity (L^2/T);
W	is a combination of sources and sinks (L/T); and
x, y	are cartesian coordinates (L).

Flow between the upper and lower model layers is assumed to be vertical. The rate of flow is affected by the presence of intervening fine-grained deposits, which reduce the vertical hydraulic conductivity. This linkage between the upper and lower model layers is described by

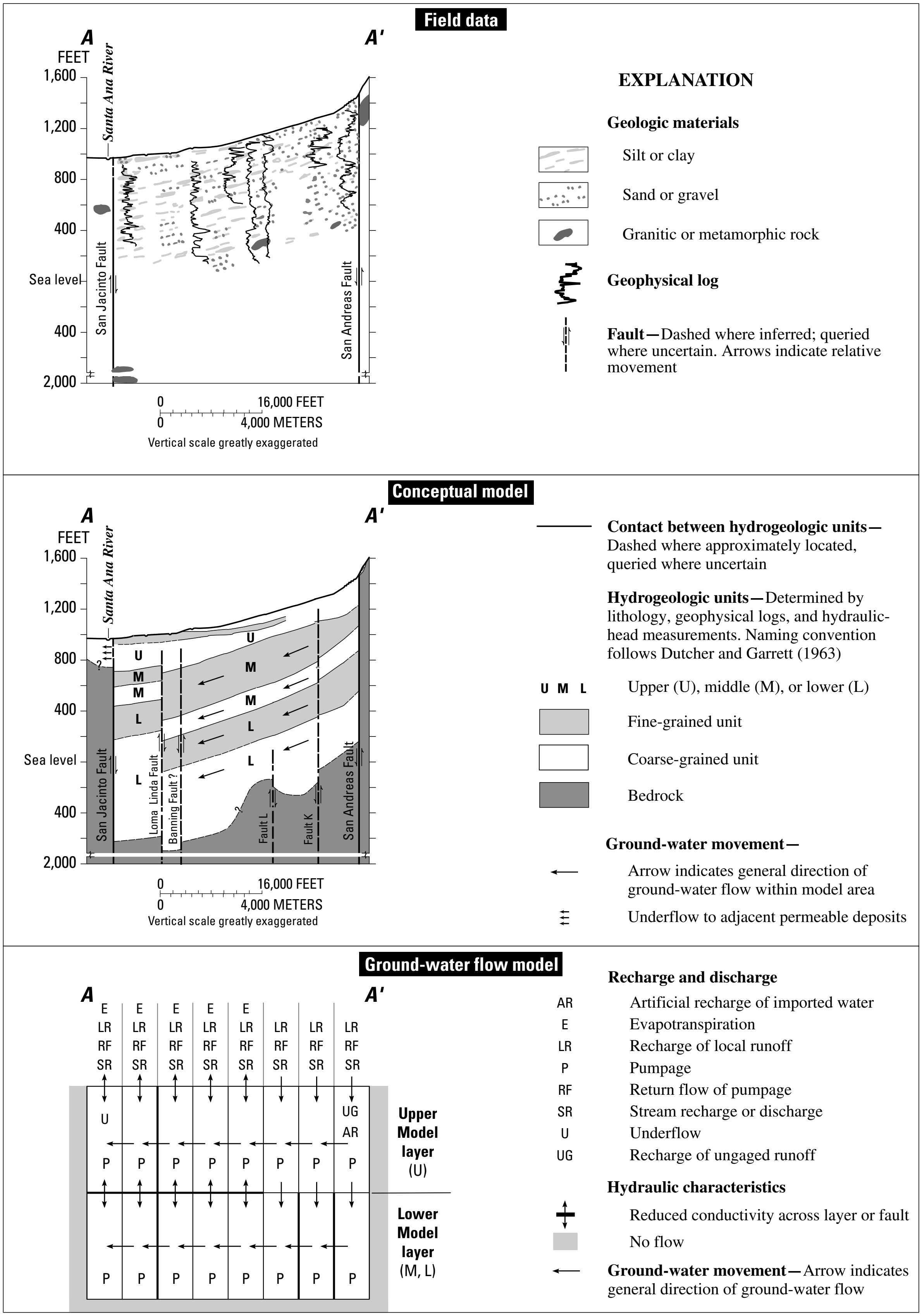


Figure 38. General relations among field data, conceptual model, and ground-water flow model of the San Bernardino area, California.

$$Q_z = -K_z \frac{dh}{dz} \qquad (9)$$

where

K_z is vertical hydraulic conductivity (L/T); and

z is a cartesian coordinate (L).

Equations 8 and 9 were solved using the modular, three-dimensional, finite-difference computer code developed by McDonald and Harbaugh (1988).

Use of finite-difference techniques requires discretization of the valley-fill aquifer into a geometrically simplified form, or model grid. The grid used for the San Bernardino area consists of 112 rows and 184 columns of individual cells, each with a uniform areal dimension of 250 m by 250 m (about 820 ft by 820 ft). Each model cell covers about 15 acres of land. Only cells inside the boundaries of the valley-fill aquifer (*fig. 22*) are used in the actual simulation.

The vertical dimension of the valley-fill aquifer was approximated by two model layers (*fig. 37*), which are synonymous with the upper and lower layers of the valley-fill aquifer. The model layers are superimposed exactly, one on top of the other. Each model layer has the same origin, the same areal extent, the same configuration of active cells, and the same number of active cells (3,844). Vertical flow between the model layers is restricted wherever intervening fine-grained deposits are present. Where the fine-grained deposits are absent, such as near the base of the mountains, flow between the model layers is not restricted, and the two layers act effectively as one.

The orientation and dimension of the model grid were chosen so that the grid would align precisely with the Universal Transverse Mercator (UTM) coordinate system (Synder, 1985, 1987). The UTM coordinate system is derived from a rectilinear projection of latitude and longitude and is displayed on most topographic and geologic maps. UTM measurements are in meters north of the equator and meters east or west of a base meridian, in this case 117°W. To avoid negative values, the base meridian is given a value of 500,000 m.

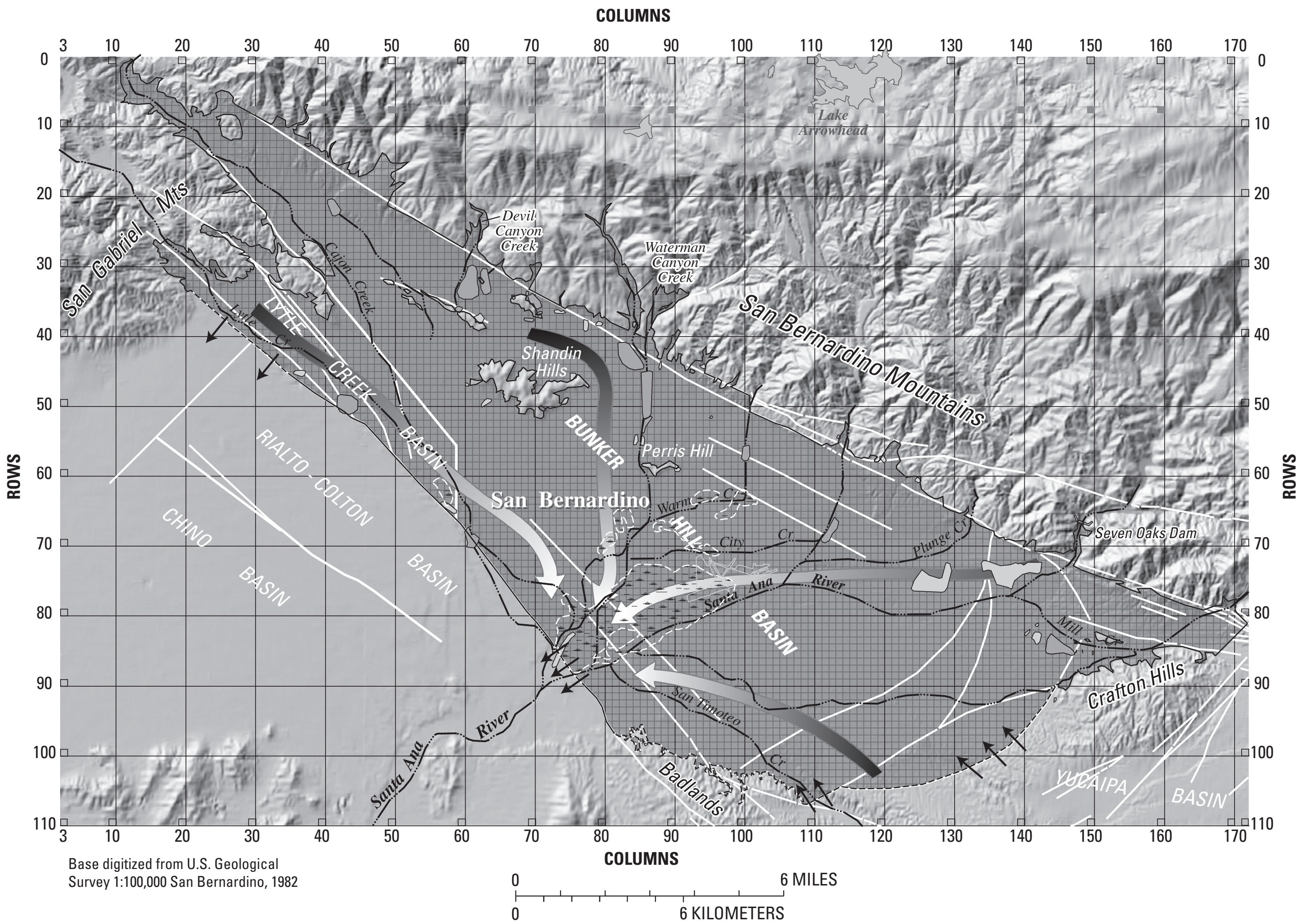
The model grid and active model cells are shown in *figure 39*. Selected coordinates of the model grid are listed in *table 13*. Transformation from one coordinate system to another (latitude/longitude, UTM, model) can be done using standard computer programs (Synder, 1985, 1987; Environmental Systems Research Institute, 1992). The model grid also was designed sufficiently large so that the Rialto-Colton area to the southwest (Woolfenden and Koczot, 2001) could be added readily to the present model. The model grid also can be extended to the east to include the Yucaipa area (*fig. 37*). Both areas are the subject of ongoing investigations, which are developing sufficient hydrogeologic knowledge to extend the simulation of ground-water flow throughout the San Bernardino, Rialto-Colton, and Yucaipa areas.

The computer code selected for the ground-water flow model was chosen largely because of its flexible design (McDonald and Harbaugh, 1988). Not only can individual recharge and discharge components be simulated by individual packages, but new packages can be added to the model code. The several packages that are used in the ground-water flow model of the San Bernardino area are described in *table 14*. Eight new packages (ART1, GHB2, LOC1, MAN1, PUM1, RFL1, UND1, UNG1) were developed as part of this study to aid in simulating specific recharge and discharge components; creation of these packages required only minor modifications of existing packages (REC1, WEL1). A new streamflow routing package was developed (STR2) from the existing package (STR1 by Prudic, 1989); these modifications are more complex and are documented as part of a separate study (Danskin and Hanson, 2003).

Table 13. Coordinates of the ground-water flow model of the San Bernardino area, California.

[Model grid is aligned with the Universal Transverse Mercator (UTM) coordinate system; coordinates below are calculated at the outside edge of the model grid using the North American Datum of 1927; each model cell is 250 meters by 250 meters]

Corner of model grid	Model coordinates		Latitude	Longitude	UTM coordinates, zone 11 (meters)	
	X (columns)	Y (rows)	(Decimal value in parentheses)		X (east)	Y (north)
Northwest	0.00	0.00	34° 15' 55.40" (34.265388)	117° 29' 58.78" (117.499662)	454,000.	3,791,500.
Northeast	184.00	0.00	34° 15' 59.06" (34.266407)	117° 00' 00.00" (117.000000)	500,000.	3,791,500.
Southwest	0.00	118.00	33° 59' 57.62" (33.999338)	117° 29' 53.16" (117.498099)	454,000.	3,762,000.
Southeast	184.00	118.00	34° 00' 01.25" (34.000347)	117° 00' 00.00" (117.000000)	500,000.	3,762,000.



EXPLANATION

Boundary of the valley-fill aquifer—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray. Solid line indicates no underflow. Dashed line indicates area of minor underflow; arrows indicate area with more than 1,000 acre-feet per year of underflow and point in the direction of ground-water flow to or from adjacent permeable materials

Fault or ground-water barrier—May be concealed or approximately located.

Former marshland

Artificial-recharge basin

Single cell of the ground-water flow model—Model finite-difference grid is registered to latitude and longitude (table 10)

Generalized direction of ground-water flow within the valley-fill aquifer

Figure 39. Location, orientation, and boundary conditions of the ground-water flow model of the San Bernardino area, California.

Table 14. Computer programs (packages) used with the ground-water flow model of the San Bernardino area, California.

Computer program (package)	Function	Reference
	Basic code	
Primary computer code	Setup and solve equations simulating a basic ground-water flow problem.	McDonald and Harbaugh (1988).
Preconditioned conjugate gradient solver (PCG2)	Improved solution of ground-water flow equations; requires convergence of heads and (or) flowrates.	Hill (1990).
	Aquifer parameters	
Horizontal flow barrier package (HFB1)	More localized simulation of faults so that model transmissivity values are not affected by simulation of faults.	Hsieh and Freckleton (1993).
	Recharge and discharge	
Precipitation recharge package (REC1)	Simulates distributed recharge from precipitation to the uppermost model layer.	McDonald and Harbaugh (1988).
Local recharge package (LOC1)	Simulates distributed recharge from local runoff in the same manner as the original recharge package (REC1).	Minor modification of McDonald and Harbaugh (1988).
Artificial recharge of imported water (ART1)	Simulates specified recharge or discharge in the same manner as the original well package (WEL1).	Minor modification of McDonald and Harbaugh (1988).
Pumpage package (PUM1)	ditto	ditto
Return flow package (RFL1)	ditto	ditto
Underflow package (UND1)	ditto	ditto
Ungaged recharge package (UNG1)	ditto	ditto
Management package (MAN1)	Simulates additional recharge or pumpage for water-management scenarios in the same manner as the original well package (WEL1).	Minor modification of McDonald and Harbaugh (1988).
Well package (WEL1)	Simulates additional recharge or pumpage for constrained optimization scenarios.	McDonald and Harbaugh (1988).
Evaporation package (EVT1)	Simulates head-dependent evaporation from the upper layer of the ground-water flow model.	McDonald and Harbaugh (1988).
General head package (GHB2)	Simulates head-dependent underflow beneath the Santa Ana River based on a nonlinear regression equation and calculated head at the Heap well (1S/4W–3Q1).	Minor modification of McDonald and Harbaugh (1988).
Streamflow-routing package (STR2)	Improved simulation of surface-water and ground-water interaction; routes and mass-balances streamflow.	Original streamflow routing package by Prudic (1989) was modified by Danskin and Hanson (2003) to allow for different types of diversions.

Several additional packages have been created that offer powerful simulation capabilities, including the transient release of water from storage (TLK1) (Leake and others, 1994); permanent loss of storage and compaction of the aquifer (ISB1 and CHD1) (Leake and Prudic, 1988); and rewetting of a dewatered model cell (BCF2) (McDonald and others, 1991). Although powerful and potentially applicable to the San Bernardino area, these packages generally require three-dimensional data not available for the permeable hydrogeologic units or create hydraulic non-linearities that cause major difficulties in use of the constrained optimization model. As additional hydrogeologic data become available, the present model could be modified to include one or more of these packages. Adding new capabilities to the present model is aided by the model's modular design and facilitates an evolutionary modeling process.

Aquifer Parameters

Simulation of the upper and lower model layers requires defining a transmissivity and storage coefficient for each active model cell in each model layer. Commonly, these values are derived from aquifer tests and from concepts about the depositional history of the aquifer materials (Hollett and others, 1991).

In the San Bernardino area, aquifer-test data are limited; as a result, the California Department of Water Resources (1971, p. 64–69, 85–98) used more than 1,000 driller's logs to estimate values of transmissivity and storage coefficient. The procedure was as follows. Initial transmissivity values were calculated from specific-capacity tests, divided by the total length of perforations as indicated on the driller's log of that well, and then multiplied by an estimate of the entire saturated thickness of the aquifer in that area. These initial values were contoured and used to select values for the mathematical, ground-water flow model of the area developed by the California Department of Water Resources (1971). The initial transmissivity values were modified later during calibration of that model. Specific-yield values were calculated from driller's logs using defined values for each type of aquifer material found in the driller's description, for example 0.03 for clay and 0.35 for medium sand. Average specific-yield values were calculated only for that part of the well log that was variably saturated during water years 1935–60. As in the calculation of transmissivity, the initial specific-yield values were used to select model values of specific yield, which subsequently were modified during calibration. The final specific-yield values ranged from 0.048 to 0.35 with a mean of 0.13 (California Department of Water Resources, 1971, table 7).

Values of transmissivity and storage coefficient developed by the California Department of Water Resources (1971) were used by Hardt and Hutchinson (1980) in developing their ground-water flow model. Transmissivity values were divided between the upper and lower model layers. Values of storage coefficient for the confined lower aquifer were estimated from aquifer tests in the San Bernardino area and in other areas with similar sediments. Calibration of the ground-water flow model by Hardt and Hutchinson (1980, p. 15) resulted in some modification of the initial values of both transmissivity and storage coefficient.

Development of the present ground-water flow model used the final values of transmissivity from Hardt and Hutchinson (1980, p. 72–80). Values for each layer of the new model grid were obtained by interpolating the previous values using an inverse distance-squared weighting. Because design of the Hardt and Hutchinson (1980) model required using a reduced value of transmissivity to simulate the effect of a fault, these reduced values were not used in the interpolation. Other methods of interpolation including kriging were tested, but these produced minimal change in the values. Use of original values of transmissivity obtained from aquifer tests and calculated by the California Department of Water Resources (1971) from driller's logs may have been preferable in revising the present model, but most of these values were no longer available.

Values of storage coefficient for the upper and lower model layers were changed substantially from those used by Hardt and Hutchinson (1980, p. 72–80). The changes were required for two reasons. First, the storage coefficient of some cells in the lower model layer of Hardt and Hutchinson (1980) inappropriately reflects unconfined conditions. A review of historical ground-water levels failed to identify any periods when unsaturated conditions were present at the top of both the upper and lower layers. In the San Bernardino area with the present model formulation, only the upper model layer should have an unconfined value of storage (specific yield). The lower layer, even if it directly underlies an unconfined body of water, should have a storage coefficient based only on expansion of water and compression of aquifer material, not on actual dewatering of the aquifer. Second, the storage coefficient of some cells in the upper model layer of Hardt and Hutchinson (1980) reflects confined conditions. The fluctuations in ground-water levels during 1945–98 in the area of the former marshland indicates that the upper model layer was actually dewatering, not simply depressurizing, as the confined storage coefficients of Hardt and Hutchinson (1980) would indicate.

The upper model layer, conceptualized as unconfined, was assigned specific-yield values from Eckis (1934, map E). Eckis' map E shows contours of specific yield for a uniform thickness of the aquifer, 50 ft above and 50 ft below the water table in 1933. The specific-yield contours were digitized, rasterized, and gridded using ARCINFO software (Environmental Systems Research Institute, 1992) in order to calculate a specific-yield value for each model cell. Some small areas of the model were outside the original contours by Eckis. For these areas, which included most of the former marshland, depositional concepts were used to extend the original contours.

The lower model layer, conceptualized as confined or not physically dewatered, was defined as having a storage coefficient of 0.0001, a value typical of a 500- to 700-ft thickness of unconsolidated sediment (Driscoll, 1986, p. 210). Distribution of storage coefficients that vary within the lower model layer would require three-dimensional data on thickness and specific storage for hydrogeologic units MCM, MWB, and LCM, and possibly LWB (*fig. 37*). As of 1998, these data were not available except for selected parts of the San Bernardino area.

Two model layers in a quasi-three-dimensional simulation, such as in the previous model by Hardt and Hutchinson (1980) and in this revised model (eqs. 8 and 9), are connected hydraulically by a leakance coefficient or vertical conductance. Leakance coefficients from Hardt and Hutchinson (1980, p. 72–80) were interpolated in the same way as the transmissivity values in order to develop vertical-conductance values for the new model cells. The original leakance coefficients were derived largely by trial-and-error during calibration (Hardt and Hutchinson, 1980, p. 17, 45). The assumptions and appropriate application of vertical conductance (leakance coefficient) are described in detail by McDonald and Harbaugh (1988, p. 2–29 to 2–35).

The restriction in ground-water flow caused by the several faults and barriers in the area (*fig. 5*) was simulated by the horizontal-flow-barrier (HFB) package of Hsieh and Freckleton (1993). This package calculates flow across a horizontal barrier, such as a fault, using a horizontal conductance term that applies to only one side of a model cell. Other methods, such as used by Hardt and Hutchinson (1980), require reducing the transmissivity of an entire model cell to simulate the effect of a fault or barrier. Use of the HFB package results in a more accurate simulation of ground-water levels near faults and barriers. This improvement is especially apparent in model cells with significant recharge or discharge, such as from streams or pumped wells. Reduced values of transmissivity used by Hardt and Hutchinson (1980, p. 72–80) to simulate faults and barriers were used directly as values of horizontal conductance for the HFB package. Locations of the faults were taken from recent mapping by Matti and Carson (1991, pl. 1), and locations of barriers were taken from Dutcher and Garrett (1963, p. 1).

Transmissivity for both model layers, vertical conductance between layers, and specific yield for the upper layer are illustrated in *figure 40*. In general, the highest values of transmissivity in both layers are near the center of the Bunker Hill basin and along the stream channels of Lytle Creek, East Twin Creek, and the Santa Ana River (*fig. 11*). In these areas, the geologic structure and depositional history of the basin have combined to produce deposits that are thick, coarse, and well-sorted.

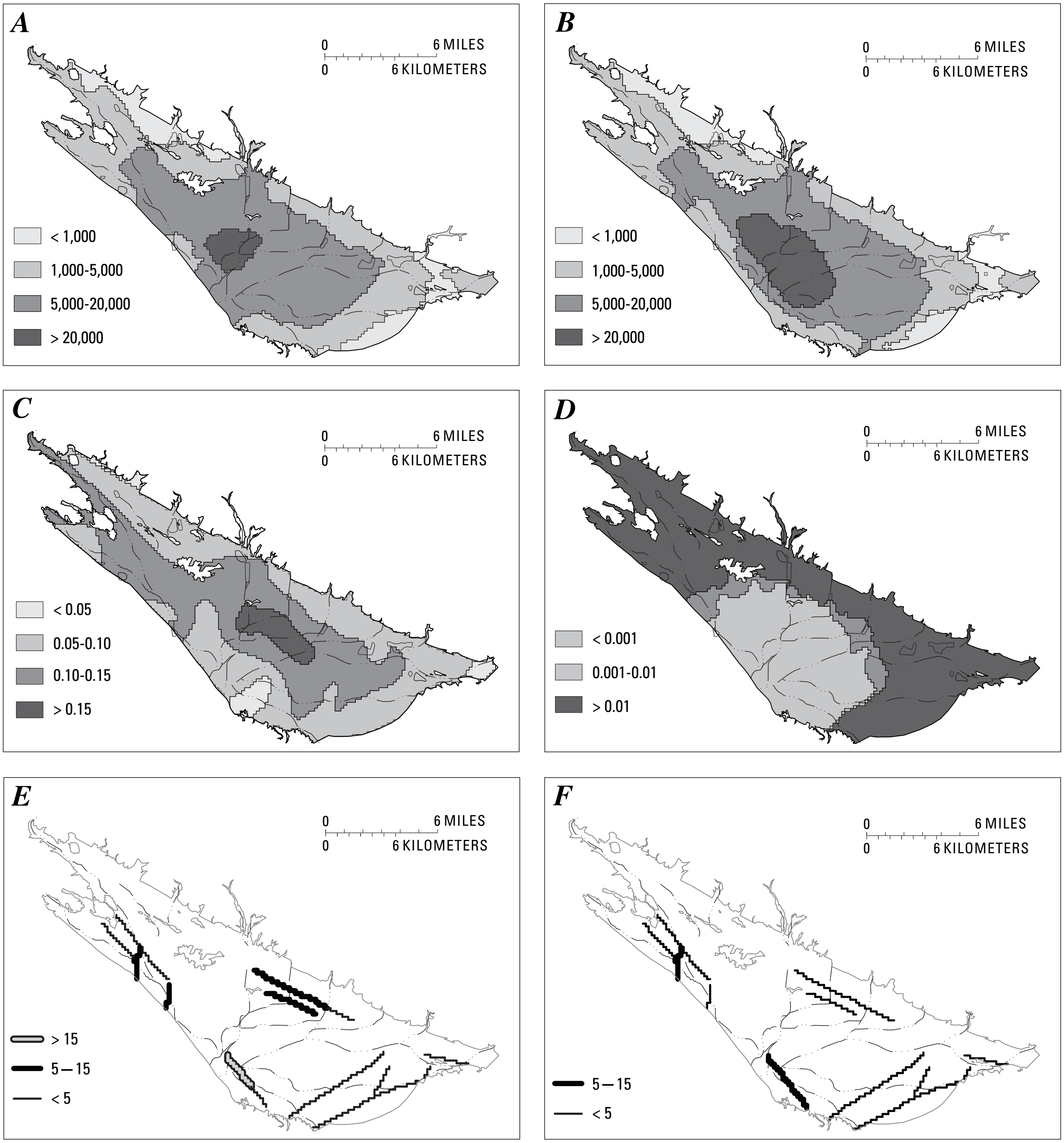
Reduced values of transmissivity are present in the lower model layer between the San Jacinto and Loma Linda faults where the more permeable valley-fill deposits are thinner (*fig. 37*). Vertical ground-water flow is impeded only where vertical conductance is less than about 0.01 per day^{-1} (*fig. 40*). The area of impediment is the confined area identified by Hardt and Hutchinson (1980, fig. 3) and is the approximate area of flowing wells identified by Mendenhall (1905, pl. 8) and shown in *figures 9* and *37*.

In other areas of the Bunker Hill and Lytle Creek basins, such as near the base of the San Bernardino and San Gabriel Mountains, vertical-conductance values are sufficiently high (greater than 0.01 per day^{-1}) that vertical flow is unrestricted. In these areas, measured ground-water levels for both the upper and lower aquifer layers are nearly the same, as are simulated hydraulic heads for the upper and lower model layers. Values for specific yield range from less than 0.05 near the former marshland, to greater than 0.15 in the middle of the Bunker Hill basin. Compared to the specific-yield values used by Hardt and Hutchinson (1980, fig. 10), the present values along the mountain front are generally lower and the present values in the former marshland are generally higher.

Recharge and Discharge

Most recharge and discharge components in the ground-water flow model were changed in some way from the previous work by Hardt and Hutchinson (1980). In so far as possible, each physical type of recharge or discharge, such as recharge from gaged runoff or discharge from wells, is simulated with a discrete model package. In this way, recharge and discharge processes can be simulated more realistically, and the values can be updated as new information is obtained or as human actions alter hydrologic characteristics of the streams, wells, or valley-fill aquifer. The various recharge and discharge components included in the ground-water flow model are summarized in *table 12*. A more complete description of each component along with measured and estimated values can be found in the section of this report entitled "Hydrology." A generalized ground-water budget of average values for 1945–98 is listed in *table 11*, and a piechart of the values is shown in *figure 33*. Also listed in *table 11* are the maximum and minimum annual values for comparision.

The following discussion describes only the numerical approximations necessary to represent the physical processes and related values in the model. Simulation of some recharge and discharge components included modifications to standard computer packages by McDonald and Harbaugh (1988); these changes are summarized in *table 14*. Simulated annual recharge and discharge values for 1945–98 are listed in *table 15*.



EXPLANATION

Units—Transmissivity in feet squared per day; vertical conductance in per day; specific yield is dimensionless; hydraulic characteristic of faults in feet per day

Basin boundary—For Bunker Hill and Lytle Creek ground-water basins

Figure 40. Areal distribution of calibrated hydraulic parameters of the ground-water flow model of the San Bernardino area, California: *A*, transmissivity of the upper model layer; *B*, transmissivity of the lower model layer; *C*, specific yield of the upper model layer; *D*, vertical conductance between model layers; *E*, hydraulic characteristic of faults in the upper model layer; and *F*, hydraulic characteristic of faults in the lower model layer. Storage coefficient for the lower model layer equals 0.0001.

Direct recharge of precipitation.—The standard recharge package (RCH1) by McDonald and Harbaugh (1988, p. 7–1) was used to simulate recharge that results from infiltration of direct precipitation on the unconsolidated deposits of the San Bernardino area. Although the quantity is undoubtedly small because of the semiarid climate, some infiltration to the water table probably occurs during extremely wet periods (*fig. 18*). The significant depth to ground water, especially near the base of the mountains (*fig. 37*), suggests that most recharge migrates slowly through the unsaturated zone, possibly taking years to reach the water table. In areas near the former marshland, infiltrated precipitation probably arrives sooner, but still essentially at a constant rate.

To simulate this small quantity of recharge from precipitation, the map of average precipitation throughout the San Bernardino area (*fig. 6*) was used to develop a model input array. Precipitation contours in figure 6 were digitized, rasterized, and gridded using ARCINFO software (Environmental Systems Research Institute, 1992) in order to calculate a recharge value for each model cell in layer 1. This array of values was scaled to total 1,137 acre-ft/yr, the quantity of direct recharge estimated to occur from precipitation for 1945–98 (*fig. 18*; p. 40). This technique for developing a model array from contoured data is the same as that used to develop a specific-yield array (p. 69). These assumptions and method of simulating direct recharge of precipitation are comparable to methods used successfully in other semiarid ground-water basins (Yates, 1988, p. 14; Bright and others, 1997, p. 41; Danskin, 1998).

Recharge and discharge of gaged streamflow.—The streamflow-routing package of Prudic (1989) with modifications by Danskin and Hanson (2003) was used to simulate interaction between the major streams (*fig. 2*) and the valley-fill aquifer. Annual discharge at the mountain-front gage (*fig. 11*; *table 1*) was used as inflow to each stream. Streamflow was routed down the stream channels, through the artificial-recharge basins, and past the outflow gages near the San Jacinto fault (*fig. 41*). Interaction between the stream and the valley-fill aquifer was simulated with a Darcian relation that calculates flow to or from the aquifer based on head in the aquifer, head in the stream, and conductance of the streambed (Prudic, 1989, p. 7).

Five different types of streambeds were simulated in the streamflow-routing model (*table 16*). Two types represent natural stream channels with earthen bottoms: one for wide streams (Santa Ana River, Lytle Creek, and Mill Creek) and one for narrow streams (Cajon Creek, Cable Creek, East Twin Creek, Warm Creek, City Creek, Plunge Creek, and Zanja). A third type represents artificial-recharge basins, where streamflow generally is wider than in the natural channel and is routed through the basin in a serpentine fashion to maximize recharge. A fourth type represents concrete-lined channels or pipes that have minimal leakage. The fifth type represents a logical connection that has no real length or recharge, but is necessary for routing.

Conductance for each type of streambed was calculated based on the average wetted width of the stream and on the estimated vertical hydraulic conductivity and thickness of the streambed deposits. The average wetted width of each stream, even the three largest streams (*fig. 12*), is less than the width of a single model cell (about 820 ft). Under most conditions and in most years, the wetted width remains relatively constant and the stream stage varies with discharge. However, during unusually wet years with major floods such as occurred in 1969 and 1998 (*fig. 14*), flow overtops the main stream channel and covers a much larger width of the entire braided stream channel. This condition is most apparent for the Santa Ana River, Lytle Creek, and Mill Creek. The maximum wetted width of these streams is indicated by the width of the river-channel deposits shown in *figure 5*.

Table 16. Simulated streambed characteristics in the San Bernardino area, California.

[Unusually wet runoff years are 1958, 1969, 1978, 1979, 1980, 1983, 1993, and 1998]

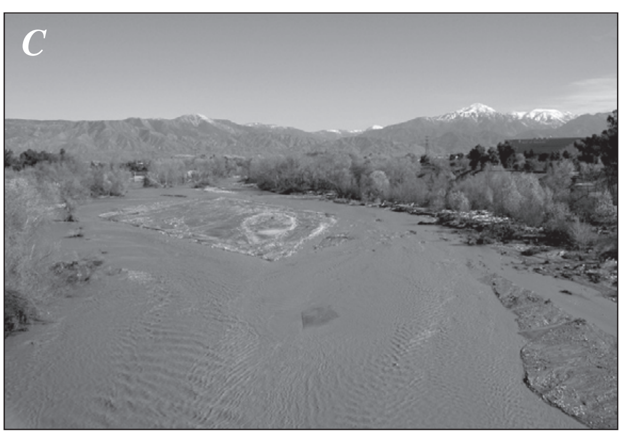
Type of streambed	Simulated streambed conductance for different types of runoff years (feet per second)		Percentage increase in simulated streambed conductance from dry or normal year to unusually wet runoff year	Comment
	Dry or normal year	Unusually wet year		
Natural channel, wide	0.075	0.375	500	Includes Santa Ana River, Mill Creek, and Lytle Creek.
Natural channel, narrow	0.05	0.15	300	All other streams.
Spreading basin	0.15	0.3	200	Artificial-recharge basin.
Concrete	0.00005	0.00005	100	Concrete-lined channel, or pipe.
Logical	0.000001	0.000001	100	No length, only a logical connection.

Seven Oaks Dam, April 1999. Photo courtesy of David Lovell, San Bernardino County.

Ungaged mountain-front runoff, January 2005.

Santa Ana River artificial-recharge basins, May 1930. Photo courtesy of San Bernardino Valley Water Conservation District.

Warm Creek natural channel, September 2004.



Santa Ana River, January 2005.

Warm Creek bypass channel, September 2004.

Figure 41. Major surface-water features in the San Bernardino area, California.

Also during wet years, increased streamflow remobilizes and removes fine-grained materials that previously were deposited on the streambed. This removal increases vertical hydraulic conductivity, and hence streambed conductance. This mechanism likely accounts for the dramatic increase in streambed conductance for large streams (500 percent), compared to narrow streams (300 percent) and spreading basins (200 percent) (*table 16*).

The version of the streamflow-routing package (STR2) used for this report does not have the capability of simulating an increased wetted perimeter involving additional model cells. Therefore, any additional recharge that occurs as a result of unusually large runoff and the concurrent increase in stream width needs to be simulated as an increase in streambed conductance (*table 16*). Perhaps because the ground-water model simulates annual recharge and discharge, this limitation does not appear to be significant in this present study. For a situation where simulated effects close to the major streams are important, especially after wet years with abundant runoff, then the lack of increasing stream width by adding more model cells may be a significant limitation.

Recorded discharge at the outflow gages (*table 1*) was used in combination with estimates of local runoff (*table 4*), measured wastewater discharge near the outflow gages (*table 7*), and the estimated quantity of rising ground water (*fig. 30*; *table 8*) to evaluate the performance of the streamflow package. Simulated values of recharge from gaged mountain-front runoff for 1945–98 are listed in *table 15*. Also listed is simulated discharge to the gaged streams, also referred to as rising ground water.

During development of the ground-water flow model, a much more complex version of simulating the surface-water system was tested using the revised streamflow-routing package (STR2). Essentially all surface water in the San Bernardino area (*pl. 1*), both gaged and ungaged, in streams and in pipes, was routed as shown on *plate 2*. Four different types of surface-water diversions were simulated using STR2. The more complex routing of surface water worked well, but the ground-water flow model became numerically unstable during periods of low runoff. As a result, this version of streamflow routing was not used in the final ground-water flow model. If the numerical instability can be resolved, the more complex surface-water routing package would be an important enhancement to the present model.

Recharge of ungaged runoff.—Ungaged runoff from the surrounding mountains and from the few, small bedrock outcrops within the San Bernardino area was estimated using average precipitation (*fig. 6*), drainage areas, an effective percentage of runoff (*table 3*), and the annual runoff index for the Santa Ana River (*table 2*). Because virtually all ungaged runoff was assumed to recharge the valley-fill aquifer, annual values of ungaged recharge equal the estimated average ungaged runoff multiplied by the annual runoff index for the Santa Ana River (refer eq. 1). These annual values were used directly in the ground-water flow model.

Recharge from ungaged runoff was simulated in the ground-water flow model as an annual specified flux using a modified version (UNG1) of the standard well package (WEL1) by McDonald and Harbaugh (1988, p. 8–1). The only modifications to the WEL1 package were changes in variable names in order to prevent conflicts in referencing. The total quantity of recharge for each area shown in *figure 17* was distributed evenly in model cells along the perimeter of the model boundary that coincides with the boundary of the ungaged area (*figs. 17* and *39*). Annual values of recharge from ungaged runoff for 1945–98 are listed in *table 15*.

Recharge of imported water.—Surface water imported into the San Bernardino area can be recharged in several artificial-recharge basins located near the head of alluvial fans (*fig. 11*; *table 5*). Some of the imported water may evaporate or may be transpired by native vegetation in the artificial-recharge basins; however, nearly all (90 percent) is assumed to recharge the valley-fill aquifer.

Recharge from imported water is simulated in the ground-water flow model as an annual specified flux using a modified version (ART1) of the standard well package (WEL1) by McDonald and Harbaugh (1988, p. 8-1). The only modifications to the WEL1 package were changes in variable names in order to prevent conflicts in referencing. Data requirements include: the measured quantity of imported water for each basin (*table 5*); the area of recharge within each basin, which was identified from topographic maps and aerial photographs; and the percentage of imported water in each basin that actually recharges the valley-fill aquifer. Annual values of recharge of imported water for 1945–98 are listed in *table 15*.

Gross pumpage and return flow.—Pumpage from wells is the major component of discharge from the valley-fill aquifer (*table 11*). Most of this pumped water is used consumptively; a lesser quantity returns as recharge (return flow) to the upper layer of the valley-fill aquifer (*fig. 25*).

Values of gross pumpage and return flow for individual wells were developed from data from the local watermaster (Western–San Bernardino Watermaster, 2000). A pre-processing program used these data and characteristics about each well to calculate the quantity of gross pumpage from each model layer and the quantity of pumpage returned as recharge to the upper model layer. These calculations are described in greater detail in the section of this report entitled "Pumpage." Annual values of gross pumpage and return flow for 1945–98 are listed in *table 15*.

Gross pumpage and return flow are simulated using separate, but virtually identical model packages. The rationale for this accounting practice is that each quantity (gross pumpage from layer 1, gross pumpage from layer 2, and return flow to layer 1) is kept separate and can be compared and analyzed individually. A common practice in previous hydrologic studies is to analyze and report net pumpage, defined as gross pumpage minus return flow. Caution is warranted, therefore, when comparing pumpage values from this report with those in prior reports to ensure that both are either gross or net, not a mixture of the two.

Gross pumpage is simulated in the model using a modified version (PUM1) of the standard well package (WEL1) by McDonald and Harbaugh (1988, p. 8–1). The only modifications to the WEL1 package were changes in variable names in order to prevent conflicts in referencing. Return flow also is simulated using a modified version (RFLl) of the standard well package (WEL1) by McDonald and Harbaugh (1988, p. 8–1). Similarly, the only modifications to the WEL1 package were changes in variable names in order to prevent conflicts in referencing.

Evapotranspiration.—Evapotranspiration is assumed to occur from the valley-fill aquifer whenever the water table is sufficiently close to land surface. Field studies in similar environments have identified this distance to be about 15 ft or less, depending on the type of vegetation and soil characteristics (Lee, 1912; Robinson, 1958; Sorenson and others, 1991; Danskin, 1998). A maximum evapotranspiration rate is reached when the water table is at land surface.

These conditions for evapotranspiration were simulated using the standard evapotranspiration package (EVT1) by McDonald and Harbaugh (1988, p. 10–1). In this package, a depth-dependent relation is used to calculate the quantity of evapotranspiration from the upper layer of the ground-water flow model. The relation assumes that at land surface, the evapotranspiration rate is a maximum; at a specified depth below land surface (extinction depth), the evapotranspiration rate is zero; in between, the evapotranspiration rate decreases linearly from the maximum to zero.

Because no spatial data were available for the type of vegetation or soil characteristics, a maximum evapotranspiration rate of 38 in/yr and an extinction depth of 15 ft were used uniformly for the entire model area. These are the same values used by Hardt and Hutchinson (1980), who used the same depth-dependent relation to simulate evapotranspiration. In the previous modeling study, however, the evapotranspiration relation was used to simulate both evapotranspiration and discharge of ground water into Warm Creek. As a result, the simulated evapotranspiration values reported by Hardt and Hutchinson (1980, table 3) are generally higher than those simulated by the revised ground-water flow model and reported in *table 15*.

Underflow.—Underflow occurs across several sections of the boundary of the ground-water flow model (*fig. 39*). In these sections, underflow is mostly through the unconsolidated deposits with a much lesser quantity through the sedimentary rocks of the badlands. Detailed analysis of the underflow across each section resulted in estimated annual values for 1945–98 (*table 9*); these values were used directly in the ground-water flow model. The only exception to this is underflow across the San Jacinto fault near Barrier J. A constant value of 2,000 acre-ft/yr ultimately was used for this section because the values in *table 9* produced a poor match between simulated hydraulic heads and measured ground-water levels in nearby wells. Additional description of underflow near Barrier J is in the section “Underflow” on page 45.

Underflow recharge and discharge were simulated in the ground-water flow model as annual specified fluxes using a modified version (UND1) of the standard well package (WEL1) by McDonald and Harbaugh (1988, p. 8–1). The only modifications to the WEL1 package were changes in variable names in order to prevent conflicts in referencing. Annual values of underflow for 1945–98 were summed as either recharge or discharge and are listed in *table 15*.

For simulation after 1998, or for any hypothetical simulation, underflow across the San Jacinto fault near the Santa Ana River used a minor modification to the head-dependent relation illustrated in *figure 28* and described in equation 5. Simulated head in the model cell containing the Heap well (1S/4W–3Q1) was used instead of measured ground-water level for that well. This change ensures that the simulated underflow responds appropriately to variations in recharge and discharge that are different from those that actually occurred during 1945–98. This head-dependent relation was simulated in the ground-water flow model using a modified version (GHB2) of the standard general head boundary package (GHB1) by McDonald and Harbaugh (1988, p. 8–1). The only modifications to the GHB1 package were changing the standard linear equation for calculating underflow to the non-linear equation

$$Q^{Underflow}_{SAR,\,k} = 96.876 \log (h_{Heap,\,k}) + 663.136 \qquad (10)$$

where

$Q^{Underflow}_{SAR,\,k}$ is underflow across the San Jacinto fault near the Santa Ana River for time period k, in acre-ft/yr; and

$h_{Heap,\,k}$ is simulated hydraulic head for the model cell containing the Heap well (row 70, column 79, layer 1) for time period k, in ft above mean sea level.

For use in the GHB2, the units in equation 10 were converted to ft and seconds. The value of $h_{Heap,\,k}$ was calculated for the same timestep as $Q^{Underflow}_{SAR,\,k}$. Although the value of $Q^{Underflow}_{SAR,\,k}$ changes more rapidly than it would if a linear head-dependent relation were used in the standard GHB1 package, no numerical problems were encountered in use of the modified GHB2 package.

Calibration

Calibration of the ground-water flow model involved simulating a historical period (calendar years 1945–98) and assuring that the simulated hydraulic heads, recharge, and discharge reasonably matched the measured and estimated data. Steady-state conditions were assumed and simulated for 1945. During this year, ground-water levels in most parts of the valley-fill aquifer remained virtually constant and ground-water recharge and discharge were approximately equal. Simulating steady-state conditions for 1945 also assured that the transient simulation of 1945–98 began with stable initial conditions, an important numerical consideration. During the transient simulation, recharge and discharge were simulated using annual values, and hydraulic head was calculated at the end of each year for each active model cell in each model layer.

Model solutions were obtained using the preconditioned conjugate-gradient solver (PCG2) by Hill (1990). One hundred timesteps were used for each 1-year stress period; a time-step multiplier of 1.2 was used to improve accuracy of the solution. Convergence criteria for both steady-state and transient conditions were 0.01 ft and 0.01 ft^3/s for head and flux, respectively. The residual mass balance error typically was less than or equal to 0.01 percent and always was less than 0.03 percent. During model development and use, some combinations of recharge and discharge resulted in convergence failure such as that described by Kuniansky and Danskin (2003). These failures typically occurred during simulation of a drought, but occasionally occurred from very small changes in recharge or discharge.

Calibration involved a trial-and-error adjustment of model parameters. Simulated hydraulic heads were compared to measured ground-water levels for about 100 wells in the San Bernardino area. Of equal importance, individual recharge and discharge components that are calculated by the ground-water flow model were compared to estimated and measured recharge and discharge. Model calculations of ground-water flow from one model cell to another (cell-by-cell fluxes, McDonald and Harbaugh, 1988, p. 3–19) were used to evaluate the spatial distribution of recharge and discharge.

Only a limited amount of calibration was necessary, probably because a previous ground-water flow model (Hardt and Hutchinson, 1980) had been calibrated for transient conditions in nearly the same area (*table 12*). Some model parameters, including components of recharge and discharge, were unchanged from initial estimates, or changed very little. A few model parameters, such as transmissivity and conductance across faults, were essentially the same as those used in the previous model by Hardt and Hutchinson (1980). The model parameters and the relative amount of adjustment during calibration are summarized in *table 17*.

The philosophy of model calibration, particularly when using a trial-and-error technique, plays a critical role in determining the final form of a numerical model. For example, the previous model by Hardt and Hutchinson (1980) was calibrated with the goal of matching ground-water levels near the former marshland; matching ground-water levels near the mountain front was of lesser importance (Hardt and Hutchinson, 1980, p. 47). During calibration of the present ground-water flow model, all simulated areas were weighted about equally, and calibration was guided most by the regional nature of the conceptual model. Matching ground-water levels at each individual well was desirable, but not mandatory.

Of greater importance was simulating each distinct recharge and discharge process and maintaining a relatively simple model with as few parameters as possible—essentially the philosophy of Albert Einstein who said, "A model should be as simple as possible, but no simpler." By following this philosophy, the effect of each model parameter on model results can be understood more readily. This philosophy prompted some parameters to be grouped together. For example, a single value of conductance was chosen for all narrow streams (*table 16*). This value was changed as necessary during calibration, for all narrow streams. Adjustment of individual parameters for individual model cells was done rarely, if ever.

Results of the calibration are shown in *figures 42, 43, 44, and 45*. *Figure 42* shows the match between measured ground-water levels and simulated hydraulic heads for 1945—the period used for the steady-state simulation, which, in turn, was used as initial conditions for the transient simulation of 1945–98. The match between steady-state levels and heads is reasonably good over most of the San Bernardino area and indicates that the model can be used to simulate regional ground-water flow in the Bunker Hill basin. Ground-water flow through the Lytle Creek basin is simulated less reliably, probably because of the numerous internal faults and ground-water barriers in that basin. Although the model does transport ground water through the Lytle Creek basin and calculates reasonable heads immediately downgradient of the basin, results of the model within the Lytle Creek basin itself should be used with caution.

The match between levels and heads during the transient simulation is shown in *figure 43* for selected wells scattered throughout the San Bernardino area. The wells were chosen to be areally distributed and to give an indication of the relative quality of the calibration. This relative quality (good, fair, or poor) was determined qualitatively for the 62 wells used in the model calibration and is shown on a map inset in *figure 43*. The most important criteria for the hydrograph match were having a symmetrical pattern of deflections, maintaining a uniform vertical offset, if any, and having a similar magnitude of multiple-year vertical deflections. The model appears to simulate ground-water levels best in the middle of the Bunker Hill basin. At some wells, the hydrograph match is remarkably good over the entire 54-year transient simulation and probably reflects accurate recharge and pumpage data more than a conscious effort at site-specific calibration. The quality of the match along the model boundaries is fair to poor, a characteristic common to many regional ground-water flow models.

Table 17. Source of parameter, recharge, and discharge estimates and their relative adjustment during calibration of the ground-water flow model of the San Bernardino area, California.

Model parameter, recharge, or discharge	Source	Relative adjustment during model calibration
Simulated area	Hardt and Hutchinson (1980).	Minor; model area extended to match bedrock boundaries.
Transmissivity	Hardt and Hutchinson (1980).	Minor; values required for new model area.
Storage	Eckis (1934).	Moderate; values from Eckis (1934) are generally lower than those used by Hardt and Hutchinson (1980).
Conductance (horizontal hydraulic conductivity) of faults	Hardt and Hutchinson (1980).	None.
Recharge from precipitation	Estimates based on values from Hardt and Hutchinson (1980) and Danskin (1998) and on an isohyetal map of long-term average precipitation (this study).	Minor; switched from uniform precipition to areally distributed values.
Streamflow	Gaged values of runoff.	Substantial re-evaluation of the quantity and timing of runoff; adjusted streambed conductance (vertical hydraulic conductivity) based on quantity of runoff.
Recharge from ungaged runoff	Estimated values using drainage areas and runoff quantities from Webb and Hanson (1972) and recharge rates from this study.	Minor adjustment to recharge rates.
Recharge of local runoff	Values from estimating individual components of annual precipitation (evaporation, direct recharge, local runoff, recharge from local runoff).	Minor; not included initially, but no adjustment after values were added to model.
Recharge of imported water	Quantity of imported water (Randy Van Gelder, San Bernardino Valley Municipal Water District, unpub. data, 1999) and estimated recharge rates (this study).	Minor adjustment to some recharge rates.
Pumpage	Western–San Bernardino Watermaster (S.E. Mains, Western Water District, and R.L. Reiter, San Bernardino Valley Municipal Water District, unpub. data, 2000).	Moderate; more complete pumpage data were obtained for individual wells for the period 1947–98. New data included verified extractions and production by non-filers. Annual pumpage is 0–15 percent greater than initial values, which were nearly identical to those used by Hardt and Hutchinson (1980).
Return flow	Hardt and Hutchinson (1980); and Western–San Bernardino Watermaster (R.L. Reiter, San Bernardino Valley Municipal Water District, unpub. data, 2000).	Return-flow percentage verified for all wells used for export. Net result is somewhat more export and less return flow.
Evapotranspiration	Hardt and Hutchinson (1980).	None.
Underflow	Multiple sources, including Dutcher and Garrett (1963), California Department of Water Resources (1971), and Dutcher and Fenzel (1972).	Moderate for most areas of underflow. Significant change to underflow beneath the Santa Ana River; many annual values are much less than those used by Hardt and Hutchinson (1980).

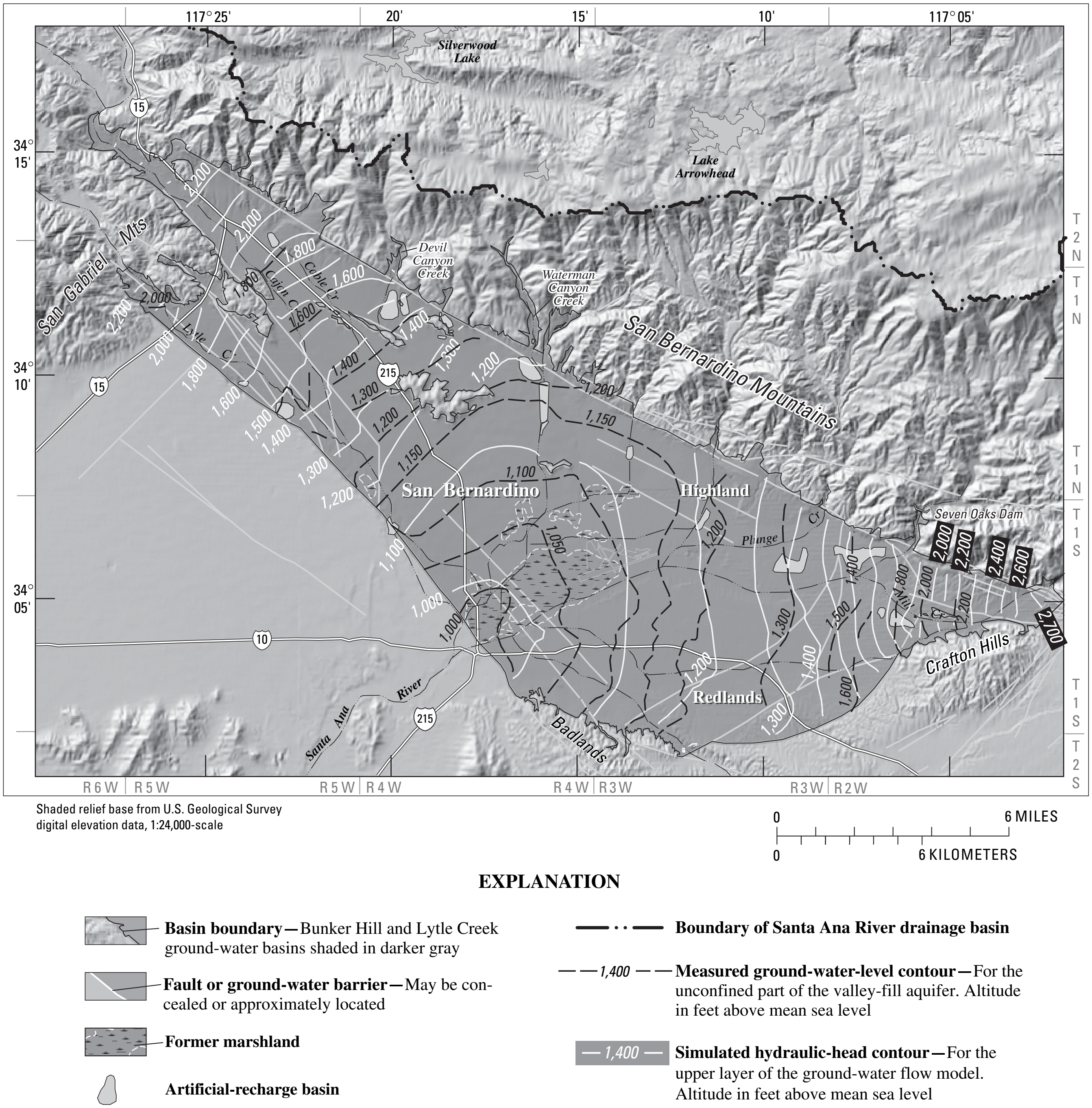
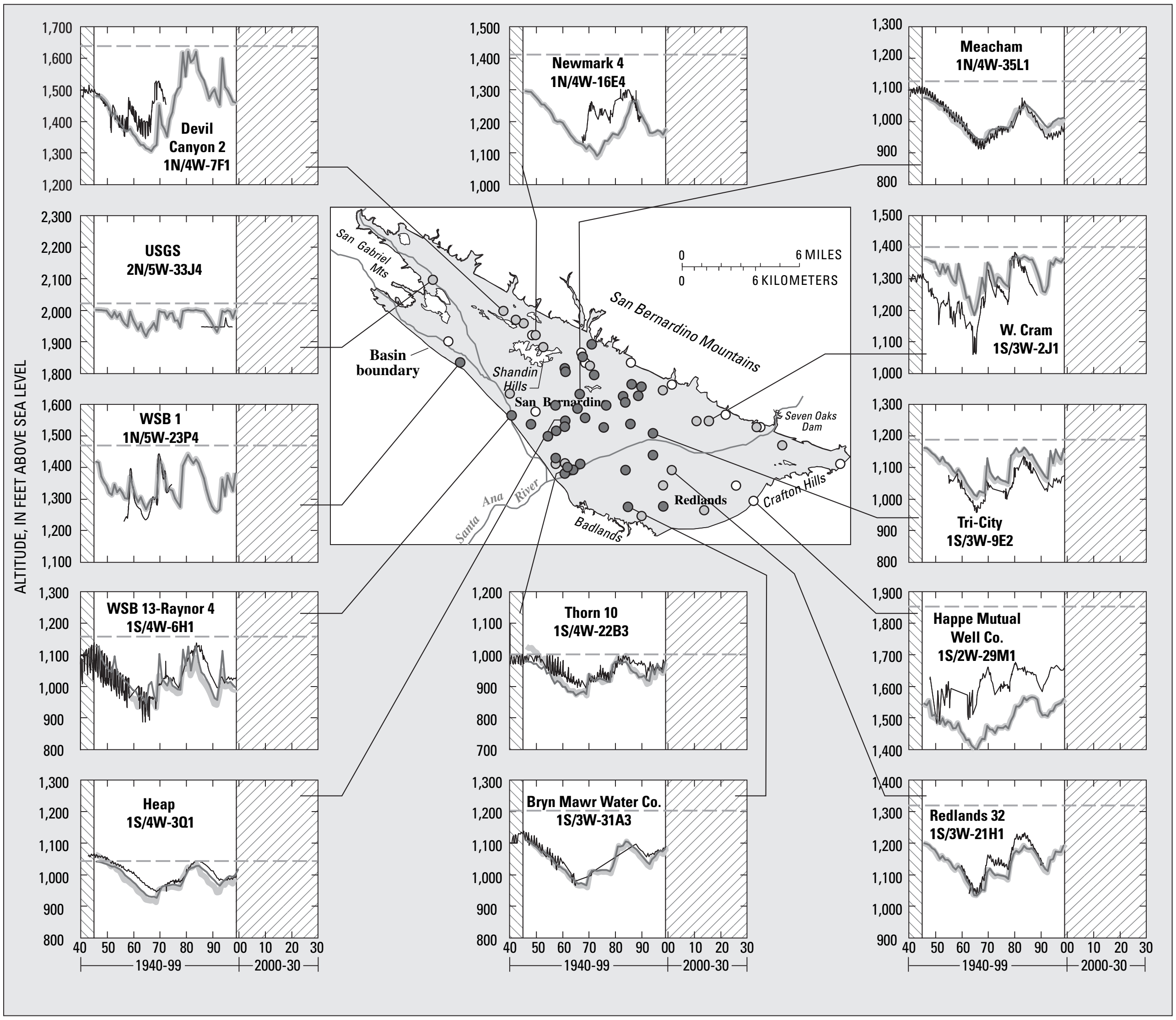


Figure 42. Measured ground-water levels and simulated hydraulic heads for the unconfined part of the valley-fill aquifer in the San Bernardino area, California, 1945. Measured contours adapted from unpublished map by F.B. Laverty (San Berndardino Valley Municipal Water District, written commun., 1998).



EXPLANATION

Graphs	Map
Simulated hydraulic head—For each layer of the ground-water flow model Upper layer Lower layer **Measured ground-water level** **Land surface** **Calibration period** 1940 1945-98 1999-2030	**Calibration wells**—Estimated quality of match between measured ground-water levels and simulated hydraulic heads indicated for each well used in calibration of the ground-water flow model Good Fair Poor

Figure 43. Results of calibration of the ground-water flow model of the San Bernardino area, California, 1945–98.

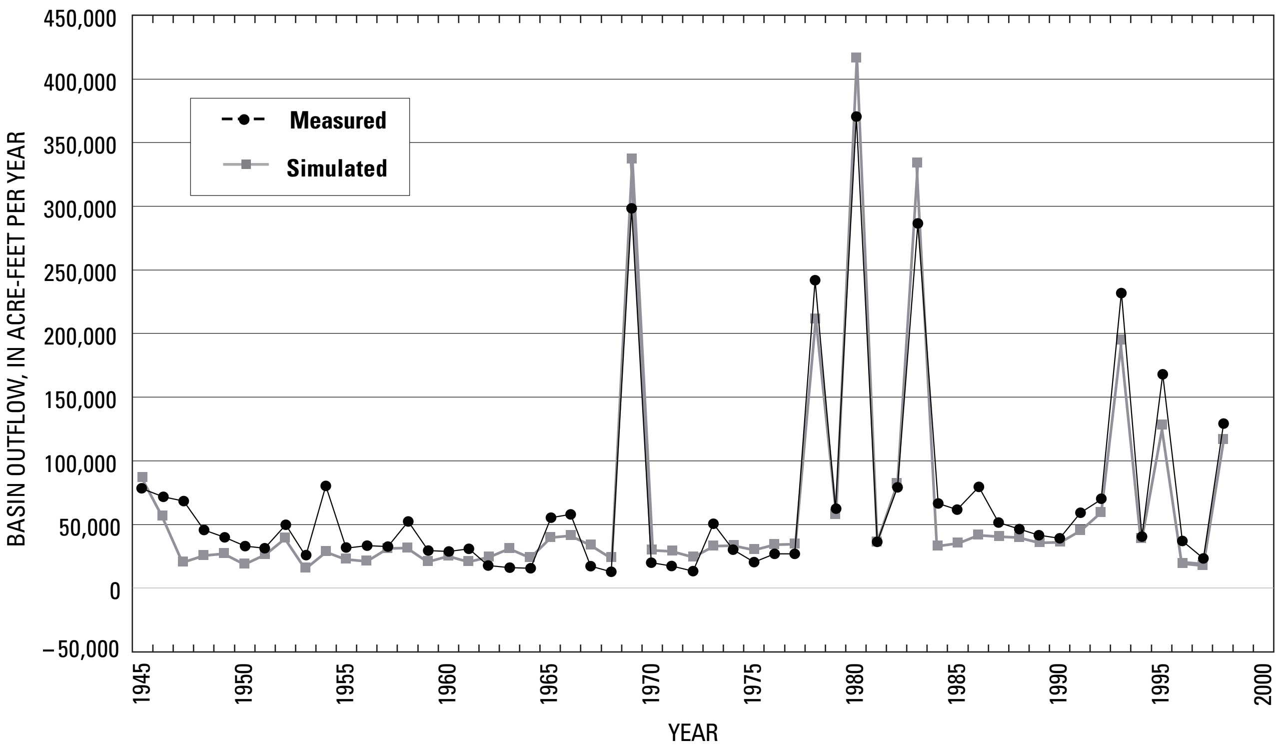


Figure 44. Comparison between measured and simulated surface-water outflow from the San Bernardino area, California, 1945–98.

A comparison between measured and simulated surface-water outflow from the basin is shown in *figure 44*. Outflow from the basin is measured at five gages (*table 1*; *figure 11*). Simulated outflow is a combination of outflow calculated by the ground-water model (STR2 package), plus local runoff (*table 4*), plus wastewater discharge (*table 7*). In general, the match between measured and simulated outflow is quite good—both as a long term trend and for most years. An important limitation of the ground-water flow model may be illustrated by *figure 44*. The model appears to be too responsive to some sequences of above-average runoff. In some wet years (1969, 1980, 1983), simulated outflow exceeds measured. Then following the wet period (1984–87), simulated outflow is less than measured outflow. This pattern suggests that water is routed through the simulated ground-water system too quickly. Perhaps via the STR2 package, surface water is not recharged in a sufficiently broad area so that it drains back to the stream system more slowly. A more complex surface-water routing package that incorporates a way to expand a simulated stream into adjacent model cells during periods of high runoff may help solve this issue.

A comparison of the change in ground-water storage for the transient calibration period 1945–98 is shown in *figure 45* using results from the ground-water flow model and from a similar storage calculation made with a GIS. The GIS method of calculation, which is described on p. 52, can yield a more accurate estimate of the actual change in ground-water storage because the method uses annually measured ground-water levels. Simulated heads calculated by the ground-water flow model are based on previously calculated heads and can accumulate errors over the simulation period. The primary errors associated with the GIS method are in selecting wells that reflect an unconfined change in ground-water levels. Both methods used the same values for specific yield. As shown in *figure 45*, the two sets of calculated values track surprisingly well over the 54-year period. This similarity tends to confirm the reasonableness of the ground-water flow model as well as of the observed wells chosen for the GIS calculation. The maximum change in ground-water storage during 1945–98 for the San Bernardino area was calculated by either method to be about 900,000 acre-ft; the annual change in ground-water storage commonly exceeded about 70,000 acre-ft.

Overall, as illustrated in *figures 42–45*, the linked surface-water/ground-water model performs well for the calibration period 1945–98. The model matches measured data and independent estimates for surface-water outflow, ground-water levels, and changes in ground-water storage. The model responds in realistic ways to a range of hydrologic conditions for an extended period of time. These features suggest that the model is well calibrated and can be used cautiously to investigate other hydrologic conditions.

Evaluation

In the development of some ground-water models, a historical period that is not part of the model calibration is used to critique the model. This type of evaluation, commonly called verification of the model, uses updated recharge and discharge values, but unchanged parameter values, to verify that the model can perform reasonably well for a different period of time with different stresses. Although this type of evaluation can be helpful in identifying problems with the model, the evaluation by itself does not confirm that the model is an accurate representation of the physical system.

In this study, no such evaluation or verification period was used, at least in the conventional sense. In some respects, the period 1975–98 can be viewed as an evaluation because the original model developed by Hardt and Hutchinson (1980) was calibrated for the preceeding period 1945–74. The key hydraulic parameters of transmissivity, vertical conductance, and conductance of faults were unchanged from the original model.

Inspection of results from the present model (*figs. 42–45*) suggests that the model performs reasonably well in both time periods, 1945–74 and 1975–98. The two periods are hydrologically fairly different. The first period has a remarkably long sequence of mostly dry years resulting in significant storage depletion (*figs. 7* and *18*). The second period has both a recovery of that storage depletion and a shorter wet-dry sequence. The second period also has many extremely wet years, which is uncharacteristic of the first period (*fig. 14*). Despite these hydrologic differences, the present model simulates both periods equally well. This capability suggests that the magnitude and distribution of hydraulic values, developed originally by Hardt and Hutchinson, are reasonable and allow the present model to simulate a variety of hydrologic conditions.

Sensitivity Analysis

A sensitivity analysis of a ground-water flow model involves observing the relative change in model output caused by a change in model inputs. Those inputs (aquifer characteristics, recharge, and discharge) that produce the greatest change in output (hydraulic heads and computed recharge, discharge, and cell-by-cell fluxes) are the most sensitive. An improvement in the most sensitive inputs will produce the greatest improvement in the ground-water flow model. This improvement may be an important goal to enhance the predictive capability of the model, but it does not necessarily mean that the simulation model becomes a more realistic representation of the system. The capability of a simulation model to represent a real system is more closely related to the compatibility among the real system, the conceptual model, and the simulation model (*fig. 38*).

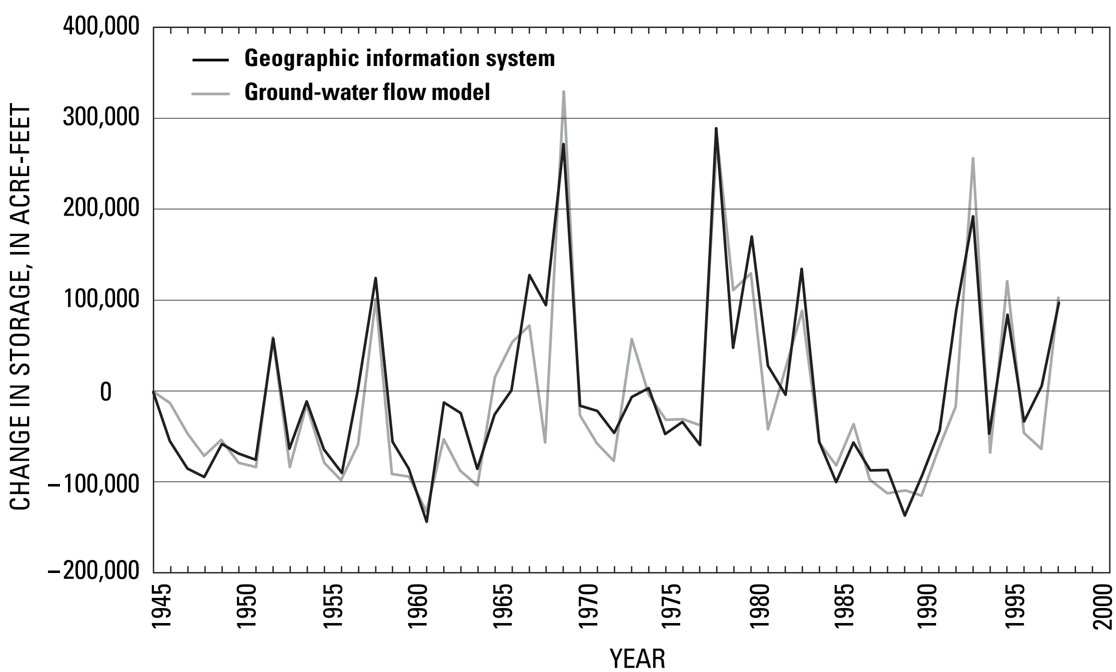


Figure 45. Estimated change in ground-water storage in the San Bernardino area, California, 1945–98. Methods of calculating values with a geographic information system and with the ground-water flow model are described on pages 52–53.

Results of the sensitivity analysis are summarized in *table 18*. The range of variation selected for each model value was based on the estimated uncertainty in the value. Attempts to test some values, such as a decrease in fault or streambed conductance, resulted in the model failing to numerically converge.

Recharge from streams and discharge from wells were found to exert the most influence on the simulated system as would be expected by their prominence in the water budget. Variations in the quantity or spatial distribution of these large inputs (*table 11*) create important changes in hydraulic heads and simulated recharge and discharge. Transmissivity and storage coefficient of the valley-fill aquifer are of lesser importance. Faults and ground-water barriers within the simulated area are critical in maintaining hydraulic head (water table) in areas outside the former marshland, particularly near Redlands. Without these barriers, the simulated water table in the outlying areas drops farther and faster.

The head-dependent relations used to approximate both evapotranspiration and the stream-aquifer interaction exert a controlling influence in the model, as demonstrated during calibration and sensitivity analysis of the model. These relations dampen fluctuations in hydraulic heads by adjusting the quantity of simulated recharge or discharge. The sensitivity analysis demonstrated that seemingly static hydraulic heads may mask substantial changes in ground-water flow rates, especially in the vicinity of the former marshland (*table 18*).

Increased streambed conductance during years with unusually large runoff was found to be critically important in providing sufficient recharge to match ground-water levels following 1965. Temporally constant values tested as part of the sensitivity analysis produced ground-water levels as much as 100 ft lower by the end of the 54-year simulation. Pumpage return flow is a significant component of the water budget, each year. Reducing return flow from 30 to 15 percent of gross pumpage at selected wells resulted in ground-water levels as much as 50 ft lower, even in the vicinity of the former marshland.

Use and Limitations

As designed and calibrated, the ground-water flow model can be used most appropriately for answering regional-scale questions about the valley-fill aquifer and for calculating boundary conditions for smaller-scale models. Development of the ground-water flow model focused mostly on general hydrogeologic themes found in the Bunker Hill and Lytle Creek basins, not on specific data from small areas encompassing a few hundred or a few thousand ft. Therefore, local-scale questions may not be answered well by using results from this model. The model, however, does integrate many different aspects of the valley-fill aquifer, and model results will reflect this complexity. The model will be particularly useful for issues involving the geometry, boundary conditions, storage, and water budget of the valley-fill aquifer. The model also will be useful in identifying regional directions and changes in direction of ground-water flow and differences in hydraulic head between upper and lower layers of the valley-fill aquifer. Regional issues of water use and distribution, in particular artificial recharge and ground-water extraction, can be analyzed with the aid of the model.

An important limitation of the ground-water flow model is that it is does not simulate the transport of chemical contaminants in the valley-fill aquifer; it is not a solute-transport model, nor should it be used as one. Rather, the ground-water flow model transports water from areas of recharge to areas of discharge through two highly generalized model layers (*fig. 37*). As of 1998, knowledge of the three-dimensional character of the valley-fill aquifer and human modifications to the aquifer is insufficient to develop a rigorous conceptual model of chemical transport in the aquifer, much less a numerical solute-transport model. For example, the specific flowpath of a chemical constituent through the valley-fill aquifer could be through a 10-ft-thick sand layer, along an erosional surface, on top of a fine-grained deposit, or through an abandoned well casing. Knowing which of these flowpaths is the real one may not be particularly important in simulating regional ground-water flow, but such knowledge is likely to be critical in understanding, tracking, and successfully mitigating a ground-water contaminant.

An area of caution in use of the ground-water flow model involves the upper confining member (UCM, *fig. 37*). This 50- to 100-ft thick, mostly fine-grained hydrogeologic unit is present in at least part of the former marshland (*figs. 2* and *24*). Within the ground-water flow model, UCM is simulated as a part of the upper model layer with a storage coefficient corresponding to a water-table condition. For most hydrologic conditions, this approximation works well, in particular for interaction with streams and for storage depletion lasting a few years. Anecdotal information and ground-water-quality data for UCM, however, suggest that UCM is a perchable unit that can remain saturated even as the rest of the upper model layer is dewatering (W.J. Hiltgen, San Bernardino Valley Water Conservation District, oral commun., 1988). Because the hydraulic complexity of UCM is only partly simulated with the ground-water flow model, some questions involving this zone may be addressed best by site-specific investigations or by more detailed ground-water flow models.

Table 18. Sensitivity of the ground-water flow model of the San Bernardino area, California.

Model parameter, recharge, or discharge	Amount or type of change	Sensitivity
	Model parameters	
Fault conductance	Increase conductance to remove any restriction to flow.	Areal ground-water levels are surprisingly similar to calibrated values except on the east side of the model near the city of Redlands.
Transmissivity	Increase by a factor of two.	Minimal change in ground-water levels over most of the Bunker Hill basin, except near the former marshland where levels rise an additional 10–20 feet.
Specific yield	Set all values to 0.10 (dimension less).	Minor change in hydrographs except along model boundaries where ground-water levels increase moderately.
Storage coefficient of lower model layer	Change from a constant value of 0.0001 to 0.001 (dimension less).	Minor change in hydrographs of lower model layer; no discernible change in hydrographs for upper model layer.
	Recharge	
Streambed conductance	Use a constant value for all years.	Recharge is insufficient; ground-water levels fail to rise sufficiently during 1970–80.
Recharge from ungaged runoff	Decrease by a factor of two.	Most hydrographs are similar; slight decrease in evapotranspiration.
Recharge from local runoff	Set to zero.	Minimal change in hydrographs, but insufficient gains to streams during wettest periods.
Pumpage return flow	Decrease return flow from 30 to 15 percent.	Minimal change in hydrographs, but significantly less evapotranspiration and gains to the streams.
Underflow across the Crafton fault	Decease by a factor of two.	Ground-water levels on the east side of the Bunker Hill basin fall dramatically and do not recover.
	Discharge	
Evapotranspiration rate	Decrease maximum rate from 38 to 18 inches per year.	Total evapotranspiration decreased by about half. Ground-water levels rise too high, but increased gains to the streams compensate for much of the decrease in evapotranspiration.
Pumpage	Increase pumpage by 10 percent.	Ground-water levels decline too far and fail to recover sufficiently following the 1960’s.
Underflow beneath the Santa Ana River	Use previous rate of 15,000 acre-feet per year.	Ground-water levels decline too low near the former marshland and fail to recover sufficiently during 1980’s; evapotranspiration is too low.

Changes in saturated thickness in the upper layer of the valley-fill aquifer (UCM and UWB, *fig. 24*) were assumed to cause relatively localized changes in transmissivity, mostly near the base of the mountains. The ground-water flow model uses a time-invariant transmissivity, which causes simulated hydraulic heads near the mountain front to fluctuate somewhat less than if the model used a time-varying transmissivity. Calculation of transmissivity as a function of saturated thickness likely would improve the simulation, but would require data about the three-dimensional configuration of the valley-fill aquifer that were not available. A time-varying transmissivity also creates a hydraulic nonlinearity that significantly increases the complexity of the constrained optimization model and would require as much as 10 to 200 times as much computational time to solve a typical water-management problem for the San Bernardino area (Danskin and Gorelick, 1985).

The streamflow-routing package (STR2) simulates the average interaction between a stream and the valley-fill aquifer during a whole year. This is a considerable simplification of an actual stream in a semiarid environment. Discharge in real streams is much more variable and commonly occurs over a period of days or months, not an entire year. Nevertheless, the streamflow-routing package does add important capabilities and potential uses of the ground-water flow model. First, the package facilitates development of linked water budgets between the surface-water and ground-water systems. Second, the package is likely to be useful in evaluating changes in streamflow and ground-water recharge and discharge caused by changes in streambed conductance. For example, the package could be used to identify the likely result of installing a concrete liner in a stream channel or increasing the size of an artificial-recharge basin. Future modifications to the ground-water flow model may benefit from including more of the actual complexity of the surface-water system, in particular adding diversions from major streams (*pl. 2*).

Additional future modifications to the ground-water flow model could include simulation of historical land subsidence (Miller and Singer, 1971). Although the quantity of inelastically released ground water from the area of the former marshland is not likely to be significant, simulating historical land subsidence may provide additional insight about the hydrogeologic setting and structure of the valley-fill aquifer.

Modification of pumpage return flow may provide an important improvement to the flow model. Presently, return flow is calculated using a constant percentage of pumpage per well, and is returned to the same location as the pumpage. Ideally, return flow could be calculated using an understanding of water distribution, land use (fig. 8), and climatic factors, such as potential evapotranspiration.

A time-consuming, but probably beneficial modification to the ground-water flow model would be to use shorter stress periods, either quarters or months, in order to simulate seasonal variability in streamflow, pumpage, and ground-water levels. Adding model layers or decreasing model grid size are unlikely to significantly improve the flow model until more hydrogeologic information is known about the three-dimensional structure of the valley-fill aquifer.

Constrained Optimization Model

The large number of complex, interrelated water-management issues in the San Bernardino area requires a quantitative evaluation of water-management alternatives. Some of this evaluation can be done with field data, and some can be done with the ground-water flow model described in this report. Much of the evaluation, however, involves answering water-management questions that are too complex to be analyzed using only these methods.

Constrained optimization techniques, as a subset of the field of operations research (H.M. Wagner, 1975; Hillier and Lieberman, 1980; Winston, 1987), were developed to analyze precisely this type of situation. Use of the techniques involves development of a *constrained optimization model*, which is a set of equations that defines a management problem. The equations mathematically describe management objectives and constraints, and the mathematical solution of the equations (optimization model) identifies the most efficient allocation of a scarce resource, such as water or money. Multiple issues can be considered simultaneously, and alternate solutions can be compared quantitatively.

Since the initial research and development in the 1940's, constrained optimization techniques have been applied to many scientific and business problems, such as finding the most efficient design of a nationwide telephone system, the greatest revenue from mixing multiple-grade iron ores, the fewest number of planes to service an area, and the cheapest arrangement of irregularly shaped patterns on a bolt of cloth. Application of the techniques to hydrologic problems has been limited, although many research studies have been conducted (Gorelick, 1983; Rogers and Fiering, 1986; B.J. Wagner, 1995).

Overtime, however, the use of optimization techniques to solve applied ground-water management problems has increased as the techniques have been taught more widely, as ground-water flow models have been developed for hundreds of basins, and as computer codes and computational power have improved. Recent examples of optimization techniques applied in various hydrogeologic settings include Reichard (1995), Nishikawa (1998), Barlow and Dickerman (2001), Czarnecki and others (2003a,b), Danskin and others (2003), Eggleston (2003), Phillips and others (2003), Reichard and others (2003), Bexfield and others (2004), and McKee and others (2004).

The optimization model documented in this report is based on well-developed techniques (Gorelick and others, 1993; Ahlfeld and Mulligan, 2000) and extends previous work by integrating surface-water, ground-water, and water-quality issues in an actual field area, and by working closely with the more than 20 water managers in the San Bernardino area who are actively seeking solutions to complex water-management questions.

General Characteristics

The particular constrained optimization techniques used to evaluate water-management alternatives in the San Bernardino area are referred to as linear and quadratic programming. Within the optimization model, the management objective is expressed as a linear or quadratic equation (*objective function*). Other equations in the model represent management *constraints* that must be met. The mathematical form of the optimization model is to maximize or minimize

the objective equation (11a)

subject to

all the constraint equations. (11b)

Items to be managed, such as recharge or pumpage at specific sites, are represented by *decision variables* in the equations. When combined with a physical model, such as a ground-water flow model, the optimization model may include *state variables*, such as heads, that indicate the state of the physical system. In applying optimization techniques to ground-water problems, state variables commonly are included in the optimization model as decision variables. Other variables and parameters can be added to the optimization model to link the decision variables to each other or to perform necessary calculations, such as determining total recharge, lift at a well, or cost of operations.

The use of constrained optimization techniques to solve a hydrologic problem is best illustrated by a simple example. In 1990, two production wells—the Ninth Street well (1S/4W–4E8) and the Perris Street well (1S/4W–4F4)—were installed to provide municipal water via the Baseline feeder pipeline (*fig. 11*) (Geoscience Support Services, Inc., 1990). Both wells were installed to a depth of about 1,000 ft below land surface and were designed to extract ground water mostly from the MCM and MWB hydrogeologic units (*fig. 24*). Initial testing of ground water pumped from the wells indicated that water from well 1S/4W–4E8 had an elevated temperature (> 80° F) and a fluoride concentration of about 1.8 mg/L, which exceeded the safe drinking-water standard defined by the State of California (California Department of Water Resources,1995a). Although this unusual temperature and ground-water quality helped to identify the likely location of the Banning (?) fault (*fig. 5*), the elevated values created a problem in supplying water of acceptable quality for municipal use. To solve the water-quality problem, ground water pumped from the two wells has been blended prior to distribution.

This water-management problem of trying to maximize production while providing acceptable water quality and not exceeding capacity of the wells can be solved by formulating it as a simple optimization model. For example, if a water-management goal is to maximize the production of water from two wells, then an objective function (Z) can be written as

$$\text{maximize } Z = Q^{Pump}_1 + Q^{Pump}_2\,, \tag{12}$$

where the value of Z is to be maximized and the decision variables Q^{Pump}_1 and Q^{Pump}_2 represent pumpage, in ft^3/s, at wells 1 and 2, respectively. In this example, well 1 is the Ninth Street well that has an unacceptably high concentration of fluoride, and well 2 is the Perris Street well that has less fluoride.

Constraints to the production of water involve the maximum capacity of wells and the necessary blending of water to achieve an acceptable quality. The constraints, which are formulated as inequalities or sometimes equalities, restrict the possible values of the decision variables (Q^{Pump}_1, Q^{Pump}_2), which in turn restrict the possible value of Z. In this example, Z is subject to

$$Q^{Pump}_1 \le 6.2 \text{ ft}^3/\text{s} \tag{13a}$$

$$Q^{Pump}_2 \le 5.5 \text{ ft}^3/\text{s} \tag{13b}$$

$$Q^{Pump}_1 + Q^{Pump}_2 = Q^{Pump}_{Total} \tag{13c}$$

$$c_1\, Q^{Pump}_1 + c_2\, Q^{Pump}_2 \le (1.4 \text{ mg/L})\,(Q^{Pump}_{Total}) \tag{13d}$$

where c_1, c_2 are concentrations of fluoride, in mg/L, of water pumped from wells 1 and 2, respectively. For this problem, values of c_1 and c_2 are 1.8 and 0.6, respectively. Q^{Pump}_{Total} is total pumpage from wells 1 and 2.

Equations 13a and 13b are capacity constraints that restrict the pumpage from wells 1 and 2 to physically realistic values. Equation 13c is a mass-balance constraint that helps formulate equation 13d. Equation 13d is a water-quality constraint that assures blended water pumped from wells 1 and 2 does not exceed the water-quality standard for fluoride of 1.4 mg/L. Water temperature for municipal use is not regulated, but will be controlled reasonably well if the fluoride standard is met. As in this example, values on the right-hand side (*RHS*) of the equations are often capacities or target values.

Mathematical solution of the optimization model requires that the unknown values (decision variables) be written on the left-hand side of the equation and that the known values (constants) be written on the right-hand side. By reorganizing terms, constraint equations 13a–d become,

$$Q^{Pump}_{1} \leq 6.2 \text{ ft}^3/\text{s} \quad (14a)$$

$$Q^{Pump}_{2} \leq 5.5 \text{ ft}^3/\text{s} \quad (14b)$$

$$Q^{Pump}_{1} + Q^{Pump}_{2} - Q^{Pump}_{Total} = 0 \quad (14c)$$

$$c_1 Q^{Pump}_{1} + c_2 Q^{Pump}_{2} - (1.4 \text{ mg/L}) (Q^{Pump}_{Total}) \leq 0 \,. \quad (14d)$$

Non-negativity constraints,

$$Q^{Pump}_{1}, Q^{Pump}_{2}, Q^{Pump}_{Total} \geq 0 \quad (14e)$$

usually are included, sometimes implicitly by the optimization computer program, to make solving the problem mathematically more efficient. Equation 14e also is a constraint on the physical operation of the wells, ensuring that they are used only for extraction of water, not injection. Together, equations 12 and 14 constitute the optimization model. Equation 12 is the objective function; equations 14a–e are the constraints.

The simple optimization problem described by equations 12 and 14 is a linear-programming problem that can be solved by hand or graphically, as shown in *figure 46A*. A graphical solution is possible when an optimization problem has three or fewer independent decision variables, each represented by the x, y, or z axis. The example problem with two decision variables (Q^{Pump}_{1} and Q^{Pump}_{2}) is represented by a two-dimensional graph (*fig. 46A*). The variable Q^{Pump}_{Total} is a linear combination of the two decision variables and, therefore, does not require an additional dimension for the solution. Constraint equations define regions of the graph where a solution is either *feasible* or *infeasible*. The *optimal solution* for a linear-programming problem is always found at a corner or along an edge of the *feasible region*; other points within the feasible region will satisfy all constraints, but will not be optimal. In the example problem, the optimal value of Z is 11.7 ft^3/s. Equations 14a and 14b are *binding constraints* that determine the value of Z. Equation 14d is a non-binding or *loose constraint* that can be eliminated from the optimization model without altering the value of Z. If the value of c_1 were greater, such as 5.1 mg/L, which is the concentration measured in the lowest part of the well, then the feasible solution space would be much smaller as shown in figure 46*B*. Equation 14d revised with the larger concentration values then becomes a binding constraint, equation 14a becomes a loose constraint, and the value of Z decreases to 6.9 ft^3/s.

Optimization models used to solve real problems typically require tens or hundreds of decision variables and hundreds or thousands of constraints. In order to more easily describe an optimization model, particularly a large or complex model, a tableau format can be helpful. Such a tableau for the simple optimization model described above (eqs. 12 and 14) is shown in *figure 47*.

If the objective of the example problem were to minimize pumping costs, then the optimization model would be an example of a quadratic-programming problem. Pumping cost is a function of pumpage and lift; lift is also a function of pumpage; therefore, pumping cost is proportional to pumpage squared. Assuming a single time period of constant pumpage, equation 11a can be rewritten for well 1 and well 2 as

$$\text{minimize } Z = c^e e_1 Q^{Pump}_{1} L_1^{Total} + c^e e_2 Q^{Pump}_{2} L_2^{Total} \,, \quad (15a)$$

where

- c^e is the unit cost of electricity, in dollars per foot of lift per cubic foot of water;
- e_i is the efficiency of the well pump and motor for well i, expressed as a decimal fraction, commonly about 0.60;
- L^{Total}_{i} is the total lift at well i, in ft;
- Q^{Pump}_{i} is pumpage from well i, in cubic ft; and
- Z is total pumping cost, in dollars.

Total lift at a well is composed of an initial fixed lift and a variable lift that is proportional to additional pumpage. Substituting these terms, the objective becomes,

$$\text{minimize } Z = c^e e_1 Q^{Pump}_{1} (L^{Initial}_{1} + L^S_1) + c^e e_2 Q^{Pump}_{2} (L^{Initial}_{2} + L^S_2) \,, \quad (15b)$$

where

- $L^{Initial}_{i}$ is initial lift at well i, in ft; and
- L^S_i is additional lift at well i caused by pumpage (stress) at all other wells, in ft.

The additional lift can be calculated using an aquifer-response relation derived from a specific-capacity test, an aquifer test, or a ground-water flow model. For an aquifer where drawdown is a linear function of pumpage, the objective equation can be expanded to

$$\text{minimize } Z = c^e e_1 Q^{Pump}_{1} (L_1^{Initial} + r_{1,1} Q^{Pump}_{1} + r_{1,2} Q^{Pump}_{2}) + c^e e_2 Q^{Pump}_{2} (L_2^{Initial} + r_{2,1} Q^{Pump}_{1} + r_{2,2} Q^{Pump}_{2}) \,, \quad (15c)$$

where

- $r_{o,s}$ is the unit drawdown response observed at location o that results from a unit pumping stress applied at location s, in ft per cubic ft per second.

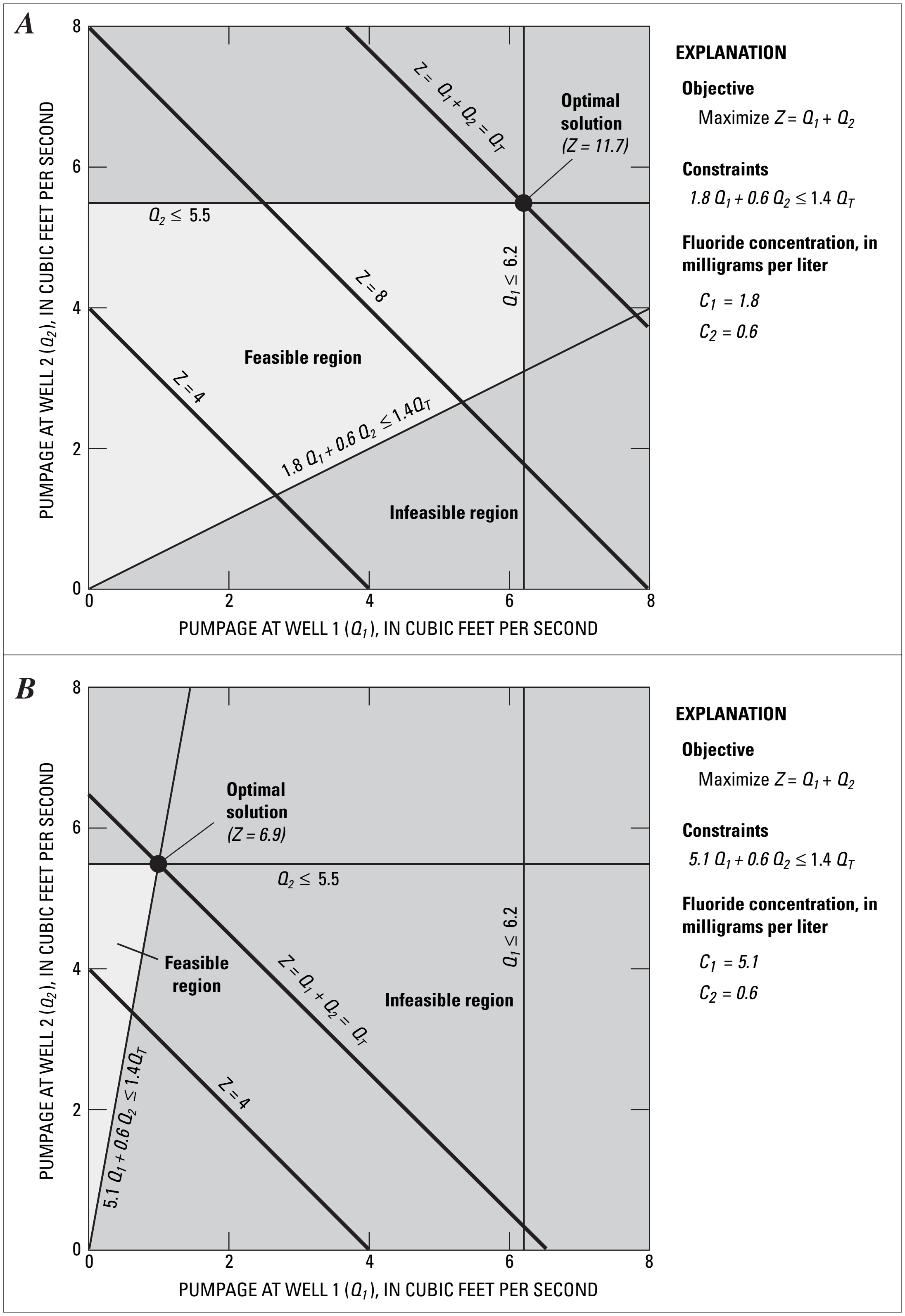


Figure 46. Graphical solution of a simple constrained optimization model. Graphs *A* and *B* show optimal solutions for fluoride concentrations in well 1 of 1.8 and 5.1 milligrams per liter, respectively.

This formulation of an aquifer-response relation is appropriate for a single management period, such as a steady-state analysis. The next section of this report, "Representing the ground-water flow system," includes a more in-depth discussion of how to include response of an aquifer in an optimization model.

Additional constraints may be desired to restrict drawdown at each well, for example,

$$L_1^{Total} \leq \text{maximum allowable drawdown at well 1, in ft;} \tag{16a}$$

and

$$L_2^{Total} \leq \text{maximum allowable drawdown at well 2, in ft.} \tag{16b}$$

The revised optimization problem (model) uses a quadratic objective function (eq. 15c) with linear constraints (eq. 14a–e and 16a,b).

A specialized computer program is required to solve essentially all realistic optimization models. The program identifies the mathematically optimal value of Z and the optimal values for all variables and equations in the optimization model. Additional information provided by the computer program includes the marginal value of the decision variables ($\delta Z/\delta Q_1$, $\delta Z/\delta Q_2$) and an identification of which constraint equations restrict the value of Z (binding constraints) and which are superfluous constraint equations (loose constraints). Effective use of the optimization model involves not only obtaining the value of the objective function (Z), but also analyzing the additional information. This analysis enables the user to better understand the relative importance of different variables and equations, and in some cases, to identify previously unknown relations between the equations. For management of a complex physical system, gaining this understanding is at least as important as obtaining the optimal value of Z (H.M. Wagner, 1975, p. 942–943).

The constrained optimization model developed to evaluate water-management alternatives for the San Bernardino area uses either linear or quadratic programming. Objective equations are linear or quadratic; all constraint equations are linear. Decision variables represent recharge, pumpage, and costs; and objective equations include different combinations of these decision variables. Constraint equations include both decision variables and state variables representing ground-water levels at selected locations in the valley-fill aquifer. Use of the optimization model involves combining a single-objective equation with appropriate constraint equations in order to answer a specific water-management question. Much of the power of constrained optimization techniques is derived from the ease of reformulating a model by using a different objective equation with a different set of constraint equations. Technically, each combination of an objective and set of constraints is itself an optimization model. For purposes of simplicity in this report, however, all objectives and constraints for water management in the San Bernardino area are referred to collectively as the constrained optimization model of the San Bernardino area. Different combinations of equations can be used to analyze a specific water-management problem, such as scenarios 5 and 6 that are described later in this report. Scenarios 1–4 were analyzed using only the ground-water flow model, not any optimization techniques.

Objective function and constraints		Decision variables			Equality condition	RHS (Right-Hand Side of equations)
		Pumpage				
		Q_1	Q_2	Q_T		
Objective function (z)		1	1			
Constraints	Capacity	1			<=	6.2
	Capacity		1		<=	5.5
	Mass balance	1	1	−1	=	0
	Water quality	1.8	0.6	−1.4	<=	0
	Non negativity	1			>=	0
	Non negativity		1		>=	0
	Non negativity			1	>=	0

Units for pumpage are cubic feet per second and for fluoride concentration used in the water-quality constraint are milligrams per liter. The water-management problem solved by the optimization model is described in equations 12 and 14 (a-e) and is shown in figure 46.

Figure 47. Objective function, constraint equations, decision variables, and RHS of a simple constrained optimization model.

Setting up and solving the optimization model of the San Bernardino area involves a number of steps, listed in *figure 48*. At the outset, the major water-management issues need to be identified. Objectives and constraint equations are written in commonly used words, and then the word equations are transformed into algebraic equations. A computer software program or set of programs is chosen to solve the equations and write out the optimization results. Finally, the optimal results are obtained and reviewed with the local water managers to ensure the results are credible and useful.

For the San Bernardino optimization model, two sets of computer programs were used. The first set used the MODMAN (Greenwald, 1993) program to run the MODFLOW ground-water flow model and automatically create an MPS input dataset, which was then solved using the optimization software MINOS (Murtagh and Saunders, 1983). Although the MPS format is an industry standard for optimization models, it is exceptionally cumbersome to create manually, hence the need for MODMAN. MINOS can be used to solve a quadratic optimization problem, but it requires additional FORTRAN subroutines written for the specific problem. LINDO software (LINDO Systems Inc., 1994) was used occasionally instead of MINOS because LINDO has a better interface with MODMAN. In particular, MODMAN can automatically retrieve optimization results from LINDO and rerun the ground-water flow model using the optimal values of decision variables. Re-running the ground-water flow model is important to quantify the difference between the optimal solution, which is based on an assumption that the ground-water flow model is strictly linear, and the simulated results from the San Bernardino ground-water flow model, which has mildly nonlinear features, such as evapotranspiration. Although less convenient for re-running the ground-water flow model, MINOS was found to be more cost effective for solving large optimization problems. This set of programs (MODFLOW, MODMAN, and MINOS) was used to solve scenarios 5 and 6 described later in this report. LINDO was used during development of scenario 5 to efficiently assess hydraulic non-linearity.

The second set of computer programs combines use of the MODFLOW ground-water flow model with the General Algebraic Modeling System (GAMS) developed by the World Bank (Brooke and others, 1988). The MODFLOW program was used to calculate the response of simulated hydraulic head to recharge and discharge. These data were entered manually into the input file for GAMS, which then required having both definitions and documentation of the optimization model in a single computer input file. The GAMS format is highly flexible and uses extensive free formatting of text and values. Solving a quadratic problem using GAMS involves simply writing the objective equation in words. GAMS software then solves the optimization problem internally by using one of several optimization solvers, including MINOS. This set of programs (MODFLOW and GAMS) was used during initial development of the ground-water flow and optimization models. The high degree of flexibility and customization offered by GAMS was helpful during the formative steps of identifying decision variables, interacting with decision makers, and defining the likely feasibility space of scenario 6.

Representation of the Ground-Water Flow System

Quantitative evaluation of water-management alternatives in the San Bernardino area requires a method of predicting the response of ground-water levels to different quantities of recharge and pumpage. The ground-water flow model is an excellent way to calculate this response because the flow model simulates the physical process and incorporates much of the complexity of the aquifer system. The calculated response can be incorporated in the optimization model using the technique of response functions (Maddock, 1972; Gorelick, 1983; Gorelick and others, 1993, p. 145–153; Ahlfeld and Mulligan, 2000).

A *response function* is the simulated response of hydraulic head to recharge or discharge. The recharge or discharge can be defined for actual or hypothetical conditions, and the effect on head can be observed at any location in the flow model and after any length of time during the simulation. The response function is in effect an encapsulated form of the ground-water flow model and includes the combined effects of geometry, boundary conditions, aquifer parameters, and other model fluxes. Response functions are an effective and commonly used method of linking a ground-water flow model to a constrained optimization model (Gorelick, 1983; Lefkoff and Gorelick, 1987; Greenwald, 1993). The other method of combining a ground-water flow model with an optimization model is referred to as *embedding*. This technique requires that all equations defining the ground-water flow model be included as constraints in the optimization model (Gorelick, 1983). This approach, however, becomes computationally too large for nearly all realistic problems.

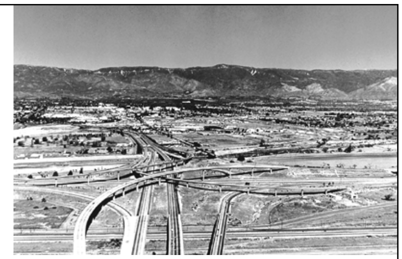
Development of the constrained optimization model for the San Bernardino area required creating a *unit response function* for each of the decision variables representing recharge to, or discharge from the valley-fill aquifer. Because a calibrated ground-water flow model was available, it was used to calculate the unit response functions—a representative set of which is shown in *figure 49*. Each graph is created by simulating a *unit stress* of recharge or discharge and observing the *unit response* in simulated hydraulic head at various *control locations* throughout the valley-fill aquifer over a 32-year *management horizon*. The unit stress is chosen to be sufficiently high so that a numerically significant unit response is produced at all locations of interest by the end of the 32-year period.

Identify and meet decision makers

Identify the physically-based and political entities that make or significantly affect water-management decisions.

Meet with progessively larger groups of these entities.

Consider forming a technical advisory committee.



Define the water-management problem

Define general water-management issues.

Define water-management items that can be controlled (decision variables).

Write objectives and constraints in commonly used words.

Define the water-management time frame (management horizon).

Formulate the problem mathematically

Write objectives and constraints as mathematical equations.

Formulate an optimization model to solve a specific problem.

Choose computer codes

Determine method of calculating response of the ground-water flow system.

Select constrained optimization software.

Review formulation with water-management decision makers.

Solve the optimization problem

Solve the constrained optimization model using selected software and, if necessary, a ground-water flow model.

Review optimization results with decision makers.

Reformulate the optimization problem, if necessary.

Develop water-management monitoring system

Define and plan to collect additional hydrologic data needed to confirm critical water-management assumptions and conclusions.

Monitor key decision variables and constraints to determine ongoing success of the water-management plan.

Implement water-management plan

Figure 48. Sequence of steps involved in design and use of a constrained optimization model to solve a ground-water management problem.

Figures 49A–C each show the unit response at several observation wells to a simulated unit stress at one recharge basin. *Figure 49D* shows the unit response at one observation well (SBVMWD) to a simulated unit stress at several recharge basins. The curve labeled “Lytle” on *figure 49D* is the same curve labeled “SBVMWD” on *figure 49A*, and the curve labeled “Sweetwater” on *figure 49D* is the same curve labeled “SBVMWD” on *figure 49B*.

For a management horizon with a single stress period, such as a steady-state simulation, the response function is calculated by applying a unit stress for the entire period and observing the unit response in head at the end of the period. The unit stress can be applied at a single model cell or apportioned among a group of model cells. The unit response is observed at a single model cell and is the difference in simulated head with and without the unit stress. Therefore, to calculate a unit response, at least two model runs are required: one with no managed stresses and one for each managed stress that requires a unit response.

If the management horizon has multiple stress periods of identical length, such as the 32 1-year periods used in the optimization model of the San Bernardino area, use of unit response functions can take advantage of the linearity of the ground-water flow system. The unit response is calculated by simulating a unit value of stress for the first stress period and zero values for all subsequent stress periods. This input is essentially a unit pulse with the width of a single stress period. The response of head to this pulse is observed for all stress periods. The principle of superposition, which is the key assumption in use of response functions, allows the unit response function (pulse) to be shifted from one time period to another in order to simulate stresses that begin at any time within the management horizon, and superposition allows the individual responses to be summed at the end of each stress period as shown in *figure 50* (Reilly and others, 1987). The two upper diagrams in *figure 50* show response functions for a unit stress applied in year 1. The two lower diagrams show the response function for a unit stress applied in years 3 and 5, which is simply the response function for year 1 shifted forward in time by 2 or 4 years.

The convention of positive and negative signs in the creation and use of response functions in ground-water optimization studies varies greatly and can cause significant problems in either formulation or interpretation of the optimization model. Pumpage in some studies is positive and in other studies negative. Increasing head in some studies is positive and in other studies negative. The sign convention used in this study is that recharge is positive, which causes a positive change in head and a positive response function; pumpage is negative, which causes a negative change in head and a negative response function.

The first step in creating the response-function part of the optimization model is to simulate the response of the aquifer to initial and boundary conditions and to all unmanaged stresses. This procedure involves eliminating all decision variables from the ground-water flow model or setting them at prescribed values. The simulation model then is used to calculate hydraulic head ($h^{Ibc}_{o,\,to}$) at each control (observation) location of interest (o) at the end of each stress period (to) over the entire management horizon. Subtracting the initial head at the beginning of the simulation ($h^{Initial}_{o}$) from initial and boundary condition head ($h^{Ibc}_{o,\,to}$) yields the *initial and boundary condition response* ($r^{Ibc}_{o,\,to}$); that is,

$$r^{Ibc}_{o,\,to} = h^{Ibc}_{o,\,to} - h^{Initial}_{o}\ . \tag{17}$$

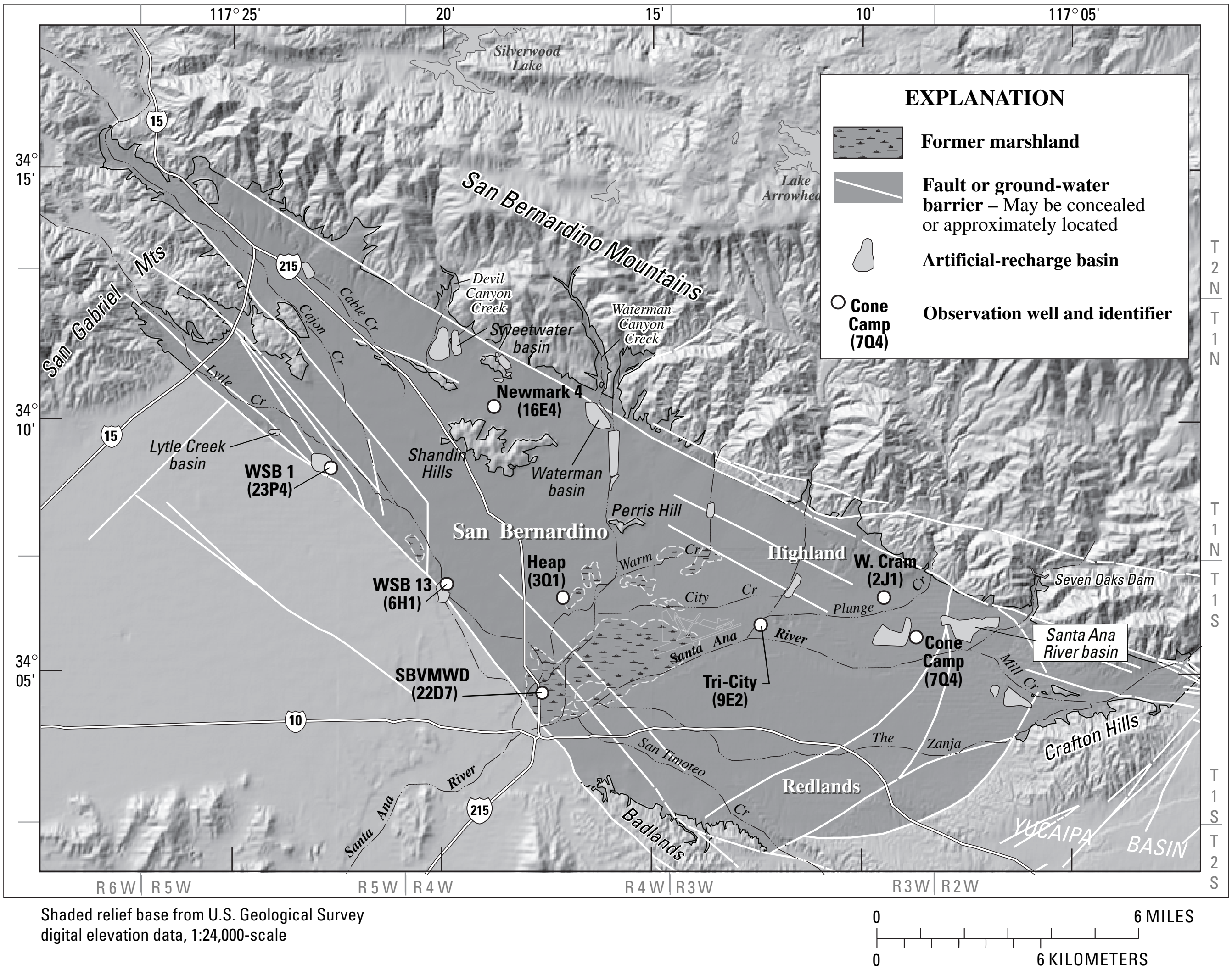
This change in head represents the net effect of everything in the model, except the decision variables. In some formulations, an estimated or average value for some managed stresses is included as part of the unmanaged simulation in order to minimize the effect of hydraulic nonlinearities. In this case, the $r^{Ibc}_{o,\,to}$ term also includes the effect of these background values, and the optimization model is used to define deviations from the background values. This technique is described in Danskin and Freckleton (1992) and Greenwald (1993).

The next step is to create a unit response function ($r_{o,\,to,\,s,\,ta}$) for each managed stress. The basic idea is to use a unit pulse of recharge or discharge to cause changes in head throughout the area of interest and over the time frame of interest. This cause (unit pulse) and effect (change in head) is the response function. The technique typically involves application of a unit stress for a single decision variable (s) in the first stress period of the simulation (ta = 1). All other recharge, discharge, and boundary conditions are the same as those used during simulation of the initial and boundary condition response. The head ($h_{o,\,to,\,s,\,ta}$) resulting from the unit stress is observed at all locations of interest (o) at the end of the first and all subsequent stress periods (to = 1, 2, 3 ...). Subtracting this head from the initial and boundary condition head yields the unit response function ($r_{o,\,to,\,s,\,ta}$) for that decision variable, at all locations of interest and at all times of interest. That is,

$$r_{o,\,to,\,s,\,ta} = h^{Ibc}_{o,\,to} - h_{o,\,to,\,s,\,ta}\ . \tag{18}$$

This procedure is repeated for each recharge or discharge component of the simulation model that is to be a managed part of the optimization model.

Once created, the unit response can be used to represent a unit stress applied during any single stress period throughout the management horizon. The response in head is simply shifted uniformly in time along with the applied unit stress. This procedure is illustrated in the lower graphs of *figure 50*. The capability of using a unit response for different time periods results from the linearity of the ground-water flow model and is a characteristic of superposition. The more nonlinear the flow model is, the more care is needed in creation and use of unit response functions. To minimize the effect of nonlinearities it may be necessary to create response functions for specific time periods and to use a different value of the unit stress.



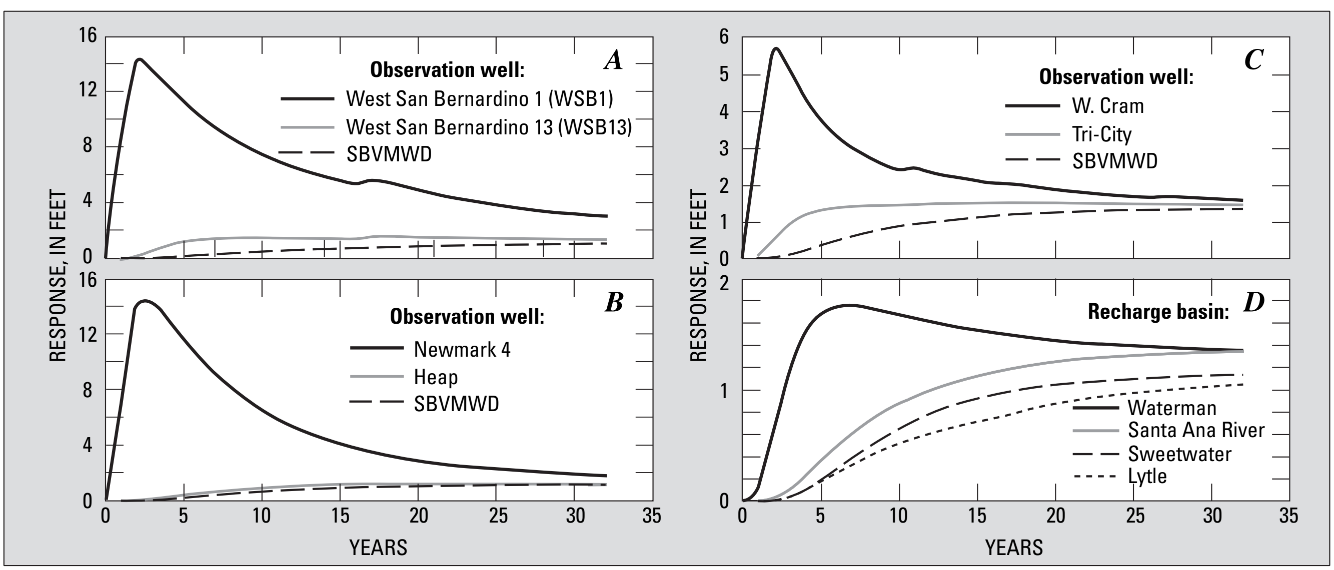


Figure 49. Representative response of simulated hydraulic head to recharge in the San Bernardino area, California. Recharge simulated at *A*, Lytle Creek basin; *B*, Sweetwater basin; and *C*, Santa Ana River (upper) basin. Responses observed at selected observation wells at increasing distance from the recharge basin. Graph *D* shows responses at San Bernardino Valley Municipal Water District (SBVMWD). All responses are for the upper model layer and result from 10,000 acre-feet of recharge in the first year. Ground-water model area shown in dark gray.

In many cases, the value of the unit stress can be chosen to be the same magnitude as the value of the decision variable determined later by the optimization model. The unit stress, however, needs to be sufficiently large that the accuracy of the unit response is essentially unaffected by convergence and roundoff errors that occur during solution of the flow model. Also, the unit stress needs to be sufficiently large that an accurate response is generated in all locations of interest, as soon as is needed. If the unit stress is too small, the response will not reach all areas of the model or will not reach them soon enough to be numerically significant. If these conditions are met, then the unit response can be included as a reliable part of the optimization model. Finally, use of round numbers for the unit stress (1 ft^3/s, 100 acre-ft/yr) tends to minimize errors in formulating the optimization model.

The development of response functions allows total drawdown (d) to be calculated from unit responses, values of decision variables, and the initial and boundary condition response. For example,

$$d_{o,\,to} = (r^{Ibc}_{o,\,to}) + [\text{SUM } s{=}1,\ ndv\ [\text{SUM } ta{=}1,\ ntp\ (r_{o,\,to,\,s,\,ta})\ (q_{s,\,ta})]]\ , \qquad (18)$$

where

d is total drawdown from managed and unmanaged stresses and from initial and boundary conditions, in ft;

r^{Ibc} is the response of simulated hydraulic head from initial and boundary conditions and from all unmanaged stresses, in ft;

r is the unit response of simulated hydraulic head, in ft per ft^3/s; and

q is a managed stress, represented in the ground-water flow model by recharge or discharge for a model cell or group of model cells, in ft^3/s;

and indices

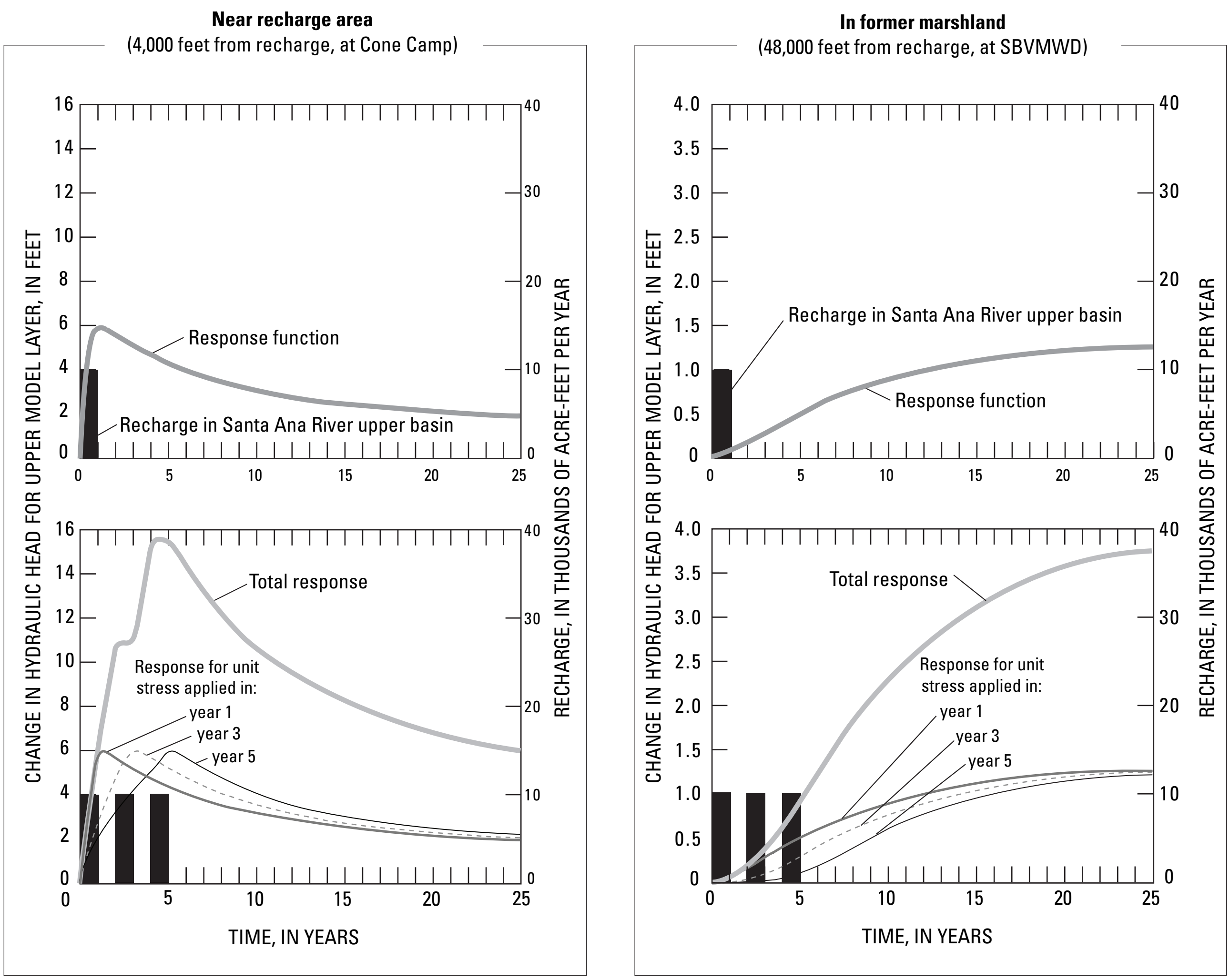


Figure 50. Superposition of response functions. Note different scale of graphs. Location of recharge (Santa Ana River upper basin) and water-level response (Cone Camp and SBVMWD) shown in figure 49.

o is an index indicating the location where simulated hydraulic head is observed, represented in the ground-water flow model by a single model cell;

to is an index indicating the time period when the response is observed, calculated in the ground-water flow model at the end of a stress period;

s is an index indicating a specific managed stress;

ndv is the number of decision variables (managed stresses) having response functions;

ta is an index indicating the time period when the managed stress is applied, represented in the ground-water flow model by a stress period; and

ntp is the number of time periods simulated in the management horizon.

This relation is sometimes expressed in compact matrix notation as,

$$D = R^{Ibc} + R\,Q\,, \qquad (19)$$

where

D is a matrix of total drawdown at all control locations, at all time periods, in ft;

R^{Ibc} is a matrix of responses, at all control locations, at all time periods, resulting from initial and boundary conditions and from all unmanaged stresses, in ft;

R is a matrix of unit responses, at all control locations, at all time periods, from all managed stresses, applied at all time periods, in ft per ft^3/s; and

Q is a matrix of all managed stresses, applied at all time periods, in ft^3/s.

In some formulations, it is advantageous to calculate simulated hydraulic head for use in the optimization model. Using results from equation 18,

$$h^{Man}_{o,\,to} = h^{Initial}_{o,\,to} + d_{o,\,to} \qquad (20)$$

where

h^{Man} is managed hydraulic head, in ft;

$h^{Initial}$ is initial hydraulic head, in ft; and

d is total drawdown, in ft, caused by managed and unmanaged stresses and by initial and boundary conditions;

or in matrix notation,

$$H^{Man} = H^{Initial} + R^{Ibc} + R\,Q \qquad (21)$$

where

H^{Man} is a matrix of managed hydraulic heads, in ft; and

$H^{Initial}$ is a matrix of initial hydraulic heads, in ft.

Because response functions are an encapsulated form of the ground-water flow model, they can be instructive by themselves prior to being used to develop the optimization model. This additional benefit was demonstrated during the present study of the San Bernardino area. Many of the key hydraulic principles of managing recharge were identified when the response functions were developed for each of the artificial-recharge basins. Unit responses were calculated using a 1-year pulse of 10,000 acre-ft/yr during the first year, and zero flux in subsequent years. This relatively large unit stress was needed to create a sufficiently large response in heads throughout the simulated area as shown in *figures 49* and *50*. The large unit stress does create a spike in simulated heads near the artificial-recharge basin, but fails to create much change in head in the vicinity of the former marshland.

Of particular interest is that although the recharge occurs only in the first year, the maximum change in simulated head does not necessarily occur in the first year. In fact, the response to recharge from the artificial-recharge basin adjacent to the Santa Ana River is not fully felt in the vicinity of the former marshland (SBVMWD, *fig. 49C*) for more than 30 years. This time lag results from the shape of the Bunker Hill and Lytle Creek basins, from the large area of high storage (specific yield), and from the great distance between the artificial-recharge basin and the control location. The time lag also illustrates the difficulty in intuitively answering complex water-management questions involving many decision variables and control locations.

An equally important effect shown in *figure 49D* is how recharge in different artificial-recharge basins creates substantially different short-term effects on simulated heads in the vicinity of the former marshland. For example, the effect of recharge in the Waterman basin peaks in year seven. In contrast, the effect from recharge in the Santa Ana River basin has not peaked even after 30 years. Despite short-term differences, the longterm effect of recharge is similar for all basins. These results suggest that different recharge basins are ideally suited for managing either short-term or longterm ground-water levels in the former marshland. Response of simulated heads near the site of recharge is similar for all basins, probably because of similar aquifer characteristics near each basin. Analysis of the unit response functions such as those shown in *figure 49* demonstrates the benefit that can be gained from setting up the optimization model, even before an optimal solution is found.

The major limitation in using response functions in linear or quadratic programming is that the response must be linear with respect to the applied stress. This in turn requires that the ground-water flow model must be linear. Over the past two decades, however, ground-water flow models have become increasingly nonlinear. As computational capabilities have increased, additional nonlinearities have been added. Optional packages, such as those listed in *table 14*, frequently are included with the standard computer code so that the aquifer system can be simulated more accurately. Some of these options include: (1) use of piecewise-linear recharge and discharge relations (streams, evapotranspiration), (2) recalculation of transmissivity and storage coefficients based on simulated hydraulic heads, and (3) dewatering and rewetting of model cells (McDonald and Harbaugh, 1988).

Methods have been developed to approximate typical nonlinearities such as those induced by stream recharge (Danskin and Gorelick, 1985), time-varying transmissivity (Danskin and Gorelick, 1985; Willis and Finney, 1985), and evapotranspiration (Danskin and Freckleton, 1992; Greenwald, 1993). These methods, however, can require significant computational capabilities and time, and some methods require use of another optimization technique (mixed-integer programming). For highly nonlinear ground-water problems, such as solute transport, nonlinear optimization techniques are required. Fortunately, many common ground-water management problems can be analyzed, at least initially, through the use of linear programming.

Design and development of the ground-water flow model for the San Bernardino area required achieving a balance between meeting the requirement of linearity and providing an accurate simulation of the valley-fill aquifer. Despite the restrictions of linear programming, it was necessary to include three nonlinearities in the ground-water flow model—streamflow routing, evapotranspiration, and underflow across the San Jacinto fault near the Santa Ana River. To determine how much these model nonlinearities alter the optimal solution, results of the optimization model were re-simulated with the full ground-water flow model and compared. A small disparity is to be expected because of the nonlinearities. A small additional disparity is caused by numerical approximations in the ground-water flow model; this difference is generally less than the number of decision variables times the closure criteria of the flow model solution and can be controlled sufficiently well by reducing the closure criteria. This technique was used by Yeh (1990) to critique optimization results from a similar, mildly nonlinear ground-water flow model, and the technique is included as an option in the optimization computer code by Greenwald (1993).

For water-management problems investigated as part of this present study, the head difference caused by hydraulic nonlinearities was acceptably small, and the linear-programming solutions were sufficiently accurate. The presence of a large disparity between the solutions, however, could occur for other optimization problems in the San Bernardino area and would indicate the need for another approach, such as the iterative techniques used by Danskin and Freckleton (1992) or Greenwald (1993).

Mathematical Form of Objectives and Constraints

Water-management objectives and constraints are expressed in the constrained optimization model as mathematical equations. The objective function (equation) defines a specific objective that is to be maximized or minimized subject to a set of constraint equations. To answer a slightly different water-management question, a slightly different objective function can be combined with the same or nearly the same set of constraint equations. Commonly, reformulation of the optimization model to answer a related water-management question requires only that a specific constraint equation be used as the objective function, and the former objective function be included as a constraint. This capability of slight, but powerful modifications makes optimization techniques in general and this model in particular, an efficient way to investigate related water-management questions. A slightly different formulation of the optimization model not only provides additional insight about overall water management, but also can be used to represent the specific viewpoint of a different water-management entity. The following objectives and constraints summarize the major water-management issues in the San Bernardino area as of 1998. These issues are described in detail in a subsequent section of this report entitled “Water-Management Issues.”

Formulation of the objective function and constraints for the optimization model uses the same discretization of time as the ground-water flow model, which is used to calculate response functions. The shortest time period representing uniform stress in the flow model is a calendar year; therefore, the optimization model also uses calendar years. Use of a shorter time period for the optimization model is possible, but not recommended unless the ground-water flow model is revised and recalibrated using the shorter time period. The management horizon for the optimization model is defined as any number of sequential stress periods, or in this case calendar years. For the water-management scenarios described later in this report, a horizon of 32 years was used. The following formulation of objectives and constraints is sufficiently general so that a different management horizon could be used with only minor modifications to the objectives and constraints.

Objective Function

The objective function (equation) includes decision variables representing recharge of imported water, ground-water pumpage, and the cost of each of these. Other recharge and discharge components of the ground-water flow model are included in the optimization model as part of the right-hand side (RHS). Although these ground-water fluxes are not managed explicitly, they are an implicit part of the optimization model. Use of the RHS to account for unmanaged (background) stresses is described on page 91 of this report and in greater detail by Danskin and Freckleton (1992) and Gorelick and others (1993).

Imported water can be distributed via the Foothill pipeline to several artificial-recharge basins in the San Bernardino area (*fig. 11*). If a water-management goal is to limit use of this resource, then the objective function is to

$$\text{Minimize } Z = \text{sum } k=1, ny \text{ [sum } iar=1, nar\ (Q^{ArtRech}_{iar,k})] , \qquad (22a)$$

where

- iar is the index for a specific artificial-recharge basin;
- k is the index for a specific year;
- nar is the total number of artificial-recharge basins;
- ny is the total number of time periods (years) in the water-management period; and
- $Q^{ArtRech}_{iar,k}$ is the quantity of imported water distributed to artificial-recharge basin iar, during time period k, in acre-ft.

Distributing imported water to an additional, proposed artificial-recharge basin can be analyzed by including the proposed basin in equation 22a.

The actual quantity of water recharging the valley-fill aquifer is slightly less than the quantity of water distributed to the artificial-recharge basin. During the calibration period of the ground-water flow model (1945–98), the recharge rate of imported water was assumed to be 90 percent. The remaining 10 percent of the water was assumed to be lost to evapotranspiration. If the water-management goal is to maximize the quantity of imported water that is recharged to the valley-fill aquifer, then a coefficient reflecting the efficiency of recharge for each basin could be included in equation 22a. For example,

$$\text{Maximize } Z = \text{sum } k=1, ny \text{ [sum } iar=1, nar\ (rr_{iar,k}\ Q^{ArtRech}_{iar,k})], \qquad (22b)$$

where

- $rr_{iar,k}$ is the recharge rate for artificial-recharge basin iar during time period k, expressed as a decimal fraction.

If minimizing the cost of supplying imported water for recharge is the primary water-management goal, then the objective function becomes

$$\text{Minimize } Z = \text{sum } k=1, ny \text{ [sum } iar=1, nar\ (C^{ImpWater}_{iar,k}\ Q^{ArtRech}_{iar,k})] , \qquad (22c)$$

where

- $C^{ImpWater}_{iar,k}$ is the cost of imported water distributed to basin iar, during time period k, in dollars per acre-foot.

The value of $C^{ImpWater}_{iar,k}$ is determined by agreement with the State Water Project that supplies the imported water (California Department of Water Resources, 1995b). This cost, however, can vary depending on whether water was banked with the State Water Project and the source and timing of electricity used to import the water. For example, peak use of electricity can be two to three times more expensive than off-peak use. In 1998, the minimum value of $C^{ImpWater}$ was about $100 per acre-ft to convey water from San Francisco Bay to the afterbay of the Devil Canyon powerplant. A small additional cost is needed to pump this water to different artificial-recharge basins within the San Bernardino area; therefore, $C^{ImpWater}$ is slightly different for each basin. Equation 22c does not assure that imported water is distributed to the artificial-recharge basins with the highest recharge rates. Achieving this additional water-management goal will depend on the specific set of constraints involving the valley-fill aquifer.

As of 1998, several new production wells were under construction in the San Bernardino area. In addition, several wells have been proposed to provide additional water for municipal use, to restrict the transport of ground-water contaminants, or to lower ground-water levels near the former marshland. The effect of these wells can be included in the optimization model either individually or as defined sets of wells. The following formulation minimizes the total quantity of ground-water pumpage over the management horizon. The formulation considers individual wells, each having a fixed ratio of extraction from the upper and lower layers of the ground-water flow model. Specifically,

$$\text{Minimize } Z = \text{sum } k=1, ny \text{ [sum } iw=1, nw\ (Q^{Pump}_{iw,k})] , \qquad (23)$$

where

- iw is the index for a specific well;
- k is the index for a specific time period;
- nw is the total number of wells; and
- $Q^{Pump}_{iw,k}$ is the quantity of pumpage from well iw during time period k, in acre-ft.

Both existing and proposed wells can be included in equation 23 as long as the extraction ratio from the two model layers is known or can be defined. The extraction ratio for all wells with pumpage during 1945–98 was defined during calibration of the ground-water flow model.

If it is important to analyze the effect of different ratios of extraction from the two model layers, then the extraction from a single well could be separated into two discharge quantities. Each discharge would be simulated in the respective layer of the ground-water flow model, and the two discharge quantities would be linked in the optimization model with a mass-balance constraint, as illustrated in equation 13c on p. 85. This more detailed formulation allows the effect of different perforated intervals for the well to be critiqued with the optimization model.

If the water-management goal is to minimize the cost of pumpage, then the objective becomes a quadratic equation, such as described on p. 86 of this report. For a single time period, the equation is

$$\text{Minimize } Z = c^{e}\, e_{iw}\, Q^{Pump}_{iw}\, L^{Total}_{iw}\,, \qquad (24a)$$

where

- c^{e} is the cost of electricity, in dollars per foot of lift per acre-foot;
- e_{iw} is the efficiency of well iw, expressed as a decimal fraction;
- L^{Total}_{iw} is total lift at well iw, in ft; and
- Q^{Pump}_{iw} is the quantity of pumpage from well iw, in acre-ft.

Expanding total lift into separately calculated components yields

$$\text{Minimize } Z = c^{e}\, e_{iw}\, Q^{Pump}_{iw}\, (L^{Initial}_{iw} + L^{Ibc}_{iw} + L^{Man}_{iw}) \qquad (24b)$$

where

- L^{Ibc}_{iw} is lift at well iw resulting from initial and boundary conditions and from unmanaged stresses, in ft;
- $L^{Initial}_{iw}$ is initial lift at well iw, in ft; and
- L^{Man}_{iw} is total lift at well iw resulting from all managed stresses, in ft.

For a single stress period, equation 24b becomes

$$\text{Minimize } Z = \text{sum iw=1, nw } [c_{e}\, e_{iw}\, (Q^{Pump}_{iw})[L^{Initial}_{iw} + L^{Ibc}_{iw} + \text{sum s=1, ns } (Q^{Pump}_{iw}\, r_{iw,\,s})]\,, \qquad (24c)$$

where

- ns is the total number of stresses; and
- $r_{iw,\,s}$ is the response of hydraulic head at location iw resulting from pumpage at location s, in ft.

For multiple stress periods, the formulation becomes more cumbersome because pumpage in one time period can create additional lift in a later time period, as is indicated by the response functions (*fig. 50*). Therefore, equation 24c becomes

$$\text{Minimize } Z = \text{sum k=1, ny [sum iw=1, nw } [c_{e}\, e_{iw,\,k}\, (Q^{Pump}_{iw,\,k})[L^{Initial}_{iw} + L^{Ibc}_{iw,\,k} + \text{sum s=1, ns [sum ta=1, ny } (Q^{Pump}_{iw,\,k}\, r_{iw,\,k,\,s,\,ta})]]\,, \quad \text{for ta} \le \text{k}, \qquad (24d)$$

where

$r_{iw,\,k,\,s,\,ta}$ is the response of hydraulic head at location iw during time period k resulting from pumpage at location s during time period ta, in ft.

Although commonly used for water-management evaluations, equations 24d is a significant simplification of the actual operation of a production well and, therefore, needs to be used with caution. Well efficiency varies with discharge; response of lift to pumpage is slightly nonlinear; and pump capacities and bowl settings typically apply for a limited range of lift. If a change in lift requires either moving the pump or resizing the bowls, then a significant cost is incurred that is not included in equation 24d.

Many water-management goals involve both recharge and pumpage. These goals can involve maintaining an adequate quantity of ground water stored in the valley-fill aquifer or simply minimizing the magnitude of any change in present water-management operations. If the optimization model does not include both recharge and pumpage in the objective function, then it is possible that an optimal solution that minimizes recharge (eq. 22a) would require an unnecessarily high pumpage, or vice versa. To correct this possible limitation, the total quantity of recharge and pumpage can be minimized by

$$\text{Minimize } Z = \text{sum k=1, ny [sum iar=1, nar } (Q^{ArtRech}_{iar,\,k}) + \text{sum i=1, nw } (Q^{Pump}_{iw,\,k})]\,, \qquad (25a)$$

which results from combining equations 22a and 23.

If the objective is to minimize the total cost of recharge and pumpage, then the objective becomes

$$\text{Minimize } Z = \text{sum k=1, ny [sum iar=1, nar } [C^{ImpWater}_{iar,\,k}\, Q^{ArtRech}_{iar,\,k}] + \text{sum iw=1, nw } [c^{e}\, e_{iw,\,k}\, Q^{Pump}_{iw,\,k}\, (L^{Initial}_{iw} + L^{Ibc}_{iw,\,k} + \text{sum s=1, ns [sum ta=1, ny } (Q^{Pump}_{iw,\,k}\, r_{iw,\,k,\,s,\,ta})])]\,, \qquad (25b)$$

for ta <= k, which results from combining equations 22c and 24d. The method for calculating pumping cost for a well is described on p. 86 and in equations 15a–c.

Constraints

Water Supply and Water Distribution

The mathematically optimal value of the decision variables is constrained by various equations representing water-supply and water-distribution constraints. These constraints assure that adequate water is supplied through the present distribution system and that the quantities determined by the optimization model are physically possible.

For example, artificial recharge in each basin must be less than or equal to the maximum recharge capacity of that basin, and the sum of artificial recharge in all basins must be less than or equal to the total quantity of water that is available from the State Water Project. During all years and even more so during droughts, the maximum quantity of imported water is less than 100 percent of the entitlement value of 102,600 acre-ft/yr (*table 5*). Expressed as a decimal fraction, the percentage of entitlement ($P^{ImpWater}_{k}$) ranges from 0.10 to 0.60, depending on the particular year (k). These two water-supply constraints, applied for each time period k, are,

$rr_{iar,k}\ Q^{ArtRech}_{iar,k}$ ≤ maximum recharge capacity for each basin iar (26a)

and

sum iar=1, nar $[Q^{ArtRech}_{iar,k}] \le (P^{ImpWater}_{k})$ (102,600 acre-ft/yr). (26b)

Water supply to each artificial-recharge basin also must be less than the capacity of the conveyance structures connecting the California Aqueduct to the basin. Specifically, for each time period k,

$Q^{ArtRech}_{iar,k}$ ≤ maximum turnout capacity for basin iar (26c)

and

sum iar=1, nar $[Q^{ArtRech}_{iar,k}]$ ≤ maximum capacity of the Foothill pipeline. (26d)

Similarly, maximum pumpage from each individual site is restricted by well, pump, and aquifer characteristics, and total pumpage from all sites is restricted by a maximum value derived from legal adjudication or from an evaluation by local water managers based on distribution capabilities or on anticipated demand. These constraints are expressed for each time period (k) as

$Q^{Pump}_{iw,k}$ ≤ maximum historical or estimated production at each site iw, (27a)

and

sum iw=1, nw $(Q^{Pump}_{iw,k})]$ ≤ maximum annual pumpage in the area. (27b)

Or to restrict cumulative production over a period of years,

sum k=1, ny [sum iw=1, nw $(Q^{Pump}_{iw,k})]$ ≤ maximum cumulative pumpage in the area. (27c)

Ground-Water Level

Management of recharge and pumpage in the San Bernardino area is constrained by requirements on ground-water levels. In the vicinity of the former marshland, ground-water levels need to be sufficiently low to prevent possible liquefaction and sufficiently high to prevent additional land subsidence. In the alluvial fan areas, ground-water levels need to be maintained sufficiently high to assure a continuous supply of ground water to nearby wells.

These water-management constraints are represented in the optimization model in the following general way for each observation water-level control location (o) at each time period (k),

$H^{Initial}_{o,k}$ + [sum iar=1, nar [sum ta=1, ny $(r_{o,k,iar,ta}\ Q^{ArtRech}_{iar,ta})]$ + [sum iw=1, nw [sum ta=1, ny $(r_{o,k,iw,ta}\ Q^{Pump}_{iw,ta})]] \le H^{LimitUp}_{o,k}$ (28a)

or

$H^{Initial}_{o,k}$ + [sum iar=1, nar [sum ta =1, ny $(r_{o,k,iar,ta}\ Q^{ArtRech}_{iar,ta})]$ + [sum iw=1, nw [sum ta=1, ny $(r_{o,k,iw,ta}\ Q^{Pump}_{iw,ta})]] \ge H^{LimitLo}_{o,k}$ (28b)

for ta <= k, where

$H^{LimitUp}_{o,k}$ and $H^{LimitLo}_{o,k}$ are the upper and lower limits, respectively, of hydraulic head at location o during time period k.

Because the response function calculates the induced drawdown from managed stresses, it is sometimes more convenient to represent the ground-water-level constraint as a drawdown constraint. For example,

sum iar=1, nar [sum ta=1, ny $(r_{o,k,iar,ta}\ Q^{ArtRech}_{iar,ta})]$ +
sum iw=1, nw [sum ta=1, ny $(r_{o,k,iw,ta}\ Q^{Pump}_{iw,ta})]$
$\le D^{LimitUp}_{o,k}$ (29a)

or

sum iar=1, nar [sum ta =1, ny $(r_{o,k,iar,ta}\ Q^{ArtRech}_{iar,ta})]$ +
sum iw=1, nw [sum ta =1, ny $(r_{o,k,iw,ta}\ Q^{Pump}_{iw,ta})]$
$\ge D^{LimitLo}_{o,k}$ (29b)

for ta <= k, where

$D^{LimitUp}_{o,k}$ $D^{LimitLo}_{o,k}$ are the upper and lower limits, respectively, of drawdown at location o during time period k.

The total number of constraint equations increases rapidly with the dimensions of iar, iw, o, and k. For example, a problem with 5 recharge sites, 15 well sites, 50 observation locations, and 32 time periods can require more than 5,000 constraint equations. Although optimization techniques are designed to address large problems, an optimization model with more than several thousand constraint equations can be cumbersome to work with, and the results can be time-consuming to interpret. Computational time typically is not a major concern in solving the optimization model itself, but computational time may become a major impediment if a ground-water flow model simulation is needed to calculate response functions for each of many decision variables, such as iar and iw.

Ground-Water Quality

Ground-water-quality issues can be included in the optimization model through the use of hydraulic gradients computed by the ground-water flow model. The technique, as described by Lefkoff and Gorelick (1986), involves adding a hydraulic-gradient constraint in the optimization model to control movement of a selected water-quality constituent at an identified location in the aquifer. The flow model, through response functions, is used to calculate the change in gradient caused by the decision variables. The basic idea is to define a hydraulic gradient across the edge of a contaminated area, then to constrain this gradient so that the contamination does not spread further through the basin. The role of the optimization model is to control the spread of the contamination while simultaneously considering other water-management issues in the basin.

Formulation of the hydraulic-gradient constraint involves selecting a control point inside and outside the contaminated area. Typically, points are chosen for convenience at the center of cells in the ground-water flow model. Hydraulic head at each control point is calculated as the sum of the head resulting from unmanaged conditions and the drawdown resulting from managed stresses. Using the method of response functions described above for each managed stress (q_s),

$$h_{o,k} = h^{Ibc}_{o,k} + r_{o,k}\, q_{s,k}\,, \tag{30}$$

for each control point. The hydraulic-gradient constraint can be written for each gradient pair p during time period k as,

$$(h^{GradIn}_{p,k} - h^{GradOut}_{p,k}) / d^{Grad}_{p} \le G^{Target}_{p,k}\,, \tag{31}$$

where

$h^{GradIn}_{p,k}$ is total hydraulic head at the control point inside the contaminated area, in ft;

$h^{GradOut}_{p,k}$ is total hydraulic head at the control point outside the contaminated area, in ft;

d^{Grad}_{p} is the distance between the inside and outside control points, in ft; and

$G^{Target}_{p,k}$ is the target gradient, in ft per ft.

The target gradient is chosen to restrict the advective flow of ground water along the path defined by d^{Grad}_{p}. Typically, the gradient is chosen to be zero in order to prevent flow away from the contaminated area. A slightly negative gradient may be used to induce ground-water flow toward the contaminated area. A reduced positive gradient can be used to reduce the rate of transport of ground water away from the contaminated area.

By incorporating information about the effective porosity and hydraulic conductivity between the two control points, the hydraulic-gradient constraint can be reformulated as a velocity constraint. For example,

$$(K_p / p^e_p)\, (h^{GradIn}_{p,k} - h^{GradOut}_{p,k}) / d^{Grad}_{p} \le V^{Target}_{p,k}\,, \tag{32}$$

where

K_p is the horizontal hydraulic conductivity along length d^{Grad}_{p} for gradient pair p, in feet per second (ft/s);

p^e_p is the effective porosity along length $d^{Grad}_{p,k}$ for gradient pair p, dimensionless; and

$V^{Target}_{p,k}$ is the target velocity for gradient pair p during time period k, in f/s.

A negative target velocity implies ground-water flow toward the contaminated area. Additional maximum and minimum constraints involving h^{GradIn} and $h^{GradOut}$ may be necessary to restrict the optimal solution to realistic values of head in the vicinity of the hydraulic-gradient or velocity constraints. For example, head (h) at any point can be constrained by

$$h \le A^{Top} \tag{33a}$$

and

$$h \ge A^{Bot} \tag{33b}$$

where

A^{Top} is altitude of the land surface, in ft; and

A^{Bot} is altitude of the bottom of the aquifer, in ft.

Equation 33a assures that inadvertent flooding of the land surface does not occur, and equation 33b assures that the aquifer is not dewatered. Similar constraints can be used to assure that managed drawdown along the contaminated boundary is physically reasonable and that saturated thickness does not change significantly so that the approximation of hydraulic linearity used to develop the response functions remains valid.

Although the use of hydraulic-gradient and velocity constraints is a powerful and useful approach, it has important limitations. The ground-water flow model does not simulate the actual transport of water-quality constituents (solutes), nor was the model calibrated using solute data. Rather, the flow model simulates advective Darcian flow in a two-layer, quasi-three-dimensional approximation of the valley-fill aquifer (eqs. 8 and 9). Neither dispersion, nor retardation of a solute is accounted for. Porosity, an aquifer characteristic that is required to compute the rate of movement of a solute, is not part of the flow model. Additionally, the vertical complexity of the 1,000-foot-thick valley-fill aquifer is highly simplified by considering only two layers. Nevertheless, the flow model may simulate the directional movement of a conservative solute, such as chloride, reasonably well because advective transport typically dominates dispersion in a regional, coarse-grained aquifer with steep hydraulic gradients like that found in San Bernardino. These limitations are inherent in all uses of a ground-water flow model to approximate solute transport, including the use of particle-tracking programs such as MODPATH (Pollock, 1994).

Economic

Depending on the formulation of the optimization model, economic considerations can be included either as a part of the objective function or as part of the constraints. If the objective is simply to minimize total recharge and pumpage (eq. 25a), then the cost of managed recharge and pumpage can be included as a constraint. For example,

$$\text{sum k=1, ny [sum iar=1, nar } (Q^{ArtRech}_{iar,\,k}\, C^{ImpWater}_{k}) + \text{sum iw=1, nw } (Q^{Pump}_{iw,\,k}\, C^{Pump}_{iw,\,k})] \leq \text{target cost.} \quad (34)$$

The target cost can be defined by historical operations or from planned future operations. Economic constraints can involve both the combined cost of operation, such as in equation 34, and the individual cost of operating a single well or artificial-recharge basin. For example, the cost of pumping well 1 during time period 3 can be constrained by

$$Q^{Pump}_{1,\,3}\, C^{Pump}_{1,\,3} \leq \text{maximum allowable cost for well 1 in year 3.} \quad (35)$$

Several pressure zones are used within the San Bernardino area to distribute pumped ground water (Camp Dresser and McKee, 1991). The zones operate at the following pressures, expressed as head: City of San Bernardino lower zone (1,250 ft), City of San Bernardino middle zone (1,316 ft), Baseline feeder (1,370 ft), City of San Bernardino upper zone (1,415 ft), City of San Bernardino Sycamore zone (1,580 ft), and Santa Ana Valley pipeline (1,850 ft). In order for ground water to be used within the valley, except locally for agriculture or domestic purposes, the water must be lifted to the pressure of the particular distribution line. In some water-management scenarios, this additional lift (L^{Zone}) will add substantially to the cost of pumpage. To incorporate this additional lift in the optimization model, L^{Zone} needs to be added to the initial lift $L^{Initital}$ in equation 15b to account for pumped ground water being delivered to the point of distribution. In this way, the additional pumping cost is incorporated in $C^{Pump}_{iw,\,k}$ and can be included either in an objective function (eq. 24b) or in a constraint (eq. 34).

Fixed costs of operation, such as installation of additional wells, sometimes are included in an optimization model, but this procedure requires the use of integer programming, a technique that can significantly increase computational time (Schrage, 1991). Therefore, fixed pumpage and recharge costs were considered heuristically outside the optimization model for the San Bernardino area. For example, additional 16- to 20-inch production wells commonly cost at least $500,000 each for drilling, pump, motor, and site preparation (Camp Dresser and McKee, 1991). Additional 48-inch-diameter pipelines to distribute water for municipal use cost approximately $1.3 million per mile (Camp Dresser and McKee, 1991). These costs can be used to balance the locations of wells with the costs of pumpage, and to compare different configurations of wells.

Bounds

Setting limits (bounds) for each decision variable can be an efficient way of constraining an optimal solution. For example, both the MINOS and GAMS software packages use an implicit non-negativity bound for each decision variable in order to reduce the computational time needed to solve a linear-programming problem. Other limits, including an upper bound, can be specified for each variable, for example

$$B^{Low} \leq \text{decision variable} \leq B^{Up} \quad (36)$$

where B^{Low} and B^{Up} are lower and upper limits, respectively. This is an efficient way to define the physical limits of a decision variable. For example in the optimization model of the San Bernardino area, bounds can be used in lieu of some water-supply constraints. Additionally, use of bounds instead of constraints is recommended for large linear-programming problems with many constraints.

Use and Limitations

The constrained optimization model of the San Bernardino area is a powerful tool for synthesizing complex water-management issues and for quantitatively comparing different methods of operation. However, analysis of a complex water-resource problem requires a systematic consideration of many different physical, political, and societal issues. Even when considering the physical issues, appropriate use of an optimization model is to provide additional information that will aid in making better, more informed decisions (Hillier and Lieberman, 1980, p. 3–5). Not all issues can be included in the model, nor should they be if the model is to be useful. And ultimately, the model, despite whatever degree of complexity is incorporated, is only a simplification of the real world.

Although optimization techniques can be powerful, the terminology can be misleading. True, an optimal solution probably is the most anticipated result from a linear-programming model, such as the one used in this study, and the optimal solution does define the best answer that can be achieved for a specific model formulation. But it is unlikely that the optimal solution is globally optimal for all water-management concerns that are present in a real system. There always will be additional concerns that could be included in a larger, more comprehensive optimization model. For nonlinear optimization, such as might be used to analyze a ground-water system with a time-varying approximation of transmissivity, an optimal solution is even more tenuous. For example, it is not possible to prove that the optimal solution from a nonlinear optimization model is globally optimal and not simply one of many different suboptimal solutions.

The key, therefore, is to define a sufficiently complex optimization model that includes the decision variables and constraints with the greatest hydrologic impact on a water-management issue. As in any modeling process, even initial formulation of the optimization model can be insightful. The formulation step requires choosing which components are most important. Objectives and constraints must be defined precisely and quantitatively—a process that commonly is more difficult and time-consuming than it first appears. Initial use of the optimization model can provide immediate insights by defining the feasibility space of potential solutions and by determining whether specific proposed operational plans are even feasible.

In advanced use of the optimization model, various proposed water-management plans can be compared quantitatively. The most important decision variables can be identified, and equally important, the binding constraints can be identified. If necessary, key hydrologic characteristics can be verified with additional data collection prior to implementing a specific water-management plan. If the optimization process results in greater hydrologic insight for the water managers and an improved solution to a water-management problem (*fig. 51*), then the optimization model will have been useful.

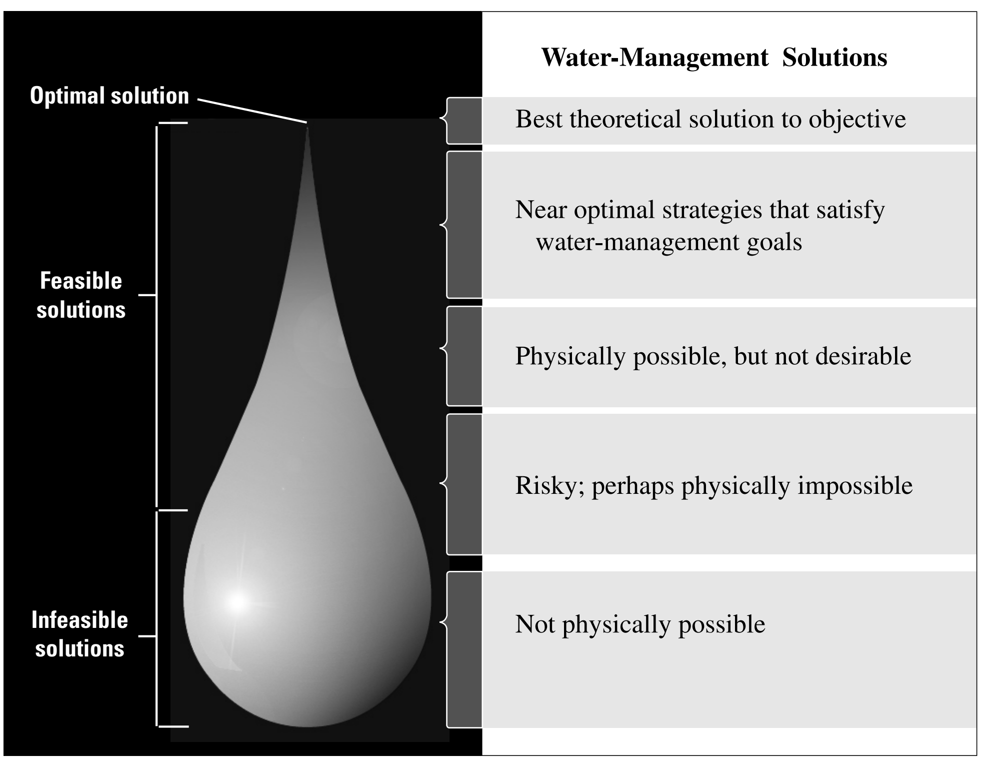


Figure 51. Water-management solutions.

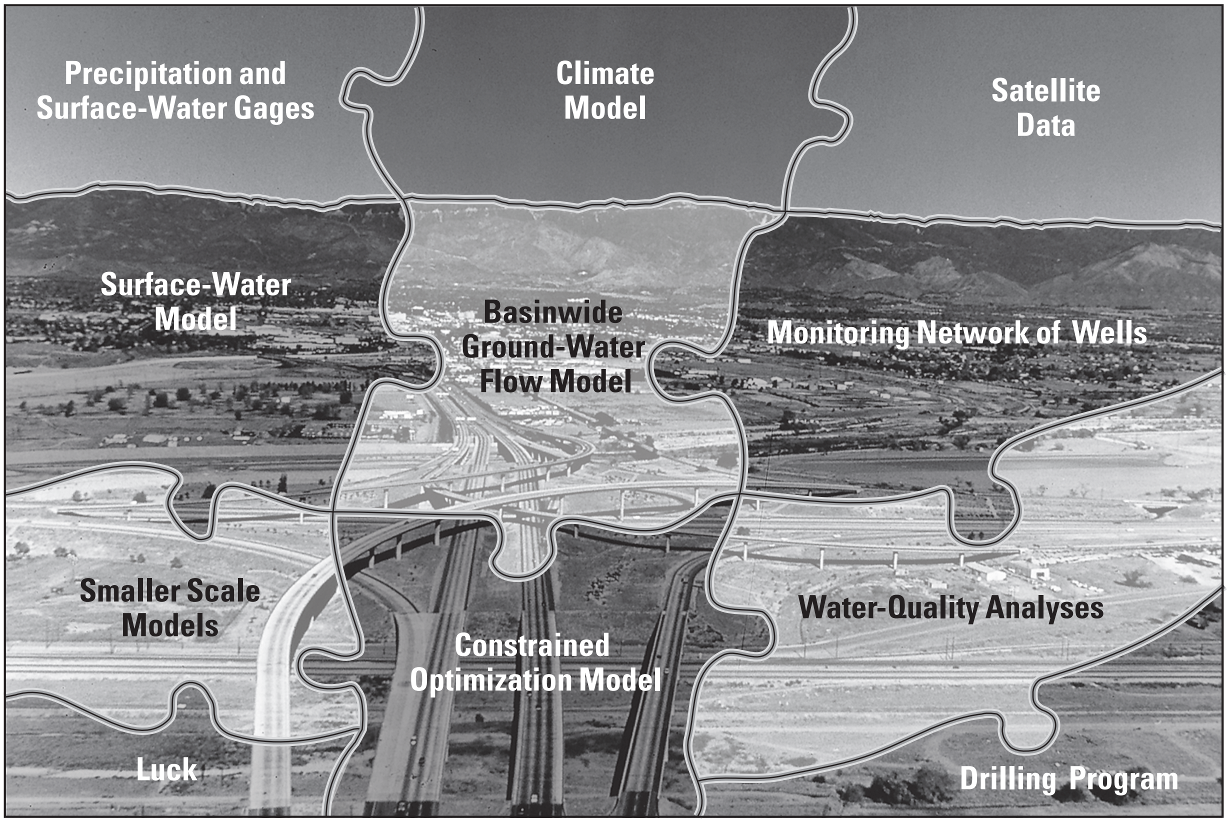


Figure 52. Water-management puzzle.

Although constrained optimization techniques can be exceptionally instructive, other traditional hydrologic investigations need to be combined with use of an optimization model. Concurrent collection of data, development and use of other mathematical models, and monitoring of the physical system all are needed to solve a typical water-management puzzle, as illustrated in *figure 52*. Information gained from each piece of the puzzle contributes in a different way to creating a more informed water-management decision.

The limitations that are specific to the optimization model of the San Bernardino area mostly are derived from limitations of the ground-water flow model that is used to calculate response functions. The true three-dimensional complexity of the aquifer system—which may be important in managing some water-quality issues—is not fully represented in the flow model, and thereby in the response functions used in the optimization model. Detailed data from multiple-depth monitoring wells (*figs. 24* and *52*) can be used to compensate for some of this simplification.

The slight nonlinearity of the ground-water flow model, which is caused by the piecewise linear and nonlinear approximation of some recharge and discharge components, may require adjustment to some results from the optimization model. Response functions that are calculated when simulated heads are near land surface will differ from those calculated when simulated heads are greater than 15 ft below land surface. Therefore, optimal pumpage calculated using the differents sets of response functions also will differ. In general, lowering simulated heads will require a greater pumpage if either evapotranspiration or gaining streams are simulated during the time period used to calculate the response functions. In this case, the additional pumpage can be as much as the quantity of evapotranspiration or gains in streamflow, as demonstrated by Danskin and Freckleton (1992). The potential adverse effects of these nonlinearitities can be determined by resimulation of the optimal values in the ground-water flow model. This resimulation is facilitated by an option in the MODMAN software, but is much more time-consuming when using GAMS software.

Limitations regarding costs include an assumption that a constant non-discounted value of dollars is sufficient to analyze pumping-cost alternatives. Because costs can be difficult to estimate and can change significantly during the time it takes water managers and the public to evaluate potential management plans, use of the optimization model may benefit from a comparison of quantities of water, rather than costs of water.

An important final caveat, similar to one regarding predictive use of the ground-water flow model, is that optimal values of recharge and discharge that are determined by the optimization model need to be reasonably close to calibrated values in the ground-water flow model. Recharge or discharge values that are significantly different from those used during calibration may prompt hydrologic conditions that are not well simulated by the flow model, for example dewatering of a hydrogeologic unit, compaction of the aquifer, or flooding of the land surface.

Evaluation of Selected Water-Management Alternatives

Future water management in the San Bernardino area of southern California was evaluated with the aid of ground-water flow and constrained optimization models described in this report. Using the models, seven water-management scenarios were developed to quantify and to better understand the important characteristics of water management in the San Bernardino area. The scenarios are not intended to be a specific plan to adopt and follow. Actual management is too complex to be adequately represented in a single simulation. Rather, the scenarios were designed with the goal of demonstrating important hydrogeologic features in the area, such as recharge of imported water along the base of the San Bernardino Mountains or pumpage near the San Jacinto fault. Improved understanding of these features will permit a comprehensive water-management plan to be developed by the local water managers.

A management horizon of 32 years was chosen to investigate the longterm effects of major water-management issues affecting the San Bernardino area. Selection of the horizon was based primarily on two criteria. First, at least two decades are needed for the effects of recharge and discharge in different parts of the Bunker Hill and Lytle Creek basins to be communicated to all other parts of each basin. As shown in figure 49, the simulated response of ground-water levels in the former marshland to artificial recharge near the base of the mountains can take more than 25 years to be fully felt.

Second, the management horizon needs to capture the historical variability of surface-water runoff in the San Bernardino area. An analysis of annual discharge data for the Santa Ana River identified a recent 16-year climatic cycle (1983–98) that has nearly the same statistical characteristics as three historical periods (1913–98, 1928–98, and 1945–98) (*fig. 53*). Average and below-average runoff for the 16–year cycle are nearly identical to the longer periods. Above-average runoff for the 16-year cycle, however, tends to be even greater than for the other periods. The prevalence of recent, high-runoff years, also illustrated in *figure 14*, is not well understood, but may be either a normal climatic cycle that is not within the historical record, or a symptom of climatic change. Creating the most credible future scenario of runoff favored using the 16-year cycle because it is the most recent data and because it may be indicative of recent changes in climate.

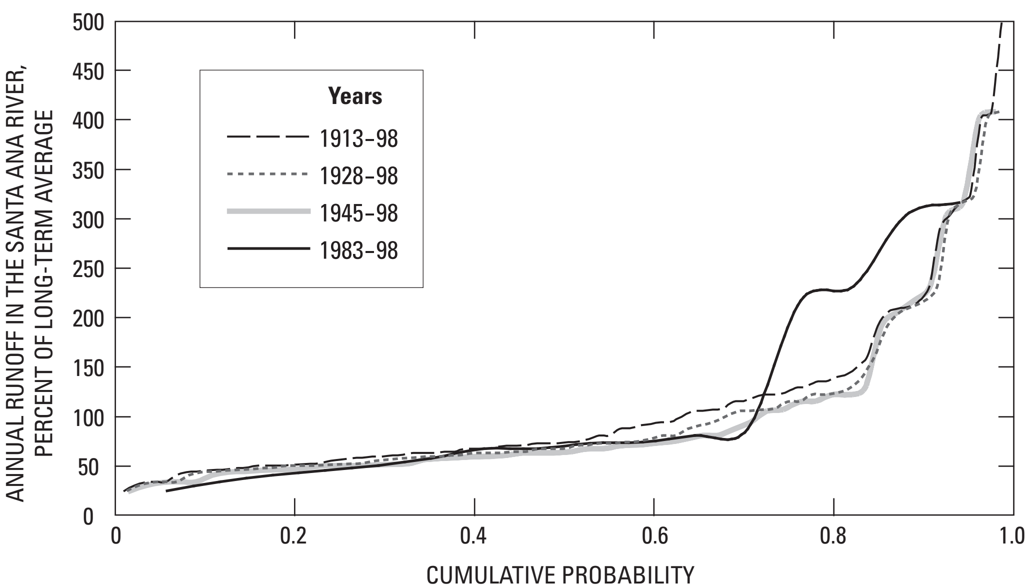


Figure 53. Cumulative probability of annual runoff in the Santa Ana River, San Bernardino area, California, for selected time periods, 1913–98. Calendar-year data for USGS gaging station 11051501.

To satisfy the criteria for a multi-decadal management horizon, two 16–year cycles were combined to extend the calibration period (1945–98) a total of 32 years into the future, to the year 2030. In order to prevent a statistical shift in runoff at the end of the calibration period, a cumulative departure-from-mean runoff curve was used to create a nearly seamless transition from the calibration period to the assumed future conditions. This method ensured no water was unintentionally added or subtracted from the simulation. Use of the cumulative runoff curve favored beginning the climatic cycle (1983–98) in 1999, then repeating it in 2015. The longest period of record for the Santa Ana River (1913–98) was used for the analysis of cumulative runoff, despite having a few months of missing data. If a shorter period, either 1928–98 or 1945–98, were used, then the match would have favored beginning the climatic cycle in 1998. Choosing one match point or the other did not appear to have a significant effect on the simulated results.

The shortest time period considered in the management scenarios is a single year—the shortest period of uniform recharge or discharge in the ground-water flow model. For some hydrologic processes, such as storm flow, this is a relatively long time. A single year also is too long to capture the seasonal effects of evapotranspiration or pumpage. But for most water-management questions in the San Bernardino area, a time frame ranging from 1 to 32 years provides an adequate ability both to simulate the hydrologic processes and to observe the effects of management decisions.

In summary, each water-management scenario simulated conditions in the San Bernardino area from 1945 to 2030. The model calibration period (1945–98) was included in each scenario to facilitate comparisons to historical conditions. This design also facilitates updating the ground-water flow model as new data becomes available and then resimulating a specific scenario. Future conditions (1999–2030) were based primarily on historical recharge and discharge that occurred during a representative climatic period 1983–98. Additional recharge or discharge was added as necesary to investigate a particular water-management issue. Scenarios 1 through 4 required use of only the ground-water flow model; scenarios 5 and 6 required use of both the ground-water flow and constrained optimization models.

Water-Management Issues

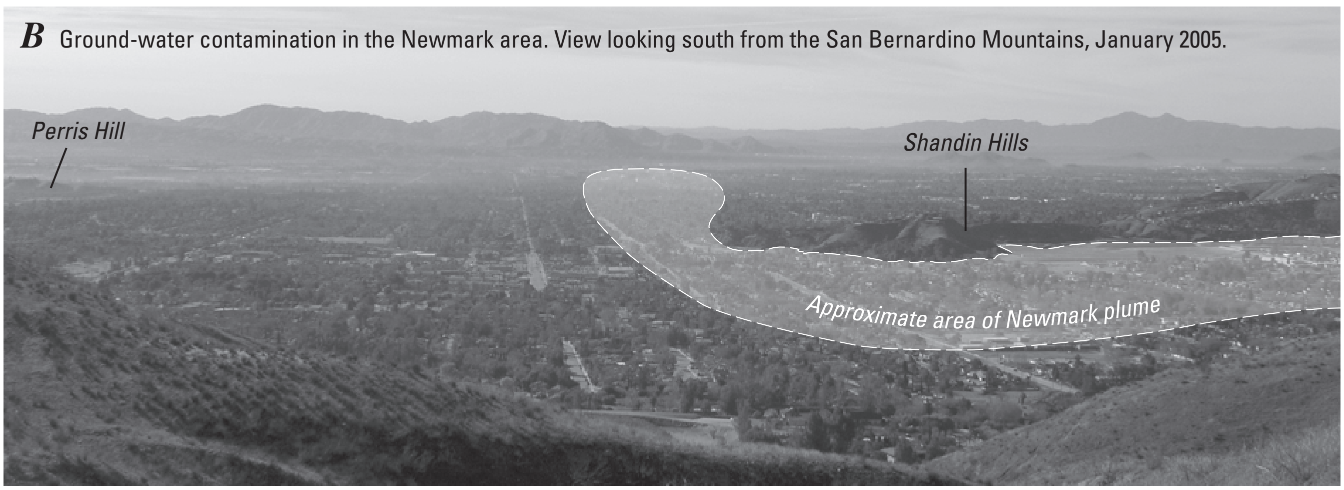
Water managers in the San Bernardino area face a variety of issues involving water supply and water quality. Solving water-supply issues often requires use of both surface water (*fig. 11*) and ground water (*fig. 31*), and in recent years, management of one typically has involved management of the other. With the large number of water-quality issues in the area (*fig. 36*), virtually all water-management decisions about water supply also involve questions and decisions about water quality.

As in many urbanizing areas, actual water management in the San Bernardino area encompasses an incredibly large number of surface-water, ground-water, and water-quality issues—far too many to evaluate in this study or to include in this report. Nevertheless, a few issues stand out from the others as having much greater than average hydrologic importance. Typically, these issues are areally extensive, create effects that last for years, and have large costs associated with them. This study attempted to integrate analysis of these primary issues, which are summarized below and are identified in photographs (*fig. 54*) and spatially (*fig. 55*).

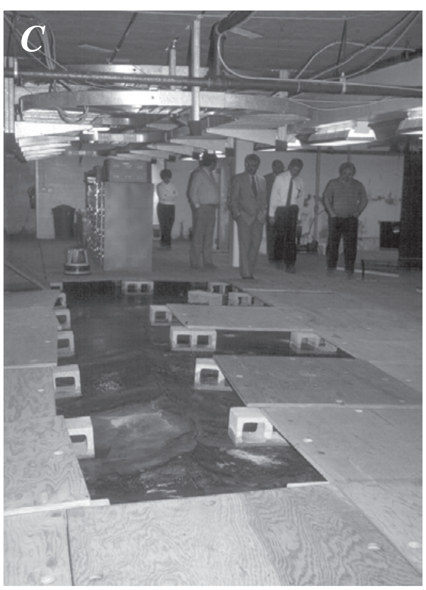
Along the base of the San Bernardino and San Gabriel Mountains, ground-water levels rise and fall dramatically with changes in recharge (*fig. 43*). During droughts, falling ground-water levels reduce the yield of wells that supply water for domestic, municipal, and agricultural use. In some cases, ground-water levels may drop below the intake bowls of a production well, requiring an expensive lowering of the pump or discontinued use of the well. Many of the water-management efforts in the San Bernardino area over the past 100 years have been to ensure a reliable source of water to users in areas near the base of the mountains. These efforts began with diversion of streams and later were expanded to include the construction of artificial-recharge basins designed to conserve surface water by recharging the valley-fill aquifer. More recently, water was imported into the San Bernardino area, and the Foothill pipeline (*fig. 11*) was constructed to transfer both native and imported water along the base of the mountains. The Seven Oaks Dam, designed originally as a flood-control facility, has been modified to include a conservation pool so that more native surface water can be distributed in the Foothill pipeline to water-treatment plants or to artificial-recharge basins. These modifications to the original surface-water system have increased the capability of managing water in the area, but at the same time, they have created numerous questions about how to manage the surface-water resources most efficiently.

A

Low ground-water levels near the base of the San Bernardino Mountains, September 2004.



B Ground-water contamination in the Newmark area. View looking south from the San Bernardino Mountains, January 2005.



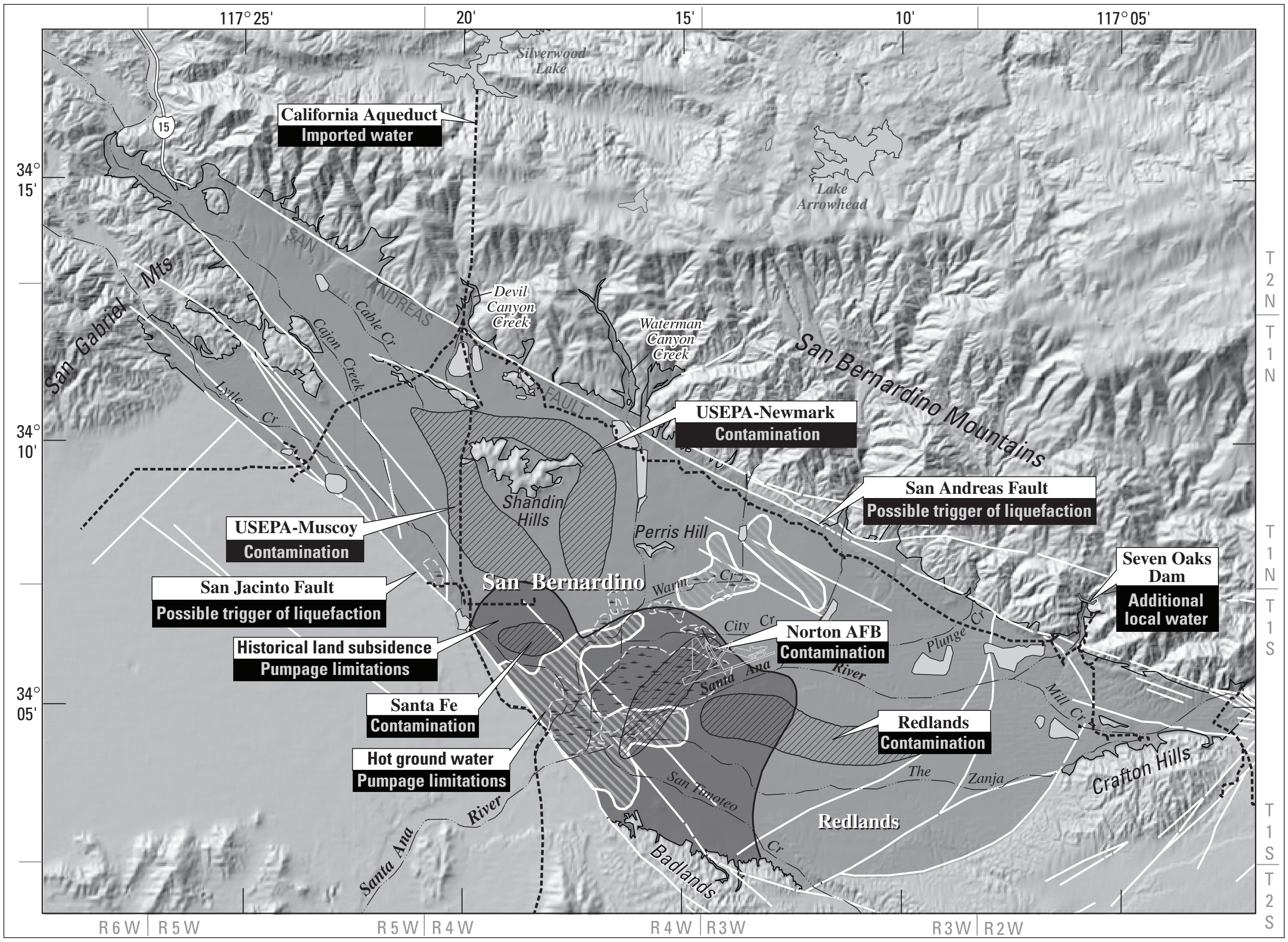
C

High ground-water levels in the former marshland. View shows groundwater up-welling through the buckled concrete basement floor of the San Bernardino Valley Municipal Water District, June 1985.

D

Possible liquifaction in the former marshland, September 2005. View is of the same area as in figure10.

Figure 54. Photographs of major water-management issues in the San Bernardino area, California, 1998.



Shaded relief base from U.S. Geological Survey digital elevation data, 1:24,000-scale

0 6 MILES
0 6 KILOMETERS

EXPLANATION

Basin boundary – Bunker Hill and Lytle Creek ground-water basins shaded in darker gray

Fault or ground-water barrier – May be concealed or approximately located. San Andreas and San Jacinto Faults are likely to produce a major earthquake causing liquefaction

Former marshland – Area of recurring high ground water and potential liquefaction

Poor ground-water quality – Contamination includes volatile organic solvents, which restricts use for public supply. Areas mapped in 1997 include USEPA (United States Environmental Protection Agency) superfund sites and Norton Air Force Base IRP (Investigation and Restoration Program) site

Geothermal areas – Ground water may be usable for heating or too warm for public supply

Artificial-recharge basin – Available for additional ground-water recharge of local or imported surface water. By controlling runoff, Seven Oaks Dam effectively increases the supply of local water

Historical land subsidence – Area of historical land subsidence caused by ground-water pumpage. During 1944-56, subsidence rate averaged more than 0.025 feet per year

-------- **Pipeline –** Imported water obtained from northern California can be used for either direct delivery or ground-water recharge. Entitlement for importation exceeds 100,000 acre-feet per year

Figure 55. Major water-management issues in the San Bernardino area, California, 1998.

In contrast to the area near the base of the mountains, ground-water levels near the former marshland remain nearly at land surface, even during droughts. The depth to ground water in this area is affected only minimally by short-term changes in recharge near the mountains; ground-water levels are affected most by nearby pumping and by the longterm quantity of recharge. The high ground-water levels, present during most of the last 100 years, temporarily dropped from about 1950 to 1975, a period of extensive ground-water pumpage and lower-than-average recharge. During this period, the town of San Bernardino expanded and built on the former marshland. Beginning in about 1980, ground-water levels rose to within a few ft of land surface, and the increase in hydrostatic pressure damaged basements, foundations, and concrete flood-control channels. Perhaps more importantly, the high ground-water levels indicate that the fine-grained deposits near land surface are saturated and, therefore, are susceptible to liquefaction during an intense earthquake. The enormously expensive and potentially deadly threat of liquefaction is of particular concern in the San Bernardino area because of the proximity of two major active faults—the San Jacinto and San Andreas—that border either side of the area (*fig. 55*).

The substantial increase in pumpage in the vicinity of the former marshland beginning about 1945 not only lowered ground-water levels, but also caused land subsidence of as much as one foot between about 1950 and 1970. Recent water-management efforts have focused on increasing pumpage from the former marshland in order to lower ground-water levels and thereby reduce the possible adverse effects of liquefaction. Additional pumpage from this area of abundant wells also may be needed to provide additional water to meet future municipal demand. Although some additional pumpage in the vicinity of the former marshland may be helpful or necessary, ground-water levels need to be maintained above historic minimum levels to prevent additional compaction of fine-grained materials in the valley-fill aquifer, and the accompanying land subsidence.

Many ground-water-quality issues are present in the San Bernardino area, including contamination by volatile organic compounds, such as TCE and PCE; high concentrations of nitrate and DBCP, a pesticide; high concentrations of minor inorganic constituents, such as fluoride; elevated temperature of ground water near geothermal areas; and an increase in dissolved-solids concentration resulting from the importation and reuse of water. Knowledge about the areal and vertical distribution of these water-quality problems varies, as does understanding of the physical processes affecting these problems. Some ground-water-quality problems, such as the TCE contamination near Newmark (*fig. 55*), have been identified spatially by extensive drilling and sampling. Other problems, such as TCE contamination in the Redlands area, are not characterized as well either horizontally or vertically. Some physical processes, such as the movement of fluoride in ground water, are well understood; others, such as the transport and fate of nitrogen compounds, are the subject of ongoing research. These uncertainties—both in describing the physical extent of ground-water-quality problems and in understanding the physical processes governing the transport and fate of the contaminants—limit the ability of water managers to make informed decisions.

All water-management issues have an economic component. For complex issues, economics by itself may not determine the eventual answer, but it does aid in discarding some alternatives and favoring others. Possible changes in water management involve capital costs, such as installing additional pipelines or wells, and operational costs, such as pumping water a greater distance to the land surface or across the land surface to other areas of use. The electrical cost of moving water typically is a sizeable percentage of the total operational cost and can vary significantly depending on when and where the electricity is obtained. The operational cost of supplying water, which in 1998 varied from about $100 to $400 per acre-ft, and the capital cost of acquiring new water supplies, which exceeded $1,000 per acre-ft, commonly are used to evaluate the economic viability of any possible change in water management.

One of the most difficult water-management issues involves the long-term operation of the valley-fill aquifer for water supply. In defining this operation, an immediate quandary appears: how to maintain both a full and an empty basin. A full basin during a period of abundant runoff is an expensive lost opportunity for recharge; an empty basin during a drought is a dangerous economic and political liability. An important part of water management in the San Bernardino area involves defining an acceptable balance between these two alternatives.

Description of Water-Management Alternatives

Water managers in the San Bernardino area generally are aware of the most important water-management issues confronting them. This knowledge has been gained from years of living and working in the area and from the many studies commissioned by the several water districts and city water departments to study possible solutions to individual water problems. During the past three decades, comprehensive water-management studies have been done, though only a few recommended solutions have been implemented. Part of the reason appears to have been a lack of technical understanding about key water-management issues, such as the transport of organic contaminants or the susceptibility of saturated sediment to liquefy. Part of the reason appears to have been a lack of agreement about who will pay for the solution. And, part of the reason may have been a lack of active involvement by all the major water-purveyors in any comprehensive water-management plan.

To help to address this last reason, the San Bernardino Valley Municipal Water District in 1987 formed an advisory commission composed of the major water purveyors in the area. At monthly meetings of the advisory commission, significant water issues and proposed water projects are presented and critiqued by the members. In this way, projects are brought forward in an open forum. A technical subcommittee of the commission also meets as needed to provide a more in-depth analysis of specific issues or projects. Membership and participation in the advisory commission is voluntary and typically includes more than 20 different public entities.

In 1990, a comprehensive water-management plan for the San Bernardino area was begun by the San Bernardino Valley Municipal Water District. The consulting firm of Camp, Dresser, and McKee was hired to oversee development of the plan; the advisory commission and, in particular, the technical subcommittee of the advisory commission provided an ongoing critique of the plan. The planning process extended over a 6-year period and included a broad range of technical investigations (Camp, Dresser, and McKee, 1990; 1991; 1995a,b; 1996). The present study—which includes a description of the hydrologic system, installation of monitoring wells, development of the ground-water flow and constrained optimization models, and evaluation of water-management alternatives—is a significant part of those technical investigations. The advisory commission and technical subcommittee continuously reviewed the planning procedures and the technical investigations, including this one, and provided suggestions that resulted in changes to both.

An important product of the planning process has been development and analysis of various water-management alternatives designed to help solve one or more of the major water-management issues affecting the San Bernardino area. These alternatives are described briefly below.

1. Continue the present water-management operations. This alternative may be perceived as the easiest path to pursue. It may be perceived that maintaining present operations will create the fewest new problems.

2. Raise low ground-water levels near the base of the San Bernardino Mountains with additional recharge. If ground-water levels near the base of the San Bernardino Mountains are unacceptably low, then raise them by recharging additional water from local sources or water imported via the State Aqueduct.

3. Recharge additional water so that the valley-fill aquifer does not become depleted. A recurring concern when ground-water levels decline is that the valley-fill aquifer may become depleted. Demand for water throughout the San Bernardino area is recognized as increasing, and there is concern that recharge may not be keeping up with increased pumpage.

4. Lower high ground-water levels near the former marshland with additional pumpage. If ground-water levels near the former marshland are too high, then pump additional ground water to lower the levels. This additional pumpage ideally might be located in the immediate area of the high-ground-water problem and would be pumped from either existing or new wells.

5. Extend the Baseline feeder pipeline to the east or south. This extension along with existing or new production wells along the pipeline would enable additional ground water to be pumped from the area of the former marshland. This additional ground-water production could be provided to urbanizing areas west of the Lytle Creek basin or it could be pumped into the State Aqueduct.

6. Create a water-transfer credit with the State Aqueduct system. If there is excess ground water near the former marshland, then one alternative would be to pump some of the excess ground water into the State Aqueduct in exchange for a future credit. The quality of the ground water pumped into the aqueduct and the willingness of the State to accept this water may be important concerns.

7. Pump excess ground water into the Santa Ana River. If there is excess ground water near the former marshland and no demand for the water, then simply pump it and discharge it into the Santa Ana River. This alternative would relinquish a valuable quantity of ground water, but might avoid expensive damages from liquefaction caused by a major earthquake.

8. Satisfy future increased demand for water by providing surface water. A major advantage of using surface water is that it comes into the San Bernardino area at a higher head than that required for distribution. The major disadvantage is that surface water must be treated prior to being distributed for municipal use. In most cases, pumped ground water is delivered directly from the production wells to the consumer with minimal treatment.

9. Satisfy future increased demand for water by providing ground water. Major advantages of using ground water are that it can be obtained throughout most of the San Bernardino area relatively inexpensively, and it generally does not need to be treated except for adding chlorine prior to being distributed for municipal use. Major disadvantages are that ground water has a lower head by the time it is pumped from wells and therefore may need to be boosted to a higher head for distribution.

10. Understand the possible effects of climate change on water availability. Discussions about possible climate change cause concern among both local residents and water purveyors. Neither they nor scientists yet understand what if any effects of climate change will need to be incorporated into future water-management plans.

11. Install and operate extraction wells along the leading edge of the Newmark contamination site to prevent contaminated ground water from moving further downgradient. These wells, referred to locally as the barrier wells, would presumably create a hydraulic barrier to prevent further ground-water movement. The number, placement, perforations, and pumping rate of the wells have been persistent technical questions.

12. Determine effective joint cleanup of the Newmark and Muscoy contamination sites. These two USEPA superfund sites are physically close to each other and are hydraulically connected via the ground-water system. Operational decisions about how to cleanup one site will affect the ability and cost of cleaning up the other site.

13. Install and operate extraction-injection wells on Norton Air Force Base to cleanup localized ground-water contamination. As part of the closure of Norton Air Force Base, several areas and types of ground-water contamination were found. As of 1998, remediation plans included three new wells to extract contaminated ground water, a processing plant to remove the contamination, and seven new wells to re-inject the treated water.

14. Install a set of production wells near the Redlands contamination site to pump and treat contamination. This alternative attempts to restrict migration of contaminated ground water by controlling the hydraulic gradients near the assumed site of contamination. As of 1998, however, the areal and vertical extent of the contamination was not well defined.

15. Treat ground water from any production wells adversely affected by contamination from the Redlands site. This option recognizes the difficulty in restricting movement of the ground-water contamination and instead remedies any contamination when it adversely impacts a municipal supply well.

Simulated Scenarios

Selection and Design of Scenarios

The selection and design of water-management scenarios was made in consultation with numerous individuals living and working in the San Bernardino area. These individuals included staff from several of the local water districts and city water departments and staff from the consulting firm of Camp, Dresser, and McKee who were requested by the San Bernardino Valley Municipal Water District to develop an overall water-management plan for the area. A technical advisory committee, composed of these individuals, met many times over a 5-year period to identify critical water-management issues and how these issues might be analyzed objectively. The seven scenarios described in this report were designed to provide a quantitative analysis of these issues and sufficient hydrologic insight to allow water managers to understand the effects of their decisions. Each scenario, its purpose, and major results are summarized in *table 19*. Simulated recharge and discharge components for each scenario are listed in *table 20*.

Scenario 1: Average Recharge and Discharge, 1999–2030

Scenario 1 uses the ground-water flow model to simulate the effects of average recharge and discharge for the next 32 years (1999–2030). Annual variations in recharge and discharge are not simulated so that the effects of average conditions, in particular on hydraulic heads, can be seen more readily as the valley-fill aquifer moves progressively toward equilibrium. In essence, scenarios 1 and 2 show the effects of not changing the 1998 water-management operations. Scenario 1 is the steady-state version of scenario 2, which is the standard transient simulation used as the baseline for scenarios 3–6.

Simulated recharge and discharge components used in scenario 1 are described in detail below. In summary, average values for 1983–98 were used for gaged runoff, recharge from ungaged runoff, recharge from local runoff, pumpage, and return flow. Head-dependent relations were used to simulate evapotranspiration, recharge from gaged runoff, and underflow across the San Jacinto fault near the Santa Ana River. Recharge of imported water was assumed to be zero. Annual declining values of underflow were estimated for San Timoteo Canyon and Sand Canyon. All other recharge, discharge, and parameter values in the ground-water flow model were the same as those used during calibration. As shown in *table 20*, all seven scenarios were designed with strong symmetry in recharge and discharge components. Therefore, much of the description below also applies to other scenarios.

Recharge from direct precipitation for scenario 1 is assumed to continue at the same constant value used for calibration of the ground-water flow model. This value was estimated for long-term conditions (1945–98) and was assumed to represent the average arrival of recharge at the water table. Any annual variations in recharge (*fig. 18*) were assumed to be damped by water percolating tens or hundreds of ft through the unsaturated zone. In this respect, the assumption of average recharge from direction precipitation for scenario 1 is consistent with the calibrated model and probably a good estimate of any future hydrologic conditions.

One caveat, however, may be that the recent climatic cycle (1983–98) used for other recharge and discharge components in scenario 1 has a greater percentage of years with much greater than average runoff than the calibration period (1945–98) (*fig. 53*). These wetter conditions might cause some additional recharge from direct precipitation on the valley fill, although the amount is probably minor. Also, concurrent with the greater runoff are generally warmer winters, which increases evapotranspiration. Much of any increase in direct precipitation on the valley fill might be consumed by increased evapotranspiration.

Simulated gaged surface-water runoff for scenario 1 is calculated as the average for 1983–98. As shown in *figure 53*, this recent 16–year climatic cycle mimics many of the important statistical characteristics of runoff that occurred during longer, historical periods. Average runoff used for scenario 1 captures these historical characteristics in a general way. One difference, however, for the recent period is the relatively greater abundance of runoff years with 100 to 300 percent of long-term average runoff.

As a result, the average gaged runoff for 1983–98 (165,683 acre-ft/yr) is approximately 110 percent of the average for the base period 1928–98, and 113 percent of the average for the calibration period 1945–98. These differences mean that scenario 1 results in an above-average amount of gaged runoff. The differences, however, do not infer that scenario 1 uses an equivalent above-average recharge from gaged runoff. Simulated recharge from gaged runoff is a nonlinear head-dependent relation, heavily weighted toward ensuring that proportionately more recharge occurs during unusually wet years. This is accomplished in the ground-water flow model by having significantly greater streambed conductance in unusually wet years to mimic natural conditions (table 16). In scenario 1, however, recharge from gaged runoff in scenario 1 was simulated with the lower stream conductance indicative of average runoff. Therefore, recharge from gaged runoff for scenario 1 is somewhat less than what would be indicated by comparing runoff for different time periods.

As for the calibration period, simulated recharge from ungaged runoff in scenario 1 is calculated using an average recharge value scaled by gaged runoff in the Santa Ana River. As described above, average gaged runoff in the river for 1983–98 is slightly greater than for longer historical periods. Unlike recharge from gaged runoff, this higher percentage does translate directly into more recharge than for longterm conditions. The over-riding decision in designing scenario 1, however, was to be consistent with scenario 2 and the 1983–98 period, even if it resulted in slightly more recharge from ungaged runoff. The effect of this decision, however, is fairly minor (about 2,000 acre-ft/yr).

Recharge from local runoff was calculated for scenario 1 in the same way as recharge from ungaged runoff. For 1983–98, the average value of recharge from local runoff is 5,900 acre-ft/yr. This value was used as a constant value in scenario 1 and tends to represent slightly wetter conditions than the longterm average. But the difference is not great, probably about 600 acre-ft/yr, so the total effect on the ground-water system is likely to be minor.

In scenario 1, no water is imported for recharge unlike during 1983–98 when a total of about 22,000 acre-ft of water was imported for this use (*table 5*). At a recharge rate of 90 percent, this quantity equates to about 20,000 acre-ft of recharge to the ground-water system, primarily near the base of the San Bernardino Mountains (*fig. 11*). Despite the historical recharge during 1983–98, design of scenario 1 favored setting imported recharge to zero in order to observe the effects of using only native water.

Also, since the mid-1980’s, the management philosophy of the San Bernardino Valley Municipal Water District and other water purveyors has been to minimize imported water for recharge in order to conserve economic resources and to limit increasing the already high ground-water levels in the former marshland. This philsophy was accentuated in 1986 when a lawsuit was filed by the city of San Bernardino against the San Bernardino Valley Water Conservation District alledging that the District’s excessive purposeful recharge was responsible for the high ground-water levels and related damage to public infrastructure (refer p. 29). Settlement of this lawsuit involved payment of $3 million to the city of San Bernardino, and became a disincentive for any public entity purposefully recharging imported water or requesting that imported water be recharged.

Underflow for scenario 1 is set at the constant values used during calibration, at decreasing annual values calculated from regression equations 3 and 4 (*fig. 27*), or at an amount calculated by a head-dependent relation (eq. 10; *fig. 28*). These conditions assume that ground-water levels continue to decline in the Yucaipa basin (*fig. 26*) and that this decline continues to decrease underflow into the Bunker Hill basin (*fig. 27*). Any changes in ground-water levels in the Rialto-Colton basin are not likely to significantly affect underflow from the Bunker Hill basin. Therefore, despite the complexity of estimating or calculating underflow at several locations, it seems likely that the underflow assumed for scenario 1 will remain valid for a variety of future hydrologic conditions within the San Bernardino area.

Pumpage and return flow used for scenario 1 are the average of the values for 1983–98. Deciding on how to best simulate pumpage for 1999–2030 was probably the most difficult decision in designing scenarios 1 and 2. During the 16-year period, many changes in the use of wells occurred: new wells were installed, a few wells were deepened, and some wells were taken out of production to ameliorate water-quality problems. Using the pumpage from the mid-1980’s for the early 2000’s means that some pumpage is overtly wrong.

The strength of using the historical data verbatim, however, is that it represents a logically paired set with the local surface-water runoff. The historical locations and quantities of pumpage were decided by many local purveyors based on local runoff and on antecedent conditions of precipitation, runoff, and pumpage. Attempting to create a purely synthetic pumpage dataset for scenarios 1 and 2 appeared to add too much unquantified uncertainty, which would make the model results difficult to interpret. Also, the primary goal of scenario 1 was to continue present water-management operations; therefore, using historical data directly seemed the most prudent choice. Subsequent investigators may wish to revisit this decision and carefully estimate future pumpage for each existing and planned well in the San Bernardino area. With many water purveyors and various future plans, developing annual estimates of future pumpage correlated to local runoff is not likely to be a precise or trivial exercise.

Results from scenario 1 are shown in *figure 56*. The most obvious result is that simulated hydraulic heads in most areas of the model tend to flatten out relatively quickly and stay at about the level they were in 1998. Heads near Devil Canyon and in parts of the Lytle Creek basin, however, continue to decline through 2030. The decline near Devil Canyon, and in the nearby Newmark and Sweetwater areas, may result from having zero recharge of imported water. The decline also may result from increased pumpage related to ground-water contamination and cleanup. This increase in pumpage would depress the simulated head even further below historically high levels, which were sustained for a time (1972–85) by recharge of imported water (*table 5*). Assumptions of average recharge and pumpage probably are not reliable in this area of the model; therefore, simulated results need to be used with caution.

The rationale for the decline in head in the Lytle Creek basin (WSB 1, *fig. 56*) is less obvious, but may be related simply to less recharge from Lytle Creek. As described above, average recharge of gaged runoff as simulated in scenario 1 does not account fully for the extra recharge that occurs during years of above-average runoff. The historical record for well WSB 1 suggests that this type of recharge is critically important for the Lytle Creek basin. Imported water used for recharge in the Lytle Creek gravel pit was minimal during 1983–98 (*table 5*) and is unlikely to explain the observed decline in simulated head.

Near the area of the former marshland, simulated head for the upper model layer, which approximates the water table, is tens of ft below land surface. This result suggests that the simulated quantity of pumpage along with head-dependent relations (evapotranspiration, underflow, and streams) is sufficient to decrease high ground-water levels—in a steady-state simulation. This result is instructive, in that it suggests the ground-water budget for scenario 1 is nearly balanced, but it greatly oversimplifies the true dynamics of the system, as illustrated below with results from scenario 2.

Scenario 2: Annual Variations in Recharge and Discharge, 1999–2030

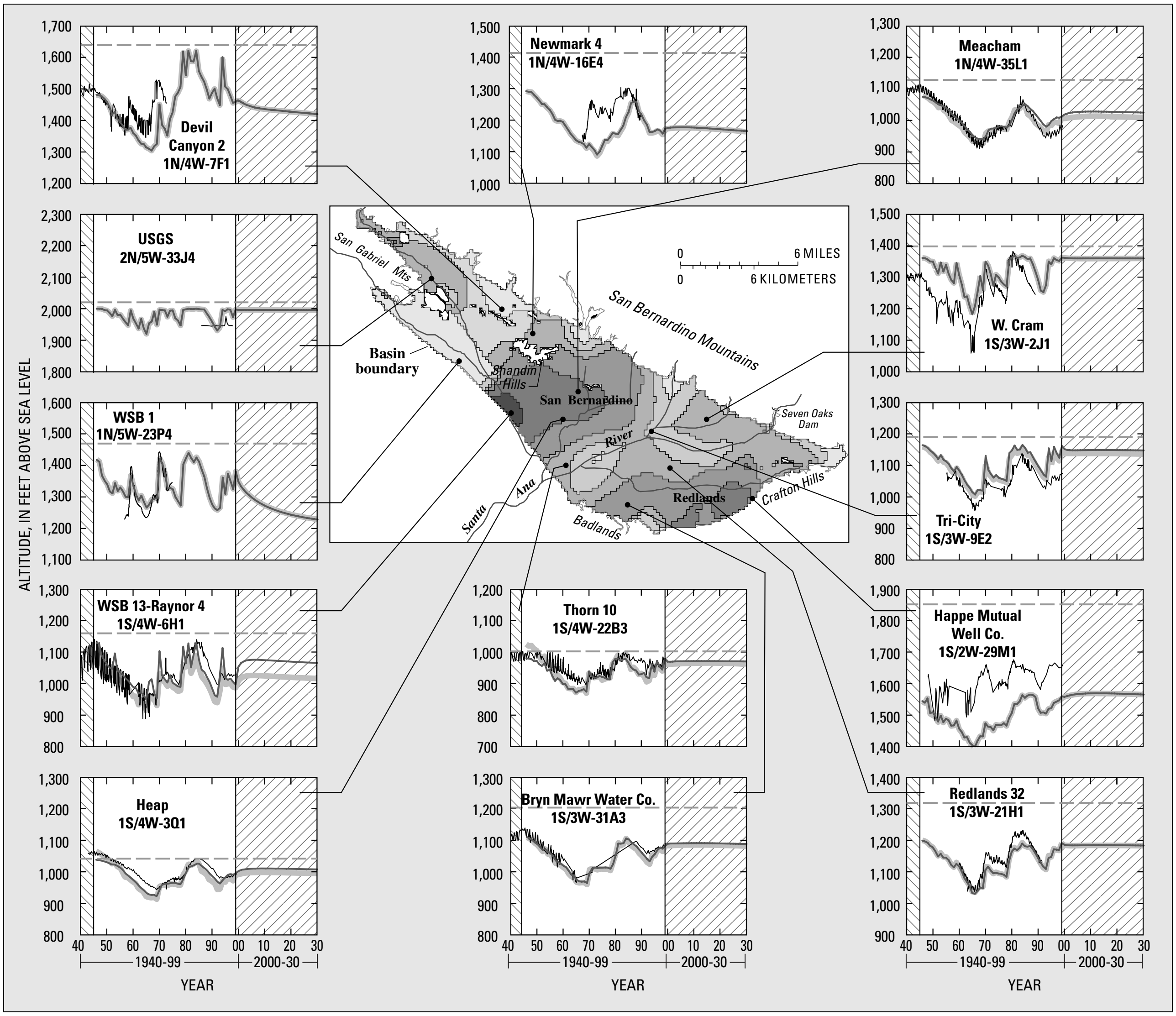
Scenario 2 uses the ground-water flow model to simulate continuing the 1998 water-management operations with annual variations in recharge and discharge. Scenario 2 is essentially the transient version of scenario 1, and the baseline for scenarios 3–6. The recent 16-year climatic cycle 1983–98 is used as the basis for the annual variations in recharge and discharge. Two of the 16-year cycles are combined to form the 32-year evaluation period, 1999–2030. Results of the simulation are shown in *figure 57*.

The simulated recharge and discharge components for scenario 2 are summarized in *table 20* and to a great extent are described above for scenario 1. The components that are the same for both scenarios 1 and 2 include: recharge from direct precipitation, recharge from imported water (zero), underflow for the constant flux areas (Badlands, Redlands Heights, Reservoir Canyon, and San Jacinto fault near barrier J), and underflow from the declining flux areas (San Timoteo Canyon and San Canyon). Two head-dependent relations (evapotranspiration and underflow across the San Jacinto fault near the Santa Ana River) also are the same though the simulated discharge will be different because the simulated heads will be different.

Differences between scenarios 1 and 2 include recharge from ungaged runoff, recharge from local runoff, pumpage, and return flow. Simulating each of these components used the annual values from 1983–98 verbatim. Because of this approach and because the components are simulated as specified fluxes, the average values for scenario 1 and the time-varying values for scenario 2 are comparable.

The major difference between scenarios 1 and 2 is recharge from gaged runoff. For scenario 2, simulating recharge from gaged runoff includes both annual runoff for 1983–98 and greater streambed conductance during unusually wet years. Use in the ground-water flow model of a head-dependent stream-aquifer relation for gaged runoff and different annual values of streambed conductance make a priori comparisons between average recharge for scenario 1 and time-varying recharge for scenario 2 qualitative at best. Scenario 1 approximates a typical “safe yield” analysis that uses average values, and, therefore, is susceptible to errors of interpretation caused by unusually wet years. For this reason, rigorous water-level and water-budget analysis should use results from the ground-water flow model simulating time-varying recharge such as done in scenario 2.

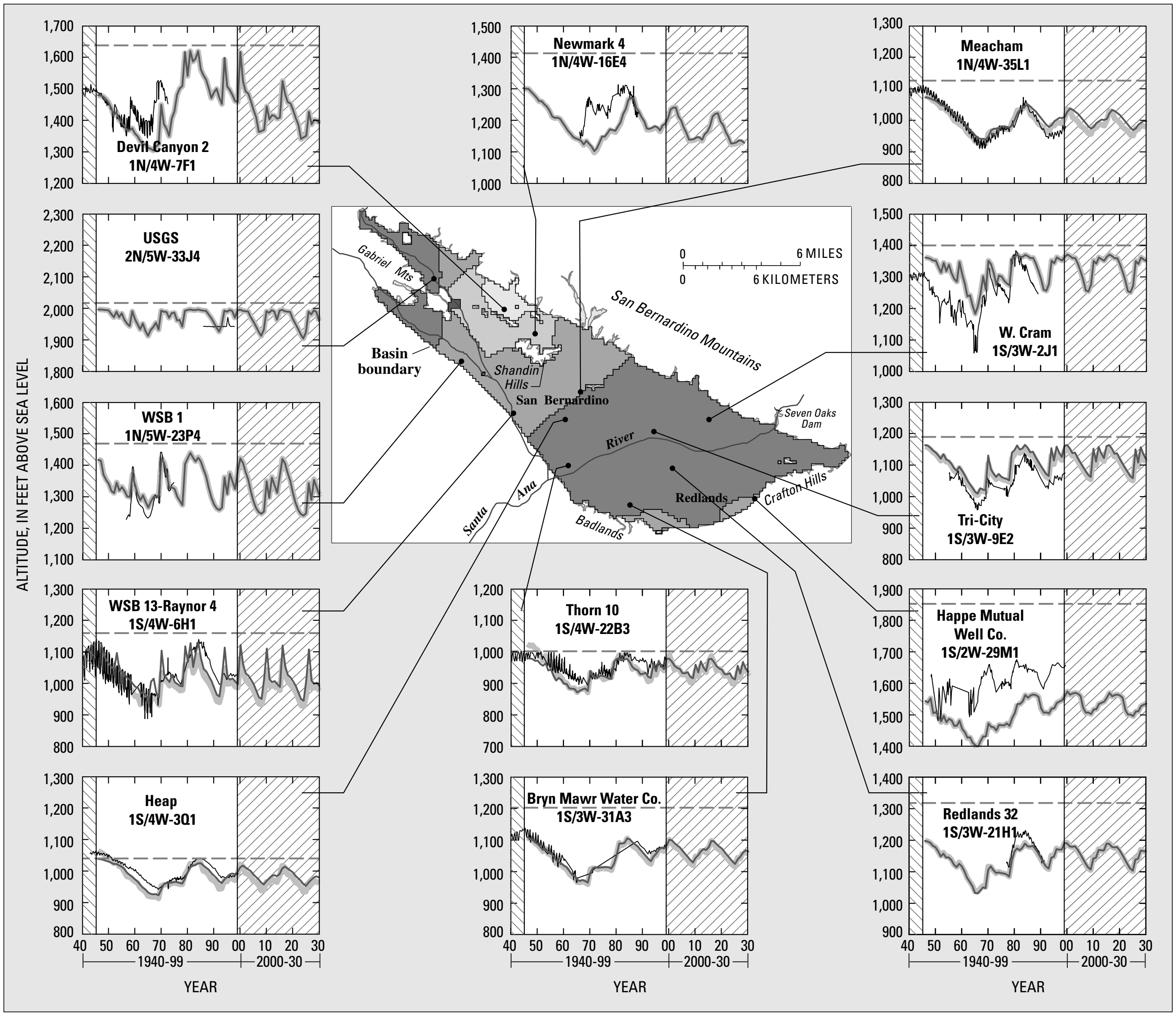
Comparing simulated results from scenario 2 (*fig. 57*) with those from scenario 1 (*fig. 56*), however, can be qualitatively instructive; a similar overall trend is present in most respective hydrographs. Identifying this trend, which appears to be approximately flat or slightly downward with time, was a primary goal in designing scenario 1. Vertical hydraulic gradients also are similar for both scenarios. These similarities confirm that that scenario 1, although simplified in particular with respect to recharge from gaged runoff, is an instructive version of scenario 2. The inset map for scenario 1 shows the complexity of simulated response throughout the valley-fill aquifer despite using average recharge and discharge. The inset map for scenario 2 may be more representative because of the symmetry of conditions preceeding 1998 and 2030.



EXPLANATION

Graphs	Map
Simulated hydraulic head—For each layer of the ground-water flow model Upper layer Lower layer **Measured ground-water level** **Land surface** **Calibration period** **Scenario 1** 1940 1945-98 1999-2030	**Difference in simulated hydraulic heads between 1998 and 2030**—Values in feet for the upper layer of the ground-water flow model. Rise indicates heads for scenario 1 in year 2030 are higher than heads in year 1998. **Rise**: **> 30**, **10–30**, **0–10** **Decline**: **0–10**, **10–30**, **> 30**

Figure 56. Results from water-management scenario 1 in the San Bernardino area, California, 1999–2030.



EXPLANATION

Graphs	Map
Simulated hydraulic head—For each layer of the ground-water flow model; Upper layer; Lower layer; **Measured ground-water level**; **Land surface**; Calibration period; Scenario 2; 1940; 1945-98; 1999-2030	**Difference in simulated hydraulic heads between 1998 and 2030**—Values in feet for the upper layer of the ground-water flow model. Decline indicates heads for scenario 2 in year 2030 are lower than heads in year 1998. **Decline**: **0–10**, **10–30**, **30–50**, **> 50**

Figure 57. Results from water-management scenario 2 in the San Bernardino area, California, 1999–2030.

In general, simulated results from both scenarios 1 and 2 illustrate that the valley-fill aquifer is in balance without recharge of imported water. The slight decline in heads observed during the 32-year period probably is too minor to be indicative of a longterm change; rather the decline likely is within the range of uncertainty of the recharge and discharge values chosen for scenarios 1 and 2. An isolated exception is the decline in simulated head observed near the mountain front north of Shandin Hills. An important caveat to the conclusion that the valley-fill aquifer is in balance is that recharge chosen for scenarios 1 and 2 represents recent runoff, which has been slightly greater than longterm historical conditions, and therefore may not be indicative of future conditions.

One of the characteristic results from scenario 2 is the repeating, sinusoidal pattern of simulated heads in each of the hydrographs (*fig. 57*). This pattern is a natural result of the climatic cycle used for scenario 2, but the fluctuations are enhanced by human actions. During years with below-average runoff, pumpage increases to make up for the lack of surface water. During years with above-average runoff, the reverse occurs. As a result, stress on the ground-water system is amplified during both wet and dry years.

The vertical range of this wet-dry cycle demonstrates the normal, active range of heads that can be expected to occur in the valley-fill aquifer. Significant annual and decadal fluctuations in ground-water levels are part of the historical record and, based on the results from scenario 2, similar fluctuations are likely to recur independent of any changes in water management. These fluctuations, both in the unconfined and confined hydrogeologic units, typically are 100 ft or more. Historically, many water purveyors have lowered or raised pump bowls to follow fluctuating ground-water levels. This retooling of production wells has been both a frustration and a cost to some water purveyors and seems likely to continue based on the fluctuations observed in *figure 57*.

An implicit part of future fluctuations in ground-water levels are future fluctuations in ground-water storage. In this respect, the results from scenario 1 are misleading. Achieving constant ground-water levels or no change in ground-water storage, probably is not possible. Rather, just as during the calibration period (*fig. 45*), significant changes in storage will occur. The magnitude of these changes likely will be similar to that observed during 1983–98; annual fluctuations of as much as 200,000 acre-ft and a cumulative change of as much as 500,000 acre-ft, or more, can be expected. These fluctuations create opportunities to capture native runoff, but also may cause concern when ground-water levels are dropping, and the basin appears empty, unlikely to refill. Fluctuating ground-water levels and ground-water storage are an inherent part of the present operations in the valley-fill aquifer. The dynamic ground-water flow simulated in scenario 2, coupled with the public response to varying hydrologic conditions from 1984 to 1998, prompted the quote at the beginning of this report,

> "And it never failed that during the dry years the people forgot about the rich years, and during the wet years they lost all memory of the dry years. It was always that way." John Steinbeck

This dynamic, observed by Steinbeck in the Salinas Valley of California, also typifies the hydrology of the San Bernardino area.

In several areas of the model, the maximum head reached during each cycle of increased recharge and decreased pumpage is similar to the maximum heads simulated during the historical period 1945–98. In the former marshland, the maximum head during scenario 2 is nearly at land surface, indicating a potential future concern about flooding of the land surface and possible liquefaction during an earthquake. The minimum heads reached during scenario 2 are generally from 50 to 100 ft above the minimum historical heads, which occurred about 1965. Based on these results, it does not seem likely that additional land subsidence will be induced with (1998) water-management practices even if gross pumpage continues to be augmented by the discretionary increase of 10,000 acre-ft/yr exported to the city of Riverside (San Bernardino Valley Municipal Water District, 1981b, 1985).

Scenario 3: Increased Recharge Made Possible by Seven Oaks Dam, 1999–2030

Scenario 3 uses the ground-water flow model to simulate additional recharge made possible by construction of Seven Oaks Dam on the Santa Ana River. Because the primary purpose of this massive new dam is flood control, longterm retention of water behind the dam is minimized. During winter and spring, runoff is captured, then released fairly rapidly so that the dam has available capacity to prevent any future flooding that might occur. This operational policy tends to limit purposeful ground-water recharge that could be increased by presence of the new dam.

To better satisfy local water purveyors who want Seven Oaks Dam not only to control floods, but also to retain water for later ground-water recharge, the Army Corps of Engineers added a conservation pool behind the dam. Although the pool eventually would fill with sediment, the pool would have an initial capacity of 16,000 acre-ft when the dam opened in the year 2000, decreasing to 7,000 acre-ft in the year 2050. The extra water available for ground-water recharge as a result of the conservation pool was estimated to be 4,116 acre-ft in 2000 decreasing to 2,140 acre-ft in 2050.

Formulation of scenario 3 used scenario 2 as a baseline for all recharge and discharge components (table 20). To this baseline was added annual recharge as specified flux in one or more of the artificial-recharge basins (*fig. 11*). A total of eight different combinations of quantity, location, and temporal distribution of recharge were simulated as part of scenario 3. These test cases used annual recharge ranging from about 2,000 acre-ft to about 24,000 acre-ft. This extra recharge most commonly was put into the upper Santa Ana River basin or into the upper and lower Santa Ana River basins. For one test case, all seven artificial-recharge basins were used with the average annual recharge that occurred from imported water during 1972–98 (*table 5*; *fig. 11*).

In all eight test cases, the net quantity of recharge was much less than expected, generally averaging about 3,000 acre-ft/yr or less. In some ways, this result should have been expected. Any increase in ground-water recharge, particularly over a period of years, will increase ground-water levels, which in turn will increase discharge that is dependent on ground-water levels: evapotranspiration, underflow across the San Jacinto fault near the Santa Ana River, and ground-water discharge to streams. To account for this interdependence, the change in ground-water storage compared to scenario 2 was used as a measure of the effect of increased recharge.

For example, the test case that used the estimate of 4,116 acre-ft of available recharge in 2000 declining to 2,932 acre-ft in 2030 resulted in an average net increase in ground-water storage of 1,163 acre-ft/yr. This increase equates to about 37,000 acre-ft of water contributed to the valley-fill aquifer during 1999–2030. Results of this simulation are shown in *figure 58*. The primary area of influence from this additional recharge to the valley-fill aquifer is near the upper and lower Santa Ana River basins, which were used for the recharge. The broader influence extends over the eastern half of the Bunker Hill basin, minimally changing the pattern of heads resulting from scenario 2 (*fig. 57*). Because this additional recharge is minimal compared to other recharge and discharge, the hydrographs for scenario 3 are similar to those for scenario 2.

What then is the value of the conservation pool? Viewed from one perspective, adding 37,000 acre-ft of water over a period of 32 years is nearly inconsequential in an area that averages nearly 150,000 acre-ft/yr of gaged runoff (*table 1*), and more than 180,000 acre-ft/yr of gross pumpage (*fig. 25*). However, viewed from another perspective, importing 37,000 acre-ft of water would cost a minimum of $1.5 million just for electrical costs to lift it over the San Bernardino Mountains, and could cost as much as $37 million if it were from a new source of water (refer p. 107 this report).

One of the quandries of water management in the San Bernarino area is how to recharge water and capture it for use before it leaves the basin, or rather before it prompts, via a rise in ground-water levels, other water to leave the basin (*figs. 49* and *50*). Some previous water-management efforts have addressed this issue by using a "put and take" system where additional recharge is extracted a short distance away before it can significantly affect the rest of the aquifer. This technique can be successful, but it requires that an additional source of water be contemporaneous with an additional demand for water. Often extra water occurs when local demand can be satisfied in other ways—from precipitation, surface water, or other pumpage.

One water-management idea that has been suggested is to export the pumped water for a credit to be used later when demand exceeds available local supplies. This idea is attractive because it stores the water as credit or cash, which may avoid the "user fee" associated with losing much of the additional recharge to increased discharge from the valley-fill aquifer. The presence of the State Aqueduct passing through the San Bernardino area in conjuction with large-capacity pipelines within the area (*fig. 11*) makes this a viable water-management option that would not be possible in many semiarid ground-water basins.

Analysis of the eight test cases for scenario 3 also illustrates the inherent difficulty of recharging available water during a period of abundant runoff. Ironically, the availability of surplus imported water often coincides with wetter-than-average local hydrologic conditions and decreased available storage capacity in the aquifer system. Stream channels and artificial-recharge basins are saturated, recharge tends to be rejected, and more ground-water is prompted to flow out of the basin either via streams or as underflow. A high-capacity, well-connected recharge, extraction, and distribution system might be required to overcome these hydraulic characteristics of the valley-fill aquifer and retain extra runoff for later use, as a paper credit.

In summary, water managers probably cannot escape two basic, controlling hydraulic characteristics of the San Bernardino area: (1) extra water becomes available during relatively infrequent periods of abundant runoff and (2) extra recharge prompts extra discharge from the valley-fill aquifer.

Scenario 4: Increased Pumpage Using Existing Wells, 1999–2030

Scenario 4 uses the ground-water flow model to simulate increased pumpage to lower ground-water levels in the former marshland and to meet increased future demand for municipal water. High ground-water levels in the former marshland have caused a variety of problems, including buckled foundations, damaged flood-control structures, and severed utility lines. Most of this damage occurred in the early 1980's when ground-water levels rose to near land surface as a result of increased recharge of the valley-fill aquifer (*fig. 59*). The increase in hydrostatic pressure also creates the potential for liquefaction of near-surface earth materials as a result of ground-shaking during a severe earthquake.

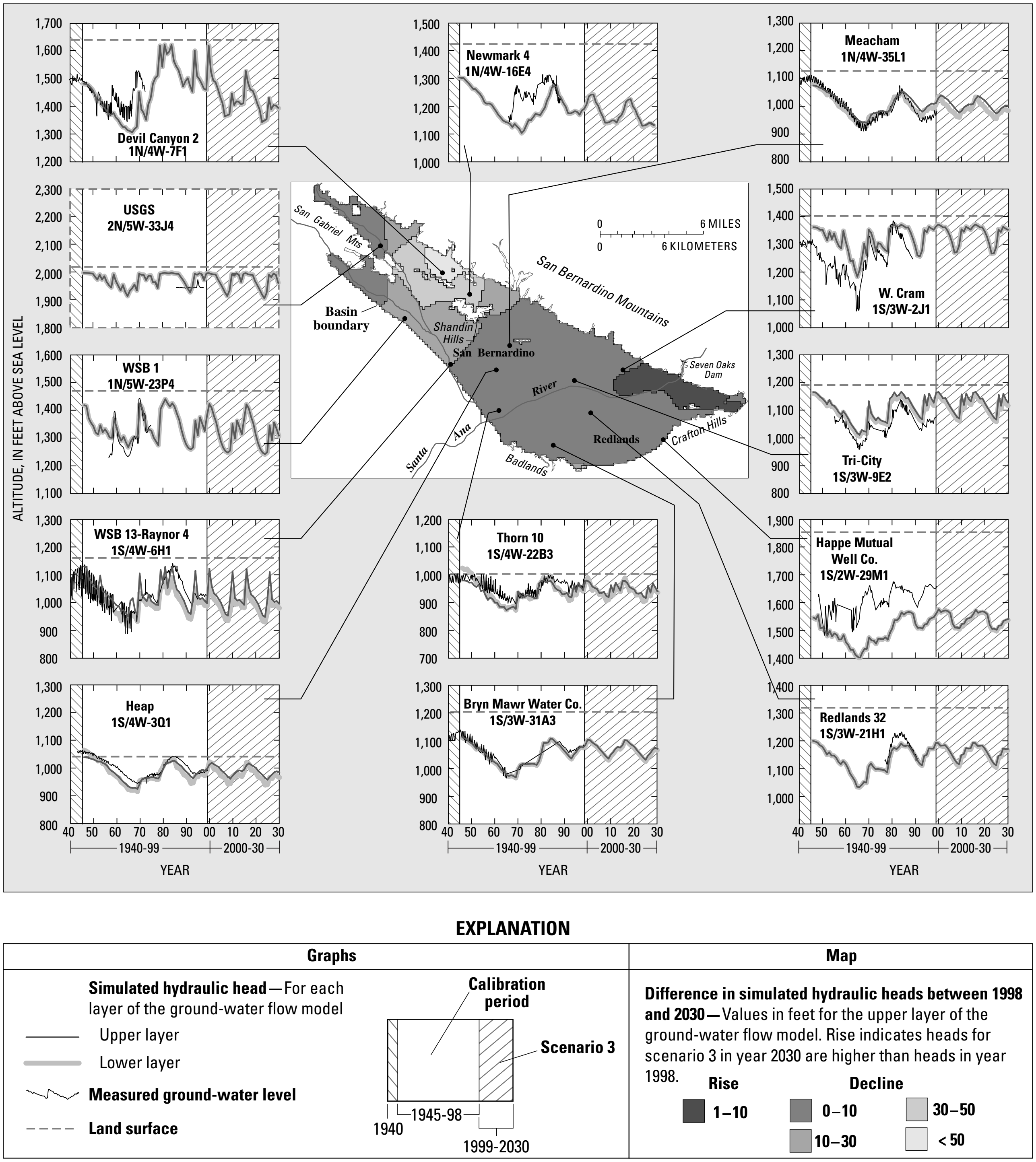


Figure 58. Results from water-management scenario 3 in the San Bernardino area, California, 1999–2030.

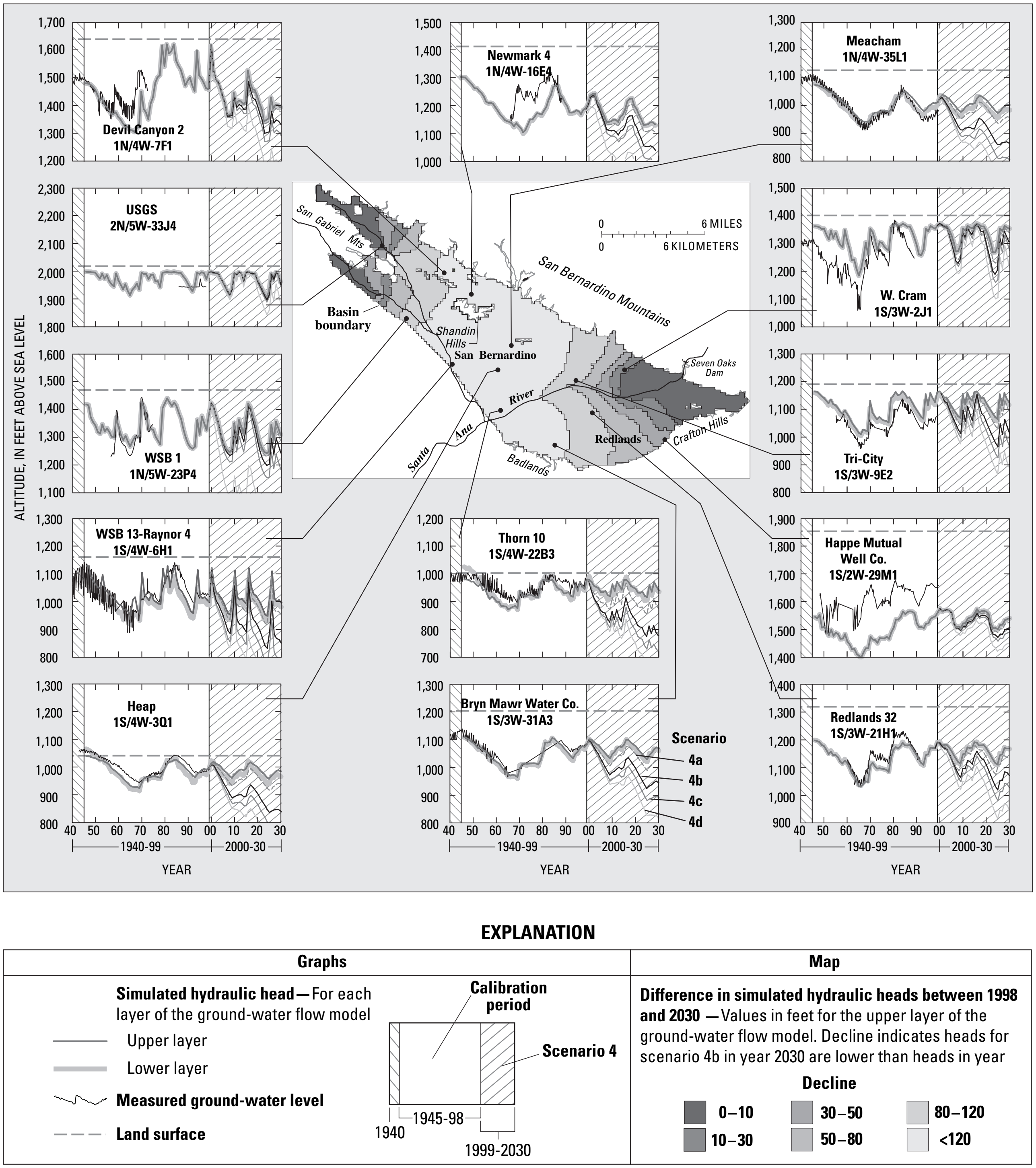


Figure 59. Results from water-management scenario 4 in the San Bernardino area, California, 1999–2030. In each hydrograph, simulated hydraulic heads for the upper model layer during 1999–2030 are shown by four lines from top to bottom for scenarios 4a–d, respectively; results for scenario 4b are depicted by the darkest line and are presented on the inset map.

Controlling high ground-water levels has been a topic of conversation in the San Bernardino area for at least 20 years and an important part of previous water-management studies (Hardt and Hutchinson, 1980; Hardt and Freckleton, 1987; Danskin and Freckleton, 1992; Camp Dresser and McKee, 1995a, 1995b). Part of the water-management challenge of lowering ground-water levels is technical: where to place wells; how much to pump them. These questions are addressed by scenario 4. Another part of the challenge is political: local concerns such as, "Don't dry up the basin; don't pump out inexpensive native water, then have to recharge expensive imported water." These concerns, commonly voiced in meetings of the advisory commission formed by the San Bernardino Valley Municipal Water District, are the more difficult, and largely need to be addressed in the local political sphere.

Scenario 4 also shows some of the effects of providing additional ground water for municipal uses. Previous investigations by the California Department of Water Resources (1970, table 7) and by Camp Dresser and McKee (1995a, fig. 1–5) determined that demand for water in the San Bernardino area is likely to increase significantly by the year 2020. Much of this increase is caused by progressive urbanization of agricultural area, which has been occurring since before 1949 (*fig. 8*). Although a significant quantity of water is used on the remaining agricultural land and will become available as the agricultural land is converted to urban use, both studies conclude that the total demand for water in the San Bernardino area will increase, perhaps by as much as 50,000 acre-ft/yr.

A study by Camp, Dresser, and McKee (1991) evaluated two alternatives in meeting this future demand: (1) delivering additional surface water via water-treatment plants, and (2) delivering additional ground water provided by increased production from existing wells and, if necessary, from new wells. Results of the study showed that meeting demand with additional ground-water pumpage is less expensive, even if additional recharge of relatively expensive imported water is necessary to prevent a longterm depletion of ground water. Scenario 4 uses the ground-water flow model to test the effects of a range of increased pumpage using existing wells. No recharge of imported water is included in scenario 4 in order to identify the effects of only using native water.

Formulation of scenario 4 used scenario 2 as a baseline for all recharge and discharge components (*table 20*). To this baseline was added excess pumping capacity at existing wells, and any induced return flow from that excess. To calculate excess pumping capacity, the most recent 5-year period (1994–98) was used. The rationale for selecting this period is twofold: (1) nearly all wells that were pumped sometime during the period probably were still operational in 1999, and (2) high-quality historical data are available to calculate recent average and maximum pumpage for each well. Excess pumping capacity was calculated for each well as the difference between the maximum and average annual pumpage for 1994–98. Return flow was calculated for each well using the same percentages used to calculate other return flow in the model.

In order to simulate the effect of excess pumping capacity on controlling high ground-water levels, the distance of each well with excess capacity was calculated from the location where ground water (and surface water) flows out of the Bunker Hill basin. This location was defined as the mid-point of the section where ground water flows across the San Jacinto fault near the Santa Ana River (*fig. 60A*). Pumpage near ground-water outflow from the basin will produce the greatest capture of ground-water discharge and will be most effective at decreasing high ground-water levels.

Cumulative values of excess pumping capacity (gross excess pumpage, net excess pumpage, and excess return flow) compared to distance from the basin outflow are shown in *figure 60B*. Both gross and net excess pumpage increase almost linearly to a distance of about 40,000 ft indicating the relatively predictable areal distribution of excess capacity. The near-zero value of excess return flow for distances less than about 18,000 ft from basin outflow results from the prevalence of wells used solely for export to the city of Riverside; these wells have no return flow. The areal distribution of excess pumping capacity with limited return flow suggests that using existing wells may be an efficient way to control ground-water levels in the area of the former marshland.

Scenario 4 was designed to test these observations. Excess pumping capacities were divided into four groups based on distance of the individual well from basin outflow: scenario 4a (all wells within 10,000 ft of basin outflow); scenario 4b (all wells within 20,000 ft); scenario 4c (all wells within 30, 000 ft), and scenario 4d (all wells with excess pumping capacity in the entire valley-fill aquifer). The areas and relative quantities of excess pumping are indicated on *figures 60A* and *60B*, respectively.

Results for the four test cases in scenario 4 are shown on each hydrograph in *figure 59*. Scenario 4b with a distance of 20,000-ft is shown on the inset map and highlighted on the hydrographs because this distance includes most of the former marshland and the area with recently high ground-water levels. For three of the four test cases, the change in ground-water levels is dramatic. Only for scenario 4a, the 10,000-ft distance, do heads remain relatively similar to the baseline condition, scenario 2. The modest downward slope of heads for scenario 2 (*fig. 57*) and more so for scenario 4a indicates that the valley-fill aquifer is on the cusp of change. Ground-water storage is sensitive to an increase in net pumpage of as little as 14,000 acre-ft/yr, the net excess pumpage simulated in scenario 4a.

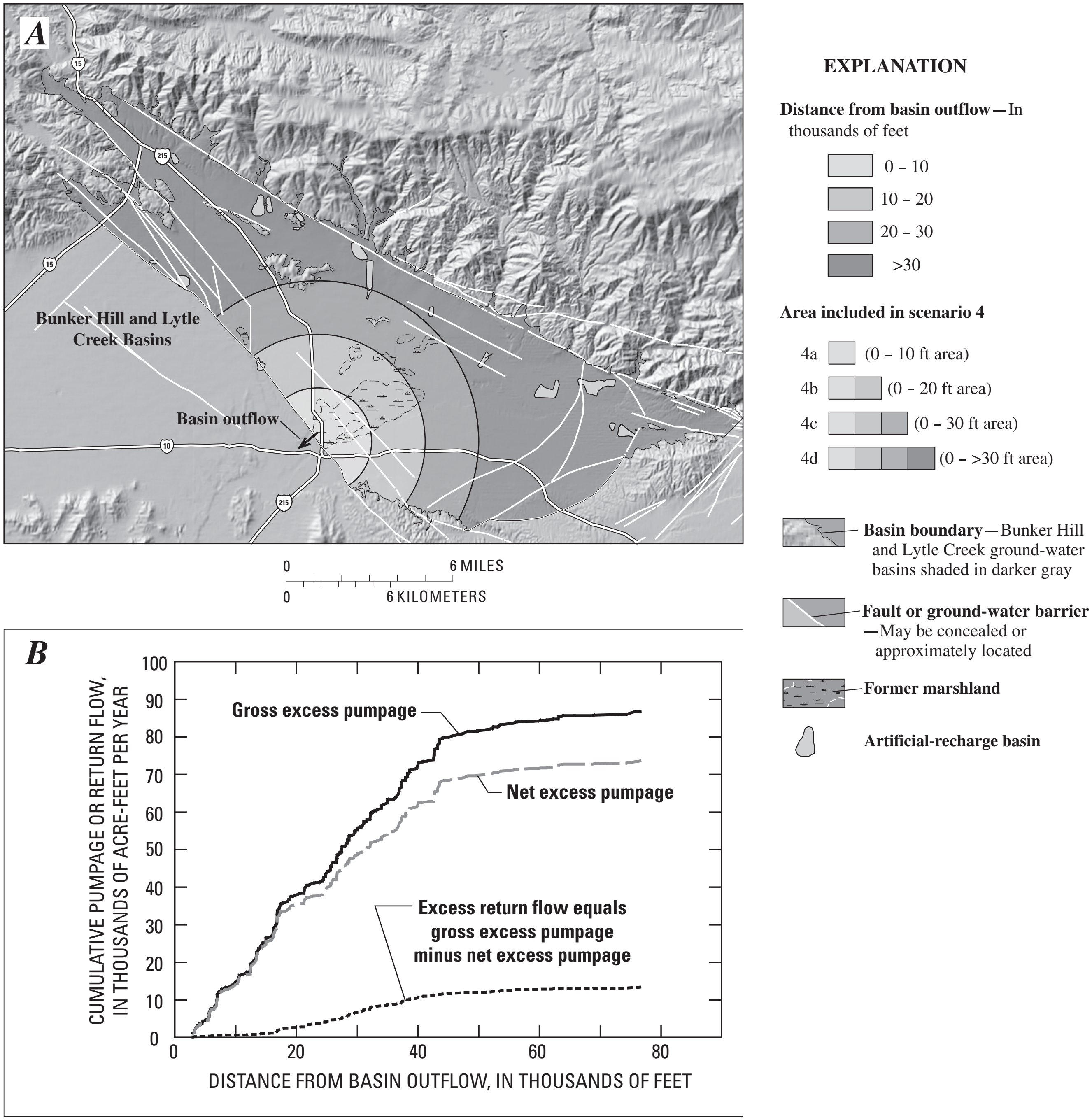


Figure 60. Excess pumping capacity in the San Bernardino area, California, 1994–98. Graph *A* shows that each component of excess capacity (gross and net pumpage and return flow) increases from basin outflow, defined as the midpoint of ground-water underflow across the San Jacinto fault (figs. 22 and 39). Map *B* shows four areas which are defined by distance from basin outflow and are used to formulate scenarios 4a–d.

For scenario 4b, with a net pumpage of about 35,000 acre-ft/yr, the decline in simulated heads is precipitious throughout nearly the entire valley-fill aquifer. Only beneath the upper reaches of the Santa Ana River, Mill Creek, Lytle Creek, and Cajon Creek do heads remain unchanged from conditions prior to 1999 (*fig. 59*). Large annual fluctuations continue even in these areas, but the deflections are similar to those that occurred during the 1980's and 1990's. Beneath the fomer marshlands, the simulated decline in head appears likely to remedy any concern about high ground-water levels. A caveat to this simulated result, however, is that the fine-grained hydrogeologic unit UCM (*fig. 24*) may create perched water-table conditions, even as head in the underlying coarse-grained hydrogeologic unit UWB declines. The presence of this condition can be identified and monitored through the use of multiple-depth piezometers (*figs. 24* and *35*).

The maximum simulated decline for scenario 4b also warrants caution relative to renewed land subsidence caused by compaction of the aquifer system. In many parts of the valley-fill aquifer, the simulated head for scenario 4b declines below the minimum ground-water level reached in about 1965. Land subsidence resulting from withdrawal of ground water is a complex process that begins slowly and once initiated may be difficult and time-consuming to stop or reverse. Monitoring equipment including multiple-depth piezometers and extensometers, and remote-sensing data including interferometric synthetic aperture radar (InSAR) can be useful to detect, map, and analyze aquifer-system compaction (Galloway and others, 1998).

An additional caveat about declining ground-water levels is that scenario 4d may not be a physically realistic simulation. The head decline far exceeds any that occurred during the calibration period. In fact, it is likely that the actual decline would be greater as the aquifer is dewatered and transmissivity and storage values are reduced from those used in the ground-water flow model.

Part of the rationale for scenario 4 was to evaluate the capability of the valley-fill aquifer to satisfy an increasing demand for municipal water. Based on the results shown in *figure 59*, extra net pumpage of more about 14,000 acre-ft/yr cannot be sustained without additional recharge of either native or imported water. Clearly, the extra net pumpage in scenario 4b (35,000 acre-ft/yr) cannot be sustained without additional recharge. This result infers that if a sizeable percentage of future municipal demand, estimated to be as much as 50,000 acre-ft/yr, is to be met using ground water, then some additional recharge is needed if ground-water levels are to be maintained above historical minimums.

Scenario 5: Optimal Hydraulic Containment of Contaminated Ground Water in the Newmark Area

Scenario 5 uses the ground-water flow and constrained optimization models to determine the minimum pumpage necessary to prevent further movement of contaminated ground water away from the Newmark area, designated by the U.S. Environmental Protection Agency as a superfund site (*fig. 61*). The water-management plan designed to accomplish this goal consists of installing several production wells along the leading edge of the contaminated ground water. The idea is that pumping from these wells will induce all contaminated ground water to flow toward the wells, in effect creating a hydraulic barrier preventing further contamination of the valley-fill aquifer. Water extracted from the wells then will be treated and distributed for municipal use. The purpose of this scenario is to determine whether this plan is likely to succeed, and if so, how the wells can be operated optimally.

The water-management plan was developed by the U.S. Environmental Protection Agency in order to mitigate the ground-water contamination related to the Newmark site. The plan was designed by URS Corporation (1993) from field data and from use of a site-specific ground-water flow model (URS Corporation, 1991a,b). The location of the planned wells, referred to locally as the Newmark barrier wells, is along the south edge of the contaminated area as defined in 1998. The plan includes actual sites and design criteria for the five planned barrier wells shown in *figure 61*. Because no contamination was found in the upper hydrogeologic units near the south edge of the contamination, each barrier well is designed to extract ground water only from the middle and lower hydrogeologic units. These units correspond to the lower layer of the ground-water flow model described in this report. The stratigraphy, hydrogeologic units, and model layers in the vicinity of the Newmark contamination site are shown on section *A–A´* (*figs. 23*, *24*, and *37*). An additional four potential sites, each having a single well, were included in the optimization model in order to identify possible operational improvements from having additional barrier wells (*fig. 61*).

The optimization model is formulated to minimize total pumpage from the nine barrier wells subject to control of the simulated head gradient along the edge of the Newmark contamination site. Each well is assumed to have a maximum extraction rate of 3.5 ft^3/s, a rate typical of other production wells in the area. None of the wells has any recharge capability. No ground water is extracted from wells located within the contaminated area. All other recharge and discharge is assumed to be unmanaged—simulated by the ground-water flow and optimization models, but not controlled.

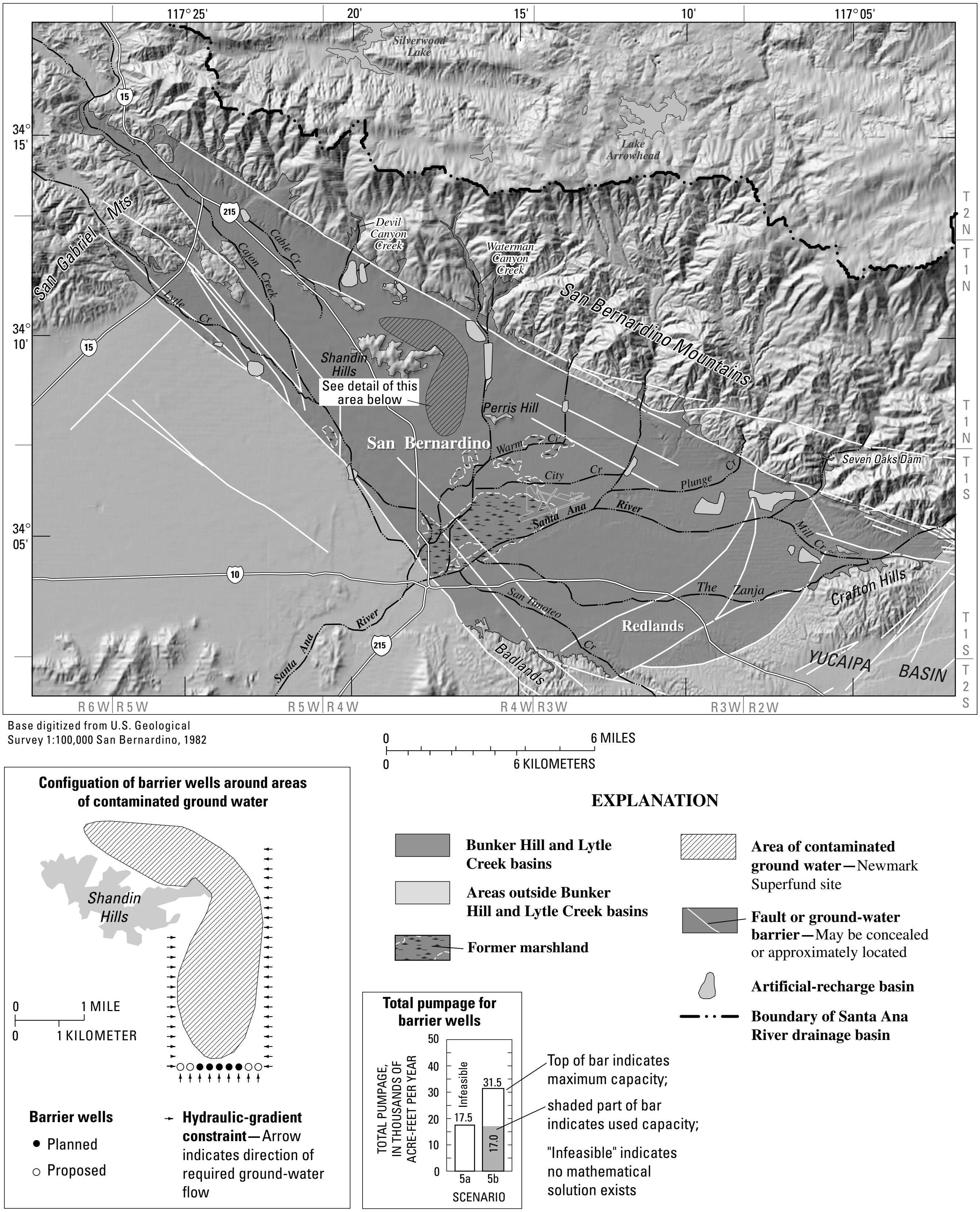


Figure 61. Results from water-management scenario 5 in the San Bernardino area, California, 1999–2030. Scenario 5a uses the 5 planned wells; scenario 5b uses the 5 planned and 4 proposed wells. Scenario 5a uses all hydraulic-gradient constraints; scenario 5b uses only the 9 hydraulic-gradient constraints along the southern boundary.

Gradients are constrained for about one half of the site as shown in *figure 61*. Ground-water flow through the northern half of the site is assumed to be restricted sufficiently by the San Bernardino Mountains and Shandin Hills, which represent impermeable bedrock boundaries. The optimization model requires that the simulated head gradient along the edge of the site slopes in toward the contaminated ground water. Only gradients in the lower model layer are included as constraints because only this layer is believed to be contaminated. The precise mathematical formulation of the optimization model minimizes total pumpage from the barrier wells (eq. 23) subject to annual constraints on hydraulic gradients (eq. 31) and bounds on pumpage (eq. 36).

As with the other water-management scenarios, a 32-year simulation period (1999–2030) was used for scenario 5. To simulate likely future hydrologic conditions, the values of recharge and pumpage used in scenario 2 also were used in scenario 5. The only difference from conditions described for scenario 2 is that during 1999–2030, no pumpage is permitted for any production well within the contaminated area and pumpage from each barrier well is assumed to range from zero to its maximum value.

Scenario 5 has a total of 288 decision variables (9 well sites times 32 time periods) and 1,472 constraints (46 gradient locations times 32 time periods). Each decision variable also has an upper and lower bound, which adds another 576 constraints (288 decision variables times two bounds). A total of ten MODFLOW simulations were required for scenario 5 (nine decision variables plus one unmanaged condition) to create the necessary response functions (*figs. 49* and *50*).

Results from scenario 5 are shown in figure 61. The most important result is that the maximum pumping rate of 3.5 ft^3/s for each of the five planned barrier wells is insufficient to satisfy the specified gradient constraints (scenario 5a, *fig. 61*). Gradients along the east and west sides of the contaminated area cannot be controlled with the five wells, nor can gradients at the far east and far west edge of the southern boundary. Only by using all nine wells can the gradient along the entire southern edge be reversed, toward the contamination (scenario 5b, *fig. 61*).

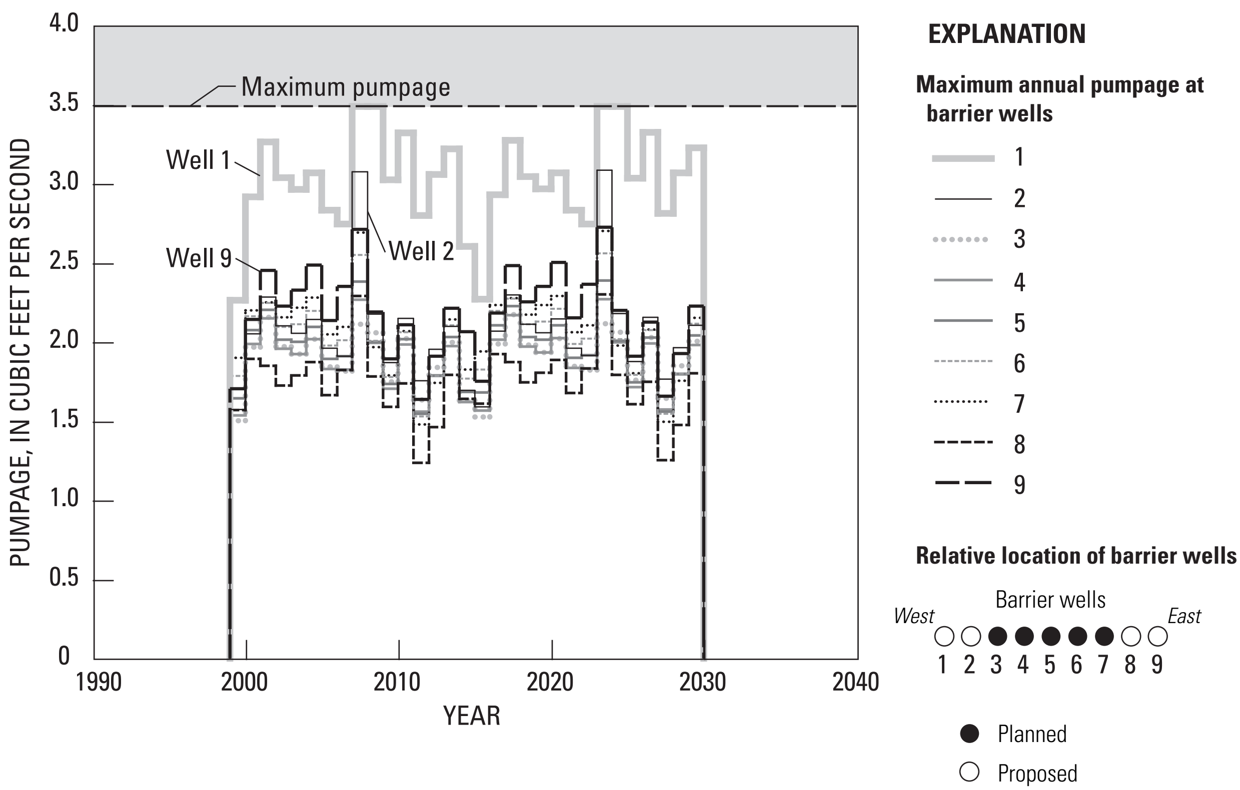


Figure 62. Minimum annual pumpage needed to control hydraulic head gradients along the southern edge of contamination in the Newmark area, San Bernardino, California, 1999–2030. The westernmost well reaches the maximum annual pumpage of 3.5 cubic feet per second during four years.

The minimum total pumpage from the nine wells is about 435,000 acre-ft for the 32-year period, or about 2.1 ft^3/s per well. Minimum annual pumpage for each well is shown in *figure 62*. The fluctuations in minimum pumpage mirror fluctuations in recharge: during years of high runoff, pumpage is low; during years of low runoff, pumpage is high. This pattern is the opposite of what was expected. The increase in head to the north as a result of recharge is not as much a determinant of optimal pumpage as the decrease in head to the south as a result of less recharge and greater pumpage from other wells during years of low runoff (*figs. 25* and *31*).

Minimum pumpage at most wells is nearly the same, fluctuating from about 1.5 ft^3/s to about 2.5 ft^3/s. In contrast, minimum pumpage at the westernmost well fluctuates from about 2.5 ft^3/s to the maximum capacity of 3.5 ft^3/s, and exceeds about 2.7 ft^3/s for most of the time. Pumpage at this well is a binding constraint, meaning the optimal solution is most sensitive to head gradients near the well. Installing an additional production well further to the west likely will improve control of the southern boundary and likely will reduce the total pumpage required from all nine wells.

Further use of the optimization model determined that total pumpage from the five planned wells would need to exceed 1,480,000 acre-ft for the 32-year period, or about 12.8 ft^3/s per well, in order to satisfy all gradient constraints along the southern edge of the contaminated area. This solution is physically unrealistic for a line of production wells in the Bunker Hill basin. Such high rates would result in excessive in-well drawdown and could induce additional land subsidence. Although this is an unrealistic solution, it illustrates the magnitude of the water-management problem. Ground water flowing east and then south around Shandin Hills tends to diverge, partially bypassing the line of barrier wells (*fig. 61*). Wells to the east and west of the contaminated area (*fig. 31*) tend to pull the contamination in those directions. Both the high transmissivity of the valley-fill aquifer and the fluctuating recharge and pumpage make the problem hydraulically difficult, as illustrated by these optimization results.

Neither solution presented in *figure 61* can adequately control head gradients along the east and west sides of the contaminated area using either five or nine wells. To better undertand this result, 38 test cases were developed using the basic formulation of the optimization model for scenario 5. Different combinations of head gradients and pumpage capacity were used to identify a range of optimal solutions, or more precisely a suite of infeasible solutions. Most test cases were infeasible, indicating that control of the contamination boundary in all locations for all years is an elusive goal. If hydraulic control of the plume is possible in the real world, it will not be easy using only the nine wells shown on *figure 61*.

A more common design of hydraulic control of ground-water contamination is to install wells to pump from within the contaminated area. Because of the difficulty identified by scenario 5 to hydraulically control contamination in the Newmark area, modifying design of the cleanup strategy to include extraction wells within the contaminated area may produce more effective hydraulic control of the contamination. Alternatively, installation of monitoring wells along the boundaries, particularly at the southwestern edge of the contamination, will help determine if contaminated ground water does evade the five barrier wells.

Part of testing scenario 5 was evaluating the difference between results from the linear constrained optimization model and the slightly non-linear ground-water flow model. Optimal pumpage from scenario 5b was simulated with the ground-water flow model. The resulting head gradients from the flow model were compared to head-gradient constraints from the optimization model. Any difference between the models is caused by a combination of numerical roundoff in both models and hydraulic non-linearities in the flow model. Although the absolute value of the heads was slightly different, the head gradients were virtually the same in both models. This similarity suggests that the hydraulic non-linearities (head-dependent stream recharge, evapotranspiration, and underflow) in the ground-water flow model are sufficiently distant from the barrier wells that any difference is muted by the relatively high transmissivity of the valley-fill aquifer.

Use of the optimization model in scenario 5 illustrates how defining the feasible and infeasible regions (*fig. 46*) can be useful in gaining a better understanding of a water-management problem. Infeasible solutions typically highlight a critical hydrogeologic aspect of a water-management problem—an aspect that may or may not have been recognized from the outset as a controlling feature. In the case of scenario 5, this critical hydrogeologic aspect is the difficulty of controlling a dynamic boundary in a highly permeable aquifer.

Scenario 6: Optimal Pumpage Using New Wells to Control Ground-Water Levels in the Former Marshland

Scenario 6 uses the ground-water flow and constrained optimization models to identify the minimum pumpage necessary to control ground-water levels in the former marshland of the San Bernardino area. The additional pumpage comes from new production wells that could be installed along three proposed extensions of the Baseline feeder pipeline (*fig. 63*). The water-management objective for scenario 6 is not only to prevent the adverse effects of high ground-water levels, such as the historical damage caused by elevated hydrostatic pressure and the potential damage that could be caused by liquefaction during a large earthquake, but also to prevent the adverse effects of low ground-water levels, such as the possible reoccurrence of land subsidence.

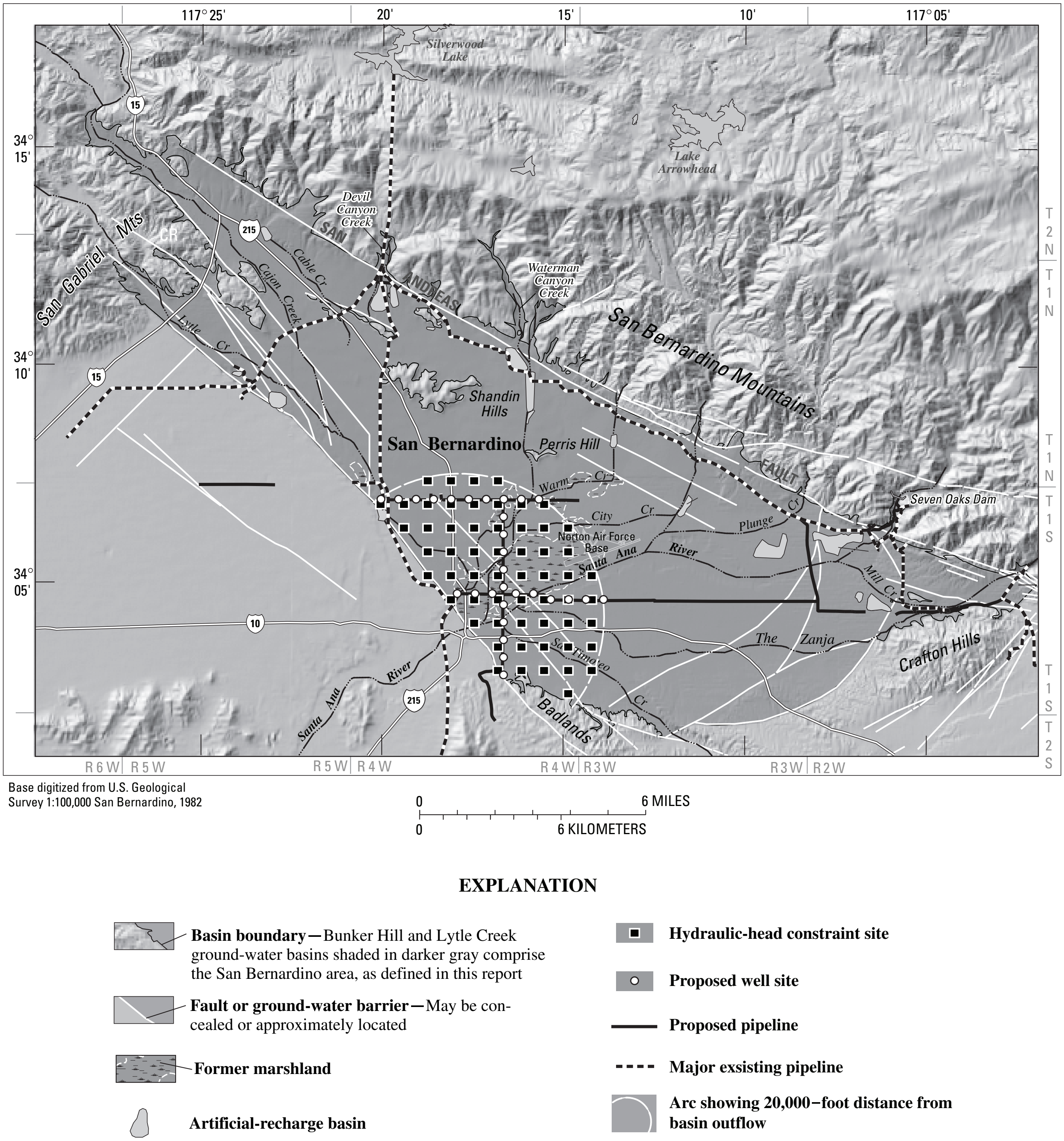


Figure 63. Design of water-management scenario 6 in the San Bernardino area, California, 1999–2030.

The area of the Bunker Hill and Lytle Creek basins that has generated the greatest concern about extreme ground-water levels is near the former marshland. Since 1945, hydrostatic pressure in this area has been sufficiently high at times (late 1940's, early 1980's) to cause damage to public infrastructure, and sufficiently low at other times (mid 1960's) to cause land subsidence of more than 1 foot (*figs. 43* and *55*). Control of ground-water levels throughout this area is achieved in scenario 6 by a uniform grid of hydraulic-head constraint sites shown in *figure 63*. Each site represents an upper and lower cell in the ground-water flow model, which is used to calculate the effect of pumpage on simulated hydraulic head. The effects of initial and boundary conditions and of all unmanaged stresses also are calculated by the flow model. The unmanaged stresses in scenario 6 are the same as those used in scenario 2. As in scenarios 1–5, the same 32-year management period, 1999–2030, is used.

For each of the 32 years, simulated head in the upper model layer at each of the constraint sites is required to be at least 30 ft below land surface, the depth necessary to prevent liquefaction and to avoid damage to subsurface structures from elevated hydrostatic pressure. Because elevated head in the lower model layer does not by itself cause hydrostatic damage, it is not restricted. To prevent additional land subsidence, head in both the upper and lower model layers at the constraint sites is required to be above the respective minimum simulated head during 1945–98, the calibration period for the ground-water flow model. These minimum heads are assumed to represent the pre-consolidation heads that probably were reset as a result of the significant decline of ground-water levels between 1945 and 1970.

As part of the overall water-management plan developed by Camp, Dresser, and McKee (1995a), three possible extensions of the Baseline feeder pipeline were identified. The first, referred to as the "9th Street feeder east," continues east on 9th Street to Sterling Avenue. The second, referred to as the "South end feeder," continues south from the first extension, down Arrowhead Avenue past the San Jacinto fault. The third, referred to as the "Central feeder," bisects the South end feeder at Orange Show Road. A generalized pattern of possible production wells was designed to accompany each extension (*fig. 63*). A total of 29 possible new well sites were chosen, each to be about 2,460 ft apart, a distance equivalent to three model cells and three times the distance between barrier wells described in scenario 5. The goal in choosing the well sites was to provide a large number of high-capacity wells that would provide an abundant pumping capacity and the necessary well interference needed to control ground-water levels over a large area. The new well sites were located adjacent to the major pipelines in order to limit the cost of installing any additional pipe needed to convey water from the new wells to the major pipelines. New well sites also were restricted to the general area of the former marshland, defined as within a distance of 20,000 ft from basin outflow. The same approximation was used as part of scenario 4 (*fig. 60*).

All water extracted from the 29 well sites was assumed to be pumped directly into the respective pipelines, or if necessary, treated or blended prior to being added to flow in one of the pipelines. The eventual use of the pumped water was not defined; however, the additional pumpage was assumed not to replace any existing pumpage in the Bunker Hill or Lytle Creek basin, nor to contribute any return flow to any part of these basins. Because the objective of the dewatering sites is to control shallow ground-water levels, each well was assumed to be perforated only opposite hydrogeologic units UCM and UWB, which are represented as the upper layer of the ground-water flow model (*fig. 37*). The maximum pumping rate at each site is assumed to be 2.2 ft^3/s, a sustainable pumping rate commonly achieved by nearby existing wells.

The mathematical formulation of the optimization model for scenario 6 minimizes the sum of total pumpage from the dewatering wells (eq. 23), subject to constraints on ground-water levels (eqs. 28a, b) and bounds on pumpage (eq. 36). This basic initial formulation was chosen for analysis in order to investigate the hydraulic characteristics controlling the solution to scenario 6. A more complex formulation could have minimized cost of additional pumpage, subject to the same constraints and bounds. Although cost is a commonly used objective in operations research, it can disguise important hydrologic characteristics of the problem and assigning costs can be highly controversial. If analysis of scenario 6 were to be expanded to include cost, important considerations would be both the fixed cost of installing the wells, likely to be about $1 million each, and the variable cost of pumping each well. Quadratic and integer programming techniques can be used to solve this type of optimization problem (Hillier and Lieberman, 1980, p. 714–755).

Ground-water levels were constrained within the general area of the former marshland, using the same 20,000-ft distance from basin outflow used to locate the possible new well sites (*fig. 63*). Ground-water-level control sites were set at every fifth model cell within the 20,000-ft distance. This spacing was chosen to balance the need to ensure simulated ground-water levels were controlled within the former marshland, with the desire to reduce the total number of constaints in the optimization model. Although computationally more time-consuming, every model cell in the former marshland, or in the entire model domain, could have been used.

Scenario 6 has a total of 928 decision variables (29 well sites times 32 time periods) and 5,568 constraints (58 ground-water-level control sites times 3 ground-water levels times 32 time periods). Each decision variable also has an upper and lower bound, which adds another 1,856 constraints (928 decision variables times two bounds).To obtain the response information for scenario 6, thirty simulations of the ground-water flow model were required: one for each managed well and one for the unmanaged condition.

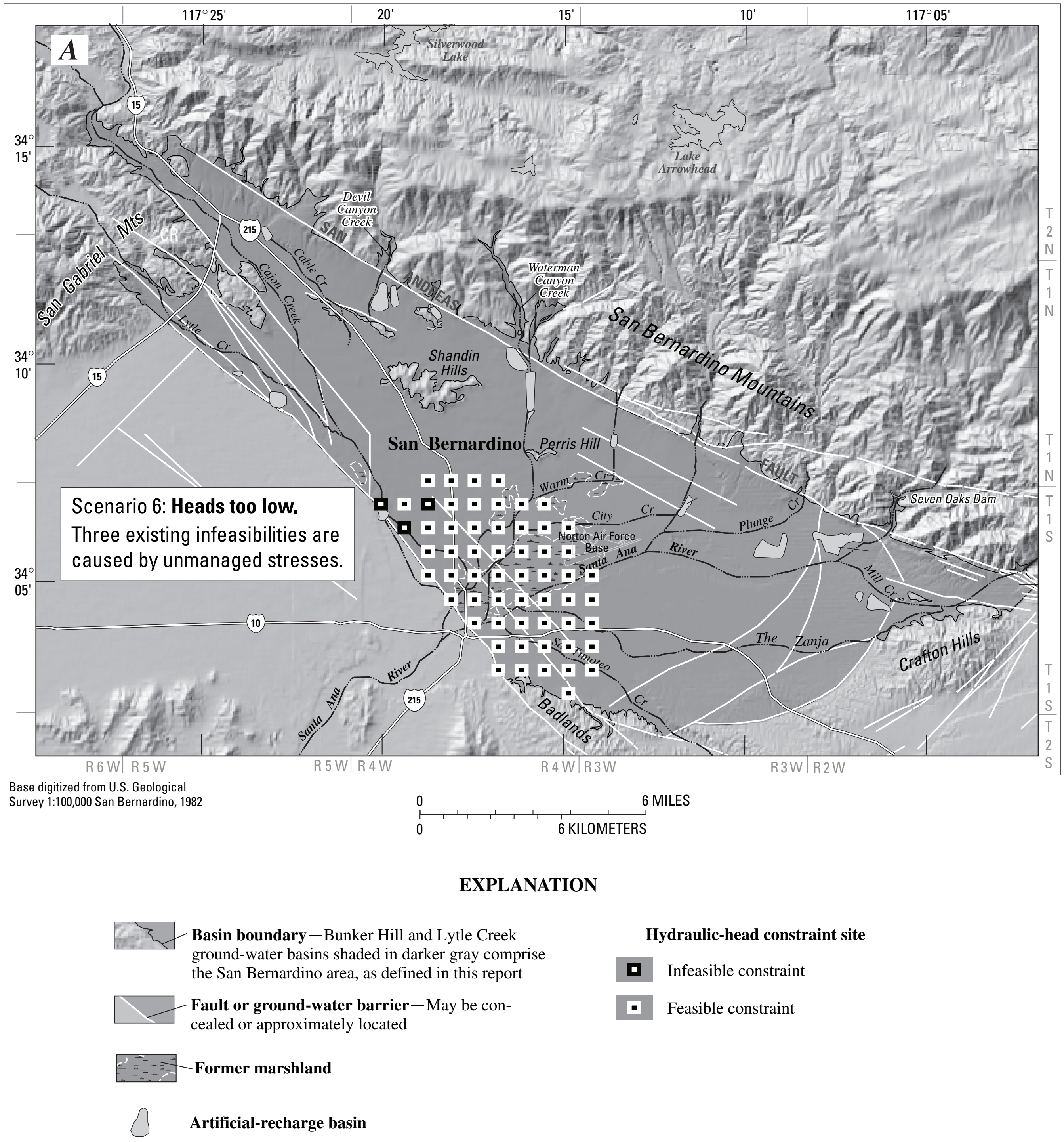


Figure 64. Infeasible head constraints for scenario 6. *A*, infeasibilities caused by unmanaged stresses. *B*, infeasibilities caused by attempting to decrease ground-water level to 30 feet below land surface, and *C*, temporal distribution of infeasibilities in *B*.

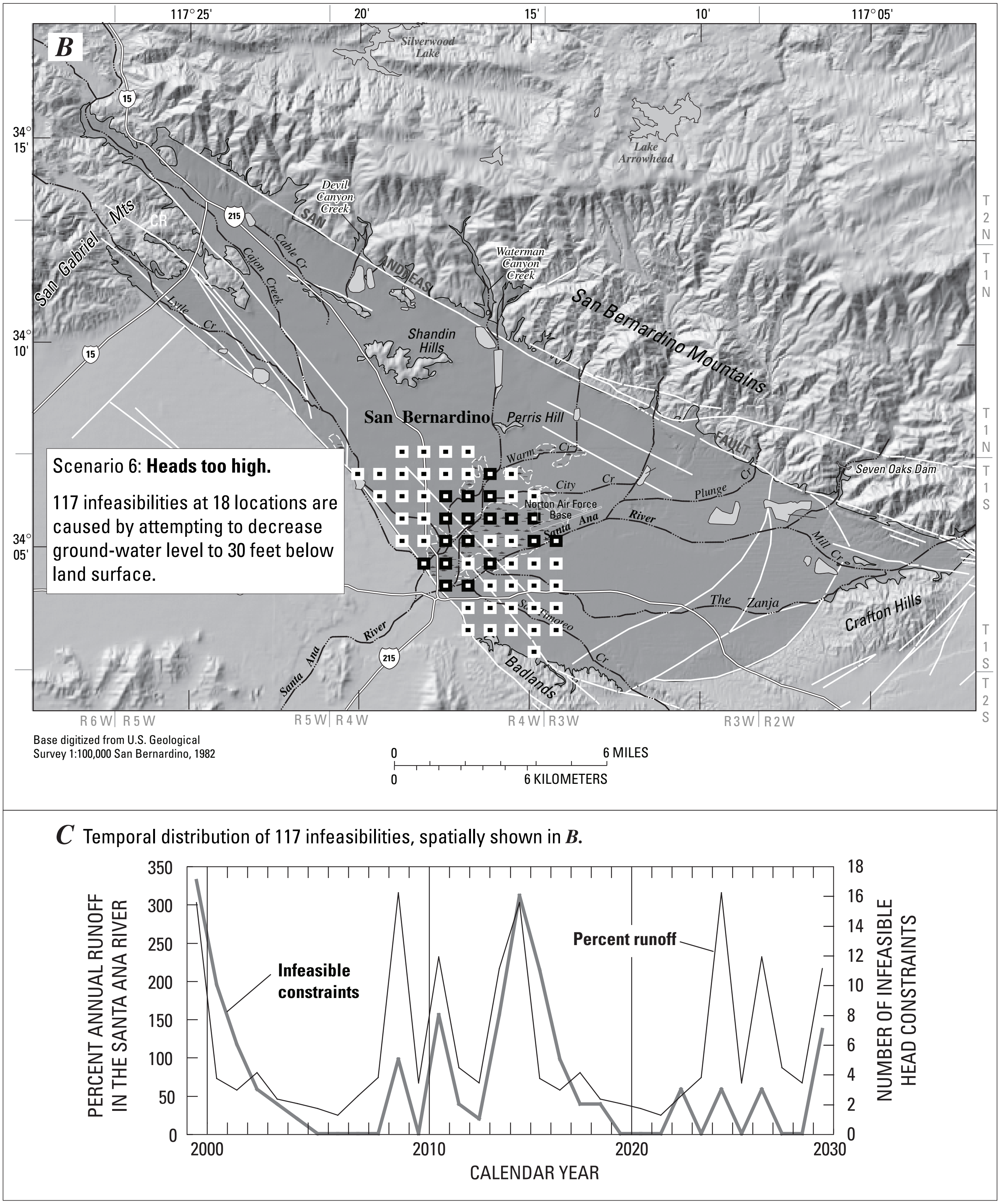


Figure 64.—Continued.

Although formulating scenario 6 was relatively simple, finding a feasible solution was not. Tens of model runs using MINOS identified hundreds of infeasibilities, and in the end no feasible, much less optimal solution was found. Most knowledge acquired from scenario 6 came from successive attempts to remove infeasiblities and progressively learning more about the hydraulic or hydrologic reason for the persistently infeasible solution.

Results from scenario 6 indicate numerous hydrologically important findings. First, in the area of Lytle Creek, recent new pumpage appears to have lowered simulated heads below any simulated value during 1945–98. This result suggests that additional pumpage without sufficient additional recharge will result in ground-water levels below the constraint to prevent land subsidence (*fig. 64A*). Whether actual land subsidence will occur is unknown, but these infeasible head constraints are in the same area as historical land subsidence (*fig. 55*).

Second, the area where ground-water levels remain too high coincides remarkably well with the former marshland (*fig. 64B*). As noted by previous researchers, ground water accumulates, especially in wet years, in the vicinity of Warm Creek. This hydrologic process was referred to as “rising ground water” by Hardt and Hutchinson (1980), who identified the impending problems of high ground-water based on predictive simulations with a previous ground-water flow model. Actual damages caused by high ground-water levels began occurring in about 1979 and continued until a sequence of years with less runoff and less imported water resulted in falling ground-water levels from about 1984 to 1992. The location of these infeasible head constaints is the same location of high ground-water levels that caused infrastructure problems, prompting the lawsuit in 1986.

Third, fluctuating recharge and pumpage during 1999–2030 make control of ground-water levels challenging. These fluctuations are essentially the background response from scenario 2. Ground-water levels in many years are adequately controlled with zero additional pumpage or with the capacity and configuration of wells in scenario 6. In some wet years, however, too much recharge arrives and the 29 proposed wells cannot overcome this rise in ground-water levels. The number of infeasible head constraints per year compared to the percent runoff for the Santa Ana River is shown in *figure 64C*. In general, years with more runoff have more infeasibilities. A subtle aspect of this result is that prior pumpage in drier years is not sufficient to adequately reduce the rising ground-water levels in wet years. Water managers will not know future conditions, but the optimization model has knowledge of impending wet years, and still cannot solve the problem.

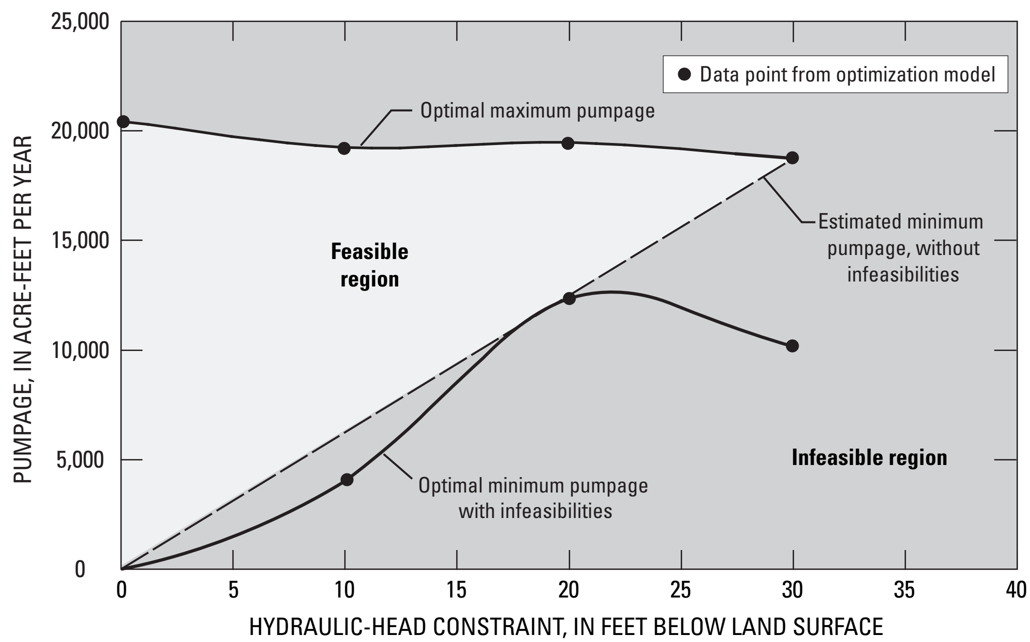


Figure 65. Minimum and maximum pumpage for scenario 6 for the San Bernardino area, California. The apparent decrease in optimal minimum pumpage is caused by omitting infeasible head constraints from the optimization model. The actual minimum pumpage required to lower high ground-water levels, as indicated by the dashed line, is likely to be somewhat greater than that predicted by the optimization model. The maximum pumpage that can be obtained without having ground-water levels fall too low is approximately 20,000 acre-feet per year.

To obtain additional information from scenario 6, all infeasible head constraints were eliminated from the optimization model. Without the four infeasible (too low) head constraints at three locations (*fig. 64A*) and the 115 infeasible (too high) head constraints at 18 locations (*fig. 64B*), optimal solutions were obtained for several test cases. The minimum pumpage required to maintain head in the upper model layer at least 0, 10, 20, or 30 ft below land surface is shown in *figure 65*. In each test case, all constraints relating to land subsidence were satisfied, except the four that were deleted (*fig. 64A*). The unusual shape of the curve of minimum pumpage likely results from the progressive omission of infeasible constraints. A more likely linear shape might result if all infeasible (too high) constraints were kept in the optimization model and adequate areally distributed pumpage capacity were simulated. If this linear relation is valid, the minimum value of pumpage needed to lower simulated heads in the upper model layer to 30 ft below land surface is about 18,000 acre-ft/yr.

Testing also identified the maximum pumpage that could be obtained from the 29 wells without violating head constraints designed to prevent additional land subsidence. Optimization results suggest that an average of about 20,000 acre-ft/yr of additional pumpage could be obtained from the San Bernardino area during the 32-year management period used for scenario 6 (*fig. 64C*).

In summary, however, pumpage quantity and spacing in scenario 6 is insufficient to solve the slowly evolving high ground-water problem—despite having more than 70,000 acre-ft of installed capacity at 29 separate well sites along the three proposed extensions of the Baseline feeder. This result from the optimization model suggests that careful monitoring of ground-water levels is needed to identify any longterm trends. As soon as an upward trend is identified, additional pumpage needs to be used to control the rising ground water because as found during 1978–84, high ground-water levels in the San Bernardino area cannot be lowered significantly in a couple of months or couple of years. The inability of the 29 wells proposed for scenario 6 to solve the rising ground-water problem over a 32-year period highlights the need to use a more areally extensive network of dewatering wells, such as illustrated by results from scenario 4.

Discussion

These seven water-management scenarios illustrate some of the more important characteristics of the hydrologic system in the San Bernardino area. Runoff from the adjacent San Bernardino and San Gabriel Mountains provides abundant recharge in an otherwise semiarid region. This recharge, in addition to ground-water pumpage that comprises essentially all ground-water discharge, causes significant, continuous fluctuations in ground-water levels and ground-water storage, as illustrated by scenarios 2, 3, and 4. The large areal size and high specific yield of the valley-fill aquifer create long-term trends in ground-water levels and storage that last for decades, as illustrated by scenarios 1 and 2. Both the short-term, detectable fluctuations and the longterm, less detectable trends make water-management decisions more difficult than they would be in a smaller, less hydraulically dynamic ground-water basin.

For example, the installed pumpage capacity necessary to control ground-water levels needs to be larger than if the aquifer storage and fluctuations in aquifer storage were less. Perceiving long-term trends is made more difficult by the widely varying conditions that occur over a period of a few years. Because of this difficulty in perceiving trends from measured data, simulations using the ground-water flow model can be both instructive and important to illustrate the wide-ranging and long-term implications of water-management decisions.

Scenarios 1 and 2 clearly indicate that ground-water levels in most of the San Bernardino area will remain similar to levels experienced during 1983–98 if similar climatic conditions occur and 1998 water-management operations are continued. This conclusion is tempered by results from scenario 4, which illustrate that different assumptions about pumpage can result in a progressive decline in ground-water levels. Even during this decline, however, annual fluctuations in ground-water levels may pose a threat of additional damage from elevated hydrostatic pressure or from liquefaction during an intense earthquake.

Results from scenarios 2, 3, and 4 suggest that additional pumpage is needed to control ground-water levels, but that the magnitude of the pumpage needs to be evaluated annually. Scenario 4 indicates that the capacity of existing wells is sufficient to accomplish this control; however, this physical capacity was not used during 1978-84 when rising ground-water levels caused extensive damage to foundations, basements, and flood-control channels. New pipelines, such as the three proposed extensions to the Baseline feeder, may aid in conveying additional pumpage to a place of beneficial use, but scenario 6 indicates that areally distributed pumpage is needed, not just extra pumpage from wells located along the new pipelines. A related finding from scenario 5 indicates that the high transmissivity of the valley-fill aquifer makes forceful control of ground-water levels or gradients difficult to achieve using only a few sites. A broader-scale solution probably is necessary, though it may be logistically and politically more difficult.

For the past 50 years, native recharge to the valley-fill aquifer has been sufficient, sometimes even excessive. If urbanization of the San Bernardino area continues and local demand for water increases as projected, then availability and use of imported water becomes critical. As indicated by scenario 4, the valley-fill aquifer cannot supply more than about an additional 10,000 acre-ft of ground water unless the recharge rate of native runoff is increased or water imported from outside the San Bernardino area is artificially recharged to the valley-fill aquifer. The great distance from the artificial-recharge basins to the production wells and the high specific yield of the valley-fill aquifer create a long response time from the largest basins on the Santa Ana River and Mill Creek to the area of most production wells. This hydraulic characteristic can be an aid in effective management of both native and imported water. Whenever inexpensive water becomes available, in effect, it can be stored in the valley-fill aquifer while in transit to the area of the production wells.

The primary caveat in this approach is that sufficient production eventually needs to extract the recharged water or high ground-water levels will result. As indicated from scenario 4, the quantity, timing, and location of the extraction need to be determined carefully so that ground-water levels do not decline and cause land subsidence. The much shorter response time from other artificial-recharge basins, such as the Waterman basin, suggests that use of these basins may be helpful in making short-term adjustments to declining ground-water levels in the area of the production wells.

Attempts to control migration of the Newmark contamination present multiple challenges as indicated by results from scenario 5. These challenges relate to the hydraulics of the simulated ground-water flow system and are compounded by dispersion of the contaminates and a more complex three-dimensional aquifer structure than that simulated with the ground-water flow model. The hydraulic challenges illustrated in this study include the high transmissivity of the valley-fill aquifer that makes creating and maintaining a hydraulic barrier difficult during fluctuating conditions of runoff, recharge, and pumpage. Also, the proximity of the Newmark barrier wells to other areas of major pumpage and to the former marshland can create a conflict in achieving both hydraulic control of the Newmark contamination and hydraulic control of ground-water levels that are either too high or too low. Resolution of this conflict may require some modification to the strategy that uses only barrier wells to control migration of the Newmark contamination.

Developing additional scenarios utilizing one or both of the ground-water flow and optimization models likely will result in additional insights about the valley-fill aquifer and about the interaction among the several conflicting water-management issues. Development of a scenario, however, often is not a quick and easy exercise despite its apparent simplicity. Consistency needs to be achieved among the many recharge and discharge components, between historical data and future projections, and between water-management concepts and numerical representation of those concepts. The payoff for the substantial effort in developing a well thought out, numerically robust scenario is hydrologic insight that would be difficult to gain in any other way.

Development and use of ground-water flow models is now common throughout the world and is an important part of gaining hydrologic insight, but numerical convergence of large, complex flow models, such as this model of the San Bernardino area, is an ongoing concern (Kuniansky and Danskin, 2003). A lack of convergence can occur in solving any of the tens of thousands of timesteps needed to prepare response simulations for an optimization model. These numerical errors, in turn, can cause uncertain results in the optimization model and may require re-formulation of the water-management scenario.

Computational time and accuracy required to solve a scenario combining ground-water flow and constrained optimization models mostly is determined by the flow model. For example, in scenario 6, calculating the 29 response functions required about 15 hours; solving the optimization model required about 1 minute. This disparity of time is typical of ground-water management problems and illustrates the need to have an efficient flow model. Total computational time for scenarios similar to those in this report will be determined by the number of decision variables requiring a response function, by the number of management time periods, and by the complexity and required numerical accuracy of the flow model.

An optimization model like a good ground-water flow model, needs to be as simple as possible. This goal of parsimony was central in designing scenarios 5 and 6. As observed by H.M. Wagner (1975, p. 6), the relatively simple process of designing an optimization model can result in a model that is too complex to critique or interpret. At the other end of the spectrum, having too few decision variables can produce an optimization model with too few degrees of freedom and essentially no feasibility space (*fig. 46*). Design of an effective optimization model is as much art as science, achieving a balance between these potential problems.

Instructive management scenarios can be formulated by using just the ground-water flow model as in scenarios 1–4, or by using the combined flow and optimization models as in scenarios 5 and 6. An important difference exists between these two approaches. Results from a flow-model scenario such as in scenario 4 commonly are observed and interpreted with the goal of identifying general trends. In contrast, results from an optimzation model require 100 percent adherence to specific requirements. In scenario 6, each of the 7,424 constraints must be satisfied, not just most, or nearly all. This provides an important advantage in use of a optimization model compared to a ground-water flow model to inspect the results of a potential water-management alternative. But it also requires a conformance that may not be as easy to achieve hydraulically, as it is to say in words during formulation of the scenario.

The experience of developing and applying constrained optimization techniques in the San Bernardino area often went something like this: posing what seems to be a realistic optimization problem based on discussions with knowledgable water managers; finding the problem is infeasible; redesigning the problem to loosen some overly optimistic constraints; finding the problem remains infeasible for another hydraulic reason; loosening or removing additional constraints; possibly continuing these last steps several more times; finding an optimal solution; then progressively tightening some constraints to achieve an instructive, but tightly constrained optimal solution. The eventual benefit of this sequence is much more than a mathematically optimal answer to the problem. Rather, the larger benefit is a dramatically improved understanding of the aquifer system.

Suggestions for Future Work

Several decades of continuing investigations in the San Bernardino area have resulted in a large quantity of data and a much improved understanding of the surface-water and ground-water systems. This understanding has enabled a quantitative analysis of the area using both ground-water flow and constrained optimization models. But, as shown in *figure 66*, applied scientific investigations typically do not follow a linear path from data, to concepts, to models. Rather, they tend to follow an iterative path of learning that involves collection of data, refinement of concepts, and testing of the new data and concepts with improved mathematical models. The models, in turn, prompt new questions about the data and concepts and can be used to define which additional data are most important to refine concepts or to improve predictive capability of the model.

This pattern of iterative investigation applies to water management in the San Bernardino area. The present analysis evolved from use of an initial ground-water flow model that identified critical data deficiencies, to collection of new data, to refinement of hydrologic concepts, to evaluation of data and concepts with improved simulation and optimization models. Future work in the San Bernardino area likely will focus first on improved data collection to validate, extend, or discard existing hydrogeologic concepts. Subsequent studies likely will use modified ground-water flow, solute-transport, and optimization models to test the new data and concepts and to refine present conclusions about water management. Suggestions for this future work are listed below by general topic and are shown by general location in *figure 67*.

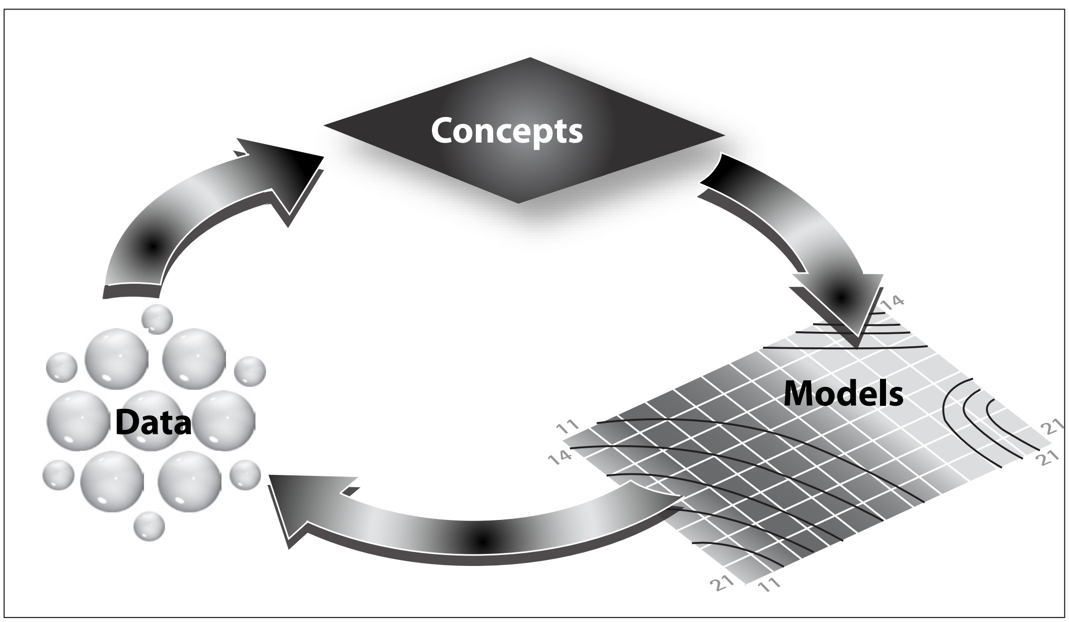
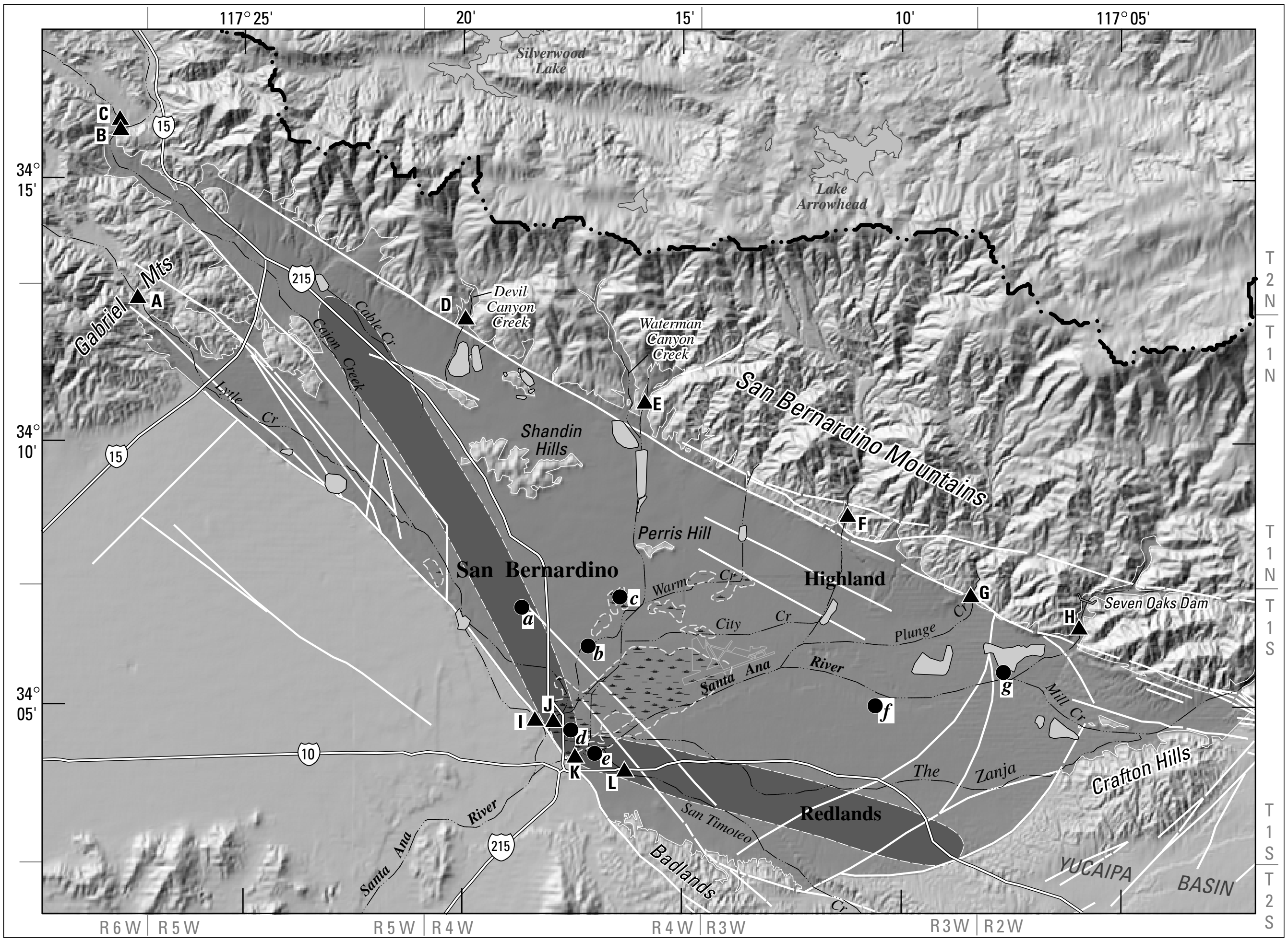


Figure 66. General process of scientific inquiry.



Shaded relief base from U.S. Geological Survey digital elevation data, 1:24,000-scale

0 6 MILES

0 6 KILOMETERS

EXPLANATION

Basin boundary—Bunker Hill and Lytle Creek ground-water basins shaded in darker gray

Priority area for installation of additional multiple-depth wells

Former marshland

Fault or ground-water barrier—May be concealed or approximately located

Artificial-recharge basin

Boundary of Santa Ana River drainage basin

▲ **Continuous gaging station**

A	11062001	Lytle Creek near Fontana
B	11063510	Cajon Creek below Lone Pine Creek, near Keenbrook
C	11063500	Lone Pine Creek near Keenbrook
D	11063680	Devil Canyon Creek near San Bernardino
E	11058500	East Twin Creek near Arrowhead Springs
F	11055801	City Creek near Highland
G	11055501	Plunge Creek near East Highlands
H	11051501	Santa Ana River near Mentone
I	11065000	Lytle Creek at Colton
J	11060400	Warm Creek near San Bernardino
K	11059300	Santa Ana River at E Street
L	11057500	San Timoteo Creek near Loma Linda

● **Multiple-depth cluster well site**

a	1S/4W-4E3-6	Encanto Park
b	1S/4W-10B2-4	Meadowbrook Park
c	1S/4W-2D6-8	Sierra High School
d	1S/4W-22D2, 4-7	SBVMWD backyard
e	1S/4W-22J1-4	Commerce Center
f	1S/3W-15K1-5	Riverview
g	1S/2W-7Q1-6	Cone Camp

Figure 67. Water-management sites in the San Bernardino area, California, 1998.

Data

1. Measurements of surface-water discharge.—Continued monitoring of surface-water discharge, at least for the Santa Ana River, will provide data needed to use the several predictive temporal relations based on flow in the Santa Ana River that are presented in this report. Continued discharge measurements also will facilitate a critique and an update of the ground-water flow model.

2. Multiple-depth monitoring wells.—Additional multiple-depth monitoring wells will help identify aquifer materials, measure vertical differences in hydraulic head, and quantify vertical changes in ground-water quality. The areas of greatest need are from where the Santa Ana River crosses the San Jacinto fault: (1) past Redlands toward the Yucaipa basin and (2) past Shandin Hills toward the upper reaches of Cajon Creek (*fig. 67*). Based on a similarly stated need in an earlier draft of this report, three multiple-depth monitoring wells (sites e, f, and g on *fig. 67*) were installed and have provided valuable new information.

3. Ground-water-level measurements.—Continuous ground-water-level measurements at multiple-depth monitoring well sites (*fig. 67*) provide valuable data to critique hydrogeologic concepts and vertical accuracy of the ground-water flow model. Semi-annual ground-water-level measurements from existing wells throughout the San Bernardino area provide an effective way to critique horizontal accuracy of the ground-water flow model.

4. Water-quality sampling.—Sampling a broad spectrum of ground-water constituents, including trace elements and naturally occurring isotopes (hydrogen, oxygen, and carbon), from production wells and multiple-depth monitoring wells will help identify the age, occurrence, and movement of ground water within the three-dimensional, valley-fill aquifer. Water-quality data from deeper sediment will facilitate identifying and interpretating interactions between the valley-fill aquifer and the surrounding bedrock. Selected samples of surface-water quality will be important to verify concepts of surface-water/ground-water interaction.

5. Flowmeter logs.—Flowmeter logs collected from as many production wells as possible will help define the relative quantity of water that enters each well from different hydrogeologic units and how the wells connect and transmit ground water between permeable zones within the aquifer system. These data will improve the definition and mapping of hydrogeologic units and will allow for a more accurate vertical distribution of pumpage in the ground-water flow model. Flowmeter logs commonly use an impeller or spinner tool to measure flow inside the well casing. Other techniques, including dye-injection or heat pulse, may be preferrable depending on access to the well and rate of flow inside the well.

6. Land-surface deformation.—Measurements of land-surface deformation will aid in identifying patterns of ground-water recharge and discharge, location and extent of poorly permeable faults, and distribution of fine-grained deposits. Monitoring could use permanently installed, continuously recording extensometers in combination with intermittantly available satellite range data processed using Interferometric Synthetic Aperature Radar (InSAR) techniques.

7. Geographic database.—Organizing the three-dimensional geometry of the valley-fill aquifer in a powerful spatial database, such as a geographic information system (GIS), will facilitate updates and revisions to the conceptual and numerical models. In particular, including the top and bottom of the hydrogeologic units at individuals wells in a spatial database will aid in developing structural contours of the units.

Concepts

8. Depositional history.—The depositional history of sediment constituting the valley-fill aquifer is known only cursorily. Additional information will be helpful to improve hydrogeologic understanding, in particular the location and age of depositional facies.

9. Hydrogeologic units.—Extending the hydrogeologic units and layering of the valley-fill aquifer, as presented in *figure 24*, to other areas of the Bunker Hill and Lytle Creek basins will help identify any inconsistencies in the present hydrogeologic concepts of the aquifer system. This mapping requires understanding the deposition history of the valley-fill aquifer.

10. Upper confining member.—An improved understanding of the upper confining member (UCM, *fig. 24*) in routine recharge water, in creating perched conditions, and in moving water to or from deeper pumped zones is critically important. Entwined with understanding the UCM is understanding the role of open well casings and gravel-packed wells in the vertical movement of ground water.

11. Pumpage return flow.—Use of a constant return flow percentage applied to pumpage from 1945 to 1998, as done in this report, has a strong effect on ground-water flow and budget. Changes in use of wells and changes in land use suggest a more complex spatial and temporal calculation may be warranted.

12. Ground-water flow from bedrock.—Defining the importance of a significantly deeper ground-water flow system through the basement complex (*fig. 24*), as indicated by the presence of geothermal water in several wells (*fig. 55*), will aid in evaluating the conceptual flow model, which presently includes only the valley-fill aquifer.

Models

13. Representing hydrogeologic units. —Updating the ground-water flow model with altitudes of individual hydrogeologic units, thereby enabling the use of convertible unconfined-confined storage coefficients and time-varying transmissivity, will create a more realistic flow model.

14. Underflow.—Underflow out of the ground-water flow model across the San Jacinto fault near Barrier J (*fig. 39*) is not well understood, but is an important part of accurately simulating ground-water flow in the Lytle Creek basin. Resolving discrepancies with underflow used in the ground-water flow model of the Rialto–Colton basin will improve reliability of both models.

15. Land deformation.—A land-deformation (subsidence) package, added to the ground-water flow model, would permit better simulation of aquifer storage properties and periods of significant change in ground-water levels. Historical subsidence of the land surface indicates that some ground-water pumpage was derived from permanent aquifer compaction; this decrease in aquifer storage is not simulated presently in the ground-water flow model.

16. Vertically distributed pumpage.—Automatically proportioning pumpage to different model layers based on changing head during a simulation will significantly improve the ability of the ground-water flow model to simulate land subsidence and advective transport of solutes. Presently, pumpage is proportioned by hand using a constant temporal distribution.

17. Geologic framework model.—A geologic framework model rigorously combines information about geologic formations, sedimentary deposits, faults, and hydrogeologic units. Such a model can be used to critique three-dimensional concepts of geologic structure, depositional history, and movement of ground water. A geologic framework model of the Bunker Hill and Lytle Creek basins would help provide the three-dimensional knowledge necessary to improve vertical resolution of the ground-water flow model or to develop a solute-transport model.

Management

18. Use of models.—Continued use of the ground-water flow and constrained optimization models described in this report can be a useful part of advancing water management of the San Bernardino area. The models are well suited to critique ideas and evaluate specific plans. The ground-water flow model was designed to facilitate updates as new data become available.

19. Management-monitoring sites.—Installation and use of monitoring sites for surface water and ground water can aid in determining the effectiveness of a water-management plan that has been implemented.

20. Real-time data.—Public access to real-time data facilitates timely water-management decisions and helps diffuse potential conflicts about what the actual hydrologic conditions are. An example of publically accessible real-time data is the website for this study [http://ca.water.usgs.gov/sanbern].

21. Public meetings.—Public meetings involving the many water purveyors in the San Bernardino area have been an effective method of hearing new ideas, dispelling rumors, critiquing ongoing technical work, and educating decision makers. Continued use of the meetings will be important as the ground-water flow and constrained optimization models are used to evaluate additional water-management alternatives.

22. Monitoring possible land subsidence.—Monitoring possible land subsidence would be an important part of any water-management plan that significantly increases ground-water pumpage.

23. Redlands contamination.—Mapping the character and extent of the Redlands contamination (*fig. 36*) in three dimensions will aid in understanding historical movement and in critiquing management alternatives.

24. Understanding nitrate contamination.—The fate and transport of nitrogen species is not well understood in the San Bernardino area, despite the significant adverse impacts of high nitrate levels on ground-water quality. Improved, validated conceptual models are critically important before either reliable simulation models or effective water-management plans can be developed.

Summary and Conclusions

The San Bernardino area of southern California has several important water-management issues that are confronting local water managers. These issues include the threat of liquefaction of saturated fine-grained deposits near the land surface in the event of a major earthquake, the possible reoccurrence of land subsidence that can result from excessive ground-water pumpage, a decrease in available water—both locally and statewide—because of possible changes in climate, and the ongoing closure of municipal-supply wells that are contaminated by organic solvents, pesticides, or nitrate. The issues, which have evolved over the past century (*fig. 68*), continue to become more intertwined; solving one requires an understanding of each of the others. The purpose of this report is to provide water managers in the San Bernardino area with improved hydrologic information and computer models to aid them in understanding and solving these issues.

The hydrologic information provided in this report includes analysis of the surface-water and ground-water systems in the San Bernardino area, with an emphasis on the valley-fill aquifer in the Bunker Hill and the Lytle Creek basins, for calendar years 1945–98. The computer models include a three-dimensional ground-water flow model of the valley-fill aquifer and a contrained optimization model that integrates information from the flow model with surface-water and economic data. The flow model was used to assure consistency among the hydrogeologic concepts and to provide quantitative information that cannot be obtained in other ways. The optimization model uses linear-programming techniques to determine the optimal quantities of recharge and pumpage, subject to constraints on ground-water levels throughout the basin, hydraulic gradients near contaminated areas, and costs of imported water and ground-water pumpage. The mathematically optimal solutions derived from the optimization model can be used with the other hydrologic information to guide water-management decisions.

Major conclusions from the study are summarized below by topic.

Ground-Water Recharge

1. Most recharge to the valley-fill aquifer occurs from recharge of gaged runoff. During 1945–98, recharge from gaged runoff averaged about 116,000 acre-ft/yr.

2. Recharge from ungaged runoff is about 10 percent of recharge from gaged runoff. During 1945–98, recharge from ungaged runoff averaged about 16,000 acre-ft/yr.

3. Recharge of imported water is a relatively minor component of total recharge to the valley-fill aquifer. After the California State project water became available in 1972, recharge of imported water averaged about 6,000 acre-ft/yr for 1972–98. For the longer period 1945–98, recharge from imported water averaged about 3,000 acre-ft/yr.

4. Return flow from pumpage is a significant source of recharge to the upper layer of the valley-fill aquifer, nearly twice the quantity of recharge from ungaged runoff. During 1945–98, return flow averaged about 26,000 acre-ft/yr.

5. Recharge from underflow and recharge of local runoff are relatively minor components of total recharge, each averaging about 5,000 acre-ft/yr during 1945–98.

6. Recharge from direct precipitation is a minor component of total recharge, averaging less than about 1,000 acre-ft/yr during 1945–98.

7. Much of the recharge to the valley-fill aquifer occurs during years with unusually large runoff, which occur about once every 5 to 10 years.

8. Keeping the highly permeable stream channels and the off-channel artificial-recharge basins, despite continuing urbanization, is necessary to maintain historical quantities of recharge from native runoff.

9. Additional recharge to the valley-fill aquifer prompts additional discharge from the aquifer, making the gain in aquifer storage less than the amount of the recharge.

Ground-Water Discharge

10. Ground-water pumpage is by far the largest component of total ground-water discharge, averaging about 88 percent of total discharge. During 1945–98, gross ground-water pumpage averaged about 175,000 acre-ft/yr.

11. Ground-water discharge components other than pumpage are fairly minor. During 1945–98, underflow averaged about 13,000 acre-ft/yr, evapotranspiration averaged about 7,000 acre-ft/yr, and rising ground water averaged about 5,000 acre-ft/yr.

12. Additional ground-water pumpage from areas near the San Jacinto fault is necessary during the next 10 to 20 years to prevent high ground-water conditions similar to those that occurred in 1945 and 1980. Much of this extraction can be from the upper water-bearing unit (UWB).

13. Additional ground-water pumpage also is needed from the upper confining member (UCM), generally less than 100 ft thick, in order to control flooding of the land surface and possible liquefaction.

14. If additional ground-water pumpage exceeds about 10,000 acre-ft/yr for several years, then some additional recharge needs to be considered.

Ground-Water Storage

15. Annual fluctuations in ground-water storage commonly range from 50,000 acre-ft to 100,000 acre-ft/yr as a result of normal fluctuations in natural runoff and historical pumpage.

16. Cumulative change in ground-water storage commonly exceeds 500,000 acre-ft during a 10-year period. This magnitude of change probably can be accommodated without causing either additional land subsidence or an increased risk from liquefaction.

17. Maximum cumulative change in storage during 1945–98 was about 900,000 acre-ft, indicating the maximum storage in the valley-fill aquifer is at least this value.

18. Projected increases in demand for municipal water will tend to cause larger fluctuations in ground-water storage either because ground-water pumpage is increased or because less surface water is available for recharge.

19. Future fluctuations in ground-water storage, however, will be dampened somewhat by construction of Seven Oaks Dam on the Santa Ana River. Flood flows will be captured and an average of about 3,000 acre-ft/yr of runoff will be available for artificial recharge.

High Ground-Water Levels

20. High ground-water levels in the former marshland can be caused by high runoff conditions, by the return of high ground-water levels in underlying hydrogeologic units, or by artificial recharge of an unusually large quantity of native or imported water, particularly in the East-Twin Creek area.

21. Ground-water levels in the upper water-bearing unit (UWB) can be expected to rise during wetter periods to levels similar to those observed in about 1945 and 1980, if basin management follows present (1998) practices.

22. Under conditions of recent recharge and pumpage (1983-98), high ground-water levels can be expected to recur about once every 16 years and to persist for at least 3 to 4 years.

23. If high ground-water levels in the former marshland return, yield of the valley-fill aquifer will decrease. Historically, this decrease has been by as much as 50,000 acre-ft/yr.

24. Control of shallow ground-water levels in the former marshland requires areally distributed pumpage. Control cannot be achieved during periods of abundant recharge by a few, large-capacity wells.

Liquefaction

25. High ground-water levels in both the upper confining member (UCM) and the upper water bearing unit (UWB) need to be controlled in order to reduce the threat of liquefaction during a strong earthquake.

26. Keeping shallow ground water in the former marshland at least 30 ft below land surface is needed to reduce the threat of liquefaction during a strong earthquake. Because of the abundant and highly variable recharge to the valley-fill aquifer, this goal is physically difficult to achieve.

Land Subsidence

27. Land subsidence, which occurred during 1945–65, can be expected to reoccur if ground-water levels decline significantly below historic levels in the upper water-bearing unit (UWB) or in deeper hydrogeologic units.

28. Additional ground-water pumpage either from new locations in the Bunker Hill or Lytle Creek basins or from lower hydrogeologic units needs to be monitored to prevent reoccurrence of land subsidence.

29. Monitoring of possible land subsidence can be achieved by installing extensometers in key areas or by analysis of satellite data (InSAR).

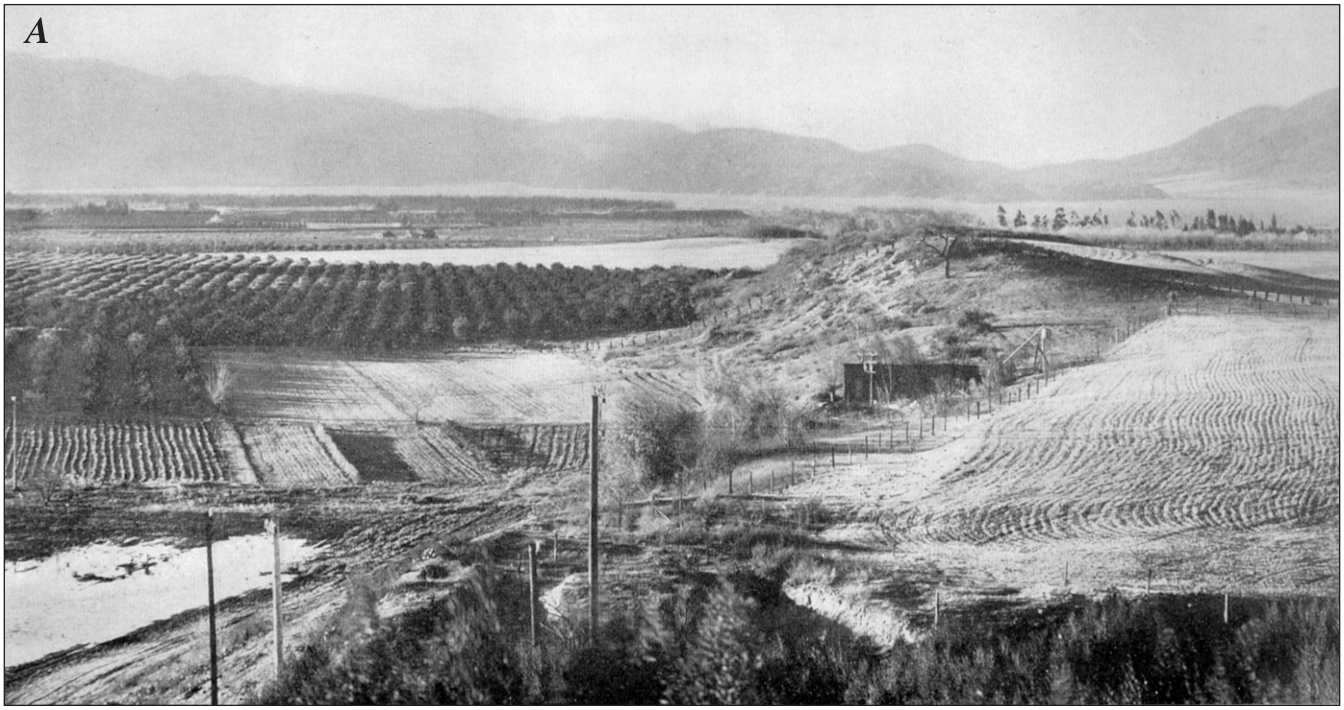
Water Quality

30. Using only five barrier wells to stop migration of contaminates at the U.S. EPA Newmark Superfund site is unlikely to be successful. Additional wells along the barrier, or pumpage from inside the contaminated area may be needed.

31. Controlling migration of contaminates at the U.S. EPA Newmark Superfund site occassionally conflicts with maintaining ground-water levels in the vicinity of the former marshland. Analyzing both objectives in a joint simulation-optimization model helps to identify the conflicts and better understand possible hydraulic solutions.

Basin Yield

32. If high ground-water levels return, the recharge rate of native runoff can be expected to decline from 60 percent of total inflow to about 40 percent of total inflow.



circa 1904 (Mendenhall, 1905, WSP 142, plate XI)

circa 2004 (W.R. Danskin, USGS)

Figure 68. Bunker Hill dike in the San Bernardino area, California, circa 1904 and 2004.

33. If high ground-water levels return, discharge of ground water into Warm Creek will reoccur, effectively decreasing basin yield.

Meeting Future Demand for Water

34. Meeting future demand for water will require a considerable increase in both ground-water pumpage and artificial recharge of imported water.

35. Use of excess pumpage capacity or creation of a new set of areally distributed wells throughout the former marshland would help meet future demand for water, as well as help control high ground-water levels.

Models

36. The ground-water flow model, calibrated with annual data for 1945–98, can be used to evaluate a range of water-management alternatives, involving annual changes in any recharge or discharge component.

37. The ground-water flow model is well suited to provide boundary conditions for smaller-scale models developed to address more localized questions.

38. The constrained optimization model can be used in concert with the ground-water flow model to determine feasible, perhaps optimal ways of managing water resources in the San Bernardino area.

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Additional Tables

Table 1. Annual discharge for gaged streams in the San Bernardino area, California, 1945–98.

[See fig. 2 for definition of the San Bernardino area; values in acre-feet per calendar year are accurate to no more than three significant figures; greater precision is shown for computation purposes only; measured data from the U.S. Geological Survey (J.A. Huff, U.S. Geological Survey, unpub. data, 2000); —, no data; (), data available but are included in another measurement; average values for 1945–98]

Station no.	Station name	1945	1946	1947	1948	1949	1950	1951	1952	1953	1954
	Surface-water inflow to the San Bernardino area										
11051501	Santa Ana River near Mentone, combined discharge	65,390	50,290	33,460	32,000	34,040	26,480	24,170	56,950	27,150	43,000
11054001	Mill Creek near Yucaipa, combined discharge	[1]34,909	26,468	17,060	11,000	12,230	11,330	9,030	26,090	13,150	18,520
11055501	Plunge Creek near East Highlands, combined discharge	[1]8,067	[1]6,013	[1]3,725	[1]3,526	[1]3,803	[1]2,775	[1]2,461	8,580	1,820	5,900
11055801	City Creek near Highland, combined discharge	9,300	6,940	3,310	2,500	4,060	2,710	2,650	11,300	2,300	7,260
11057000	San Timoteo Creek near Redlands	980	491	74	85	40	67	67	1,950	30	869
11058500	East Twin Creek near Arrowhead Springs	4,400	2,740	1,690	1,250	2,060	1,630	1,310	5,410	1,400	3,900
11058600	Waterman Canyon Creek near Arrowhead Springs	2,690	1,740	1,140	705	1,260	998	596	2,830	691	1,670
11062001	Lytle Creek near Fontana, combined discharge	31,300	33,540	25,880	13,930	11,760	10,050	8,540	38,180	13,660	18,500
11063510	Cajon Creek below Lone Pine Creek, near Keenbrook	[2]7,486	[2]9,295	[2]5,199	[2]3,645	[2]3,880	[3]2,535	[3]1,819	[3]10,600	[3]2,809	[3]4,743
11063680	Devil Canyon Creek near San Bernardino	867	772	161	53	206	104	60	1,190	9	773
	Total inflow	165,389	138,289	91,699	68,694	73,339	58,679	50,703	163,080	63,019	105,135
	Surface-water outflow from the San Bernardino area										
11065801	Warm Creek near Colton, combined discharge	62,630	57,250	47,280	42,800	37,150	31,440	27,020	35,220	24,540	24,920
11060400	Warm Creek near San Bernardino	—	—	—	—	—	—	—	—	—	—
11059300	Santa Ana River at E Street	14,740	[4]14,503	[5]21,149	2,010	2,150	1,680	3,870	14,580	1,380	[4]54,963
11066050	Santa Ana River at Colton	—	—	—	—	—	—	—	—	—	—
11065000	Lytle Creek at Colton	—	—	—	—	—	—	—	—	—	—
	Total outflow	77,370	71,753	68,429	44,810	39,300	33,120	30,890	49,800	25,920	79,883
	Inflow minus outflow	88,019	66,536	23,270	23,884	34,039	25,559	19,813	113,280	37,099	25,252

See footnotes at end of table.

Table 1. Annual discharge for gaged streams in the San Bernardino area, California, 1945–98—Continued.

[See fig. 2 for definition of the San Bernardino area; values in acre-feet per calendar year are accurate to no more than three significant figures; greater precision is shown for computation purposes only; measured data from the U.S. Geological Survey (J.A. Huff, U.S. Geological Survey, unpub. data, 2000); —, no data; (), data available but are included in another measurement; average values for 1945–98]

Station no.	Station name	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964
	Surface-water inflow to the San Bernardino area										
11051501	Santa Ana River near Mentone, combined discharge	27,620	26,820	25,640	68,810	26,560	24,690	15,790	33,470	17,410	17,630
11054001	Mill Creek near Yucaipa, combined discharge	12,890	9,180	11,560	44,020	13,330	10,990	7,240	15,030	9,820	10,030
11055501	Plunge Creek near East Highlands, combined discharge	2,010	2,880	3,770	11,870	1,910	1,650	882	4,380	1,240	1,410
11055801	City Creek near Highland, combined discharge	2,570	2,860	6,290	19,930	3,340	2,500	1,600	5,030	1,980	1,720
11057000	San Timoteo Creek near Redlands	106	194	68	1,270	54	35	216	321	47	46
11058500	East Twin Creek near Arrowhead Springs	1,750	1,580	2,140	5,910	1,160	1,030	612	1,710	795	641
11058600	Waterman Canyon Creek near Arrowhead Springs	809	873	1,280	3,260	576	406	192	959	393	325
11062001	Lytle Creek near Fontana, combined discharge	13,780	12,710	13,500	56,320	17,030	10,650	8,020	19,380	12,040	8,950
11063510	Cajon Creek below Lone Pine Creek, near Keenbrook	[3]2,587	[3]2,168	[3]4,160	[3]10,100	[3]3,714	[3]2,241	[3]2,032	[3]8,783	[3]2,021	[3]1,553
11063680	Devil Canyon Creek near San Bernardino	590	235	390	3,690	280	90	5	475	12	21
	Total inflow	64,712	59,500	68,798	225,180	67,954	54,282	36,589	89,538	45,758	42,326
	Surface-water outflow from the San Bernardino area										
11065801	Warm Creek near Colton, combined discharge	19,830	19,830	19,350	25,230	16,060	14,090	14,560	—	—	—
11060400	Warm Creek near San Bernardino	—	—	—	—	—	—	—	—	—	—
11059300	Santa Ana River at E Street	[6]11,859	[6]13,598	[6]13,562	[6]27,160	[6]12,858	[6]14,021	[7]15,804	—	—	—
11066050	Santa Ana River at Colton	—	—	—	—	—	—	—	16,560	15,780	15,100
11065000	Lytle Creek at Colton	—	—	—	()	()	()	()	()	()	()
	Total outflow	31,689	33,428	32,912	52,390	28,918	28,111	30,364	16,560	15,780	15,100
	Inflow minus outflow	33,023	26,072	35,886	172,790	39,036	26,171	6,225	72,978	29,978	27,226

See footnotes at end of table.

Table 1. Annual discharge for gaged streams in the San Bernardino area, California, 1945–98—Continued.

[See fig. 2 for definition of the San Bernardino area; values in acre-feet per calendar year are accurate to no more than three significant figures; greater precision is shown for computation purposes only; measured data from the U.S. Geological Survey (J.A. Huff, U.S. Geological Survey, unpub. data, 2000); —, no data; (), data available but are included in another measurement; average values for 1945–98]

Station no.	Station name	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974
	Surface-water inflow to the San Bernardino area										
11051501	Santa Ana River near Mentone, combined discharge	41,170	65,170	61,220	29,730	216,400	37,870	31,390	23,410	56,500	37,430
11054001	Mill Creek near Yucaipa, combined discharge	21,830	36,110	32,480	19,950	147,100	20,580	13,570	11,940	23,720	17,280
11055501	Plunge Creek near East Highlands, combined discharge	7,040	8,900	9,180	2,080	32,690	3,350	3,040	1,750	7,920	3,740
11055801	City Creek near Highland, combined discharge	7,580	10,780	13,070	3,070	57,150	4,810	4,340	1,930	7,020	3,400
11057000	San Timoteo Creek near Redlands	1,890	3,100	726	[1]285	[1]7,938	[1]619	[1]353	[1]26	[1]1,383	342
11058500	East Twin Creek near Arrowhead Springs	3,160	4,440	4,450	1,320	15,060	2,040	1,940	924	3,050	1,630
11058600	Waterman Canyon Creek near Arrowhead Springs	1,570	2,040	2,770	809	9,290	1,360	813	716	1,740	1,100
11062001	Lytle Creek near Fontana, combined discharge	33,900	42,240	45,570	17,590	145,200	21,540	16,980	13,660	29,360	22,890
11063510	Cajon Creek below Lone Pine Creek, near Keenbrook	[3]16,680	[3]6,547	[3]8,683	[3]3,454	[3]32,340	[3]8,350	[8]5,140	4,210	8,090	6,400
11063680	Devil Canyon Creek near San Bernardino	1,330	1,390	2,810	857	10,880	2,140	1,380	465	2,040	1,930
	Total inflow	136,150	180,717	180,959	79,145	674,048	102,659	78,946	59,031	140,823	96,142
	Surface-water outflow from the San Bernardino area										
11065801	Warm Creek near Colton, combined discharge	—	—	—	—	—	—	—	—	—	—
11060400	Warm Creek near San Bernardino	()	858	479	146	3,950	844	944	[10]593	[10]14,268	[10]3,617
11059300	Santa Ana River at E Street	—	[9]49,742	14,060	11,910	246,700	17,340	15,230	12,020	30,900	25,490
11066050	Santa Ana River at Colton	55,340	—	—	—	—	—	—	—	—	—
11065000	Lytle Creek at Colton	()	6,870	2,240	477	47,690	1,820	1,220	176	5,900	1,050
	Total outflow	55,340	57,470	16,779	12,533	298,340	20,004	17,394	12,789	51,068	30,157
	Inflow minus outflow	80,810	123,247	164,180	66,612	375,708	82,655	61,552	46,242	89,755	65,985

See footnotes at end of table.

Table 1. Annual discharge for gaged streams in the San Bernardino area, California, 1945–98—Continued.

[See fig. 2 for definition of the San Bernardino area; values in acre-feet per calendar year are accurate to no more than three significant figures; greater precision is shown for computation purposes only; measured data from the U.S. Geological Survey (J.A. Huff, U.S. Geological Survey, unpub. data, 2000); —, no data; (), data available but are included in another measurement; average values for 1945–98]

Station no.	Station name	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984
	Surface-water inflow to the San Bernardino area										
11051501	Santa Ana River near Mentone, combined discharge	31,460	30,800	22,940	110,200	102,600	219,600	33,530	61,590	163,100	38,710
11054001	Mill Creek near Yucaipa, combined discharge	13,930	15,490	13,580	124,900	39,440	87,290	19,750	28,420	69,490	20,570
11055501	Plunge Creek near East Highlands, combined discharge	2,870	2,860	2,710	20,600	8,560	24,750	2,510	6,920	18,800	2,880
11055801	City Creek near Highland, combined discharge	2,860	2,600	2,420	28,130	10,190	39,780	3,180	7,500	21,700	4,130
11057000	San Timoteo Creek near Redlands	291	450	425	4,100	[1]3,273	[1]8,070	[1]441	[1]1,591	[1]5,753	[1]653
11058500	East Twin Creek near Arrowhead Springs	1,470	1,300	808	9,430	3,730	15,950	3,430	4,600	13,280	2,840
11058600	Waterman Canyon Creek near Arrowhead Springs	918	725	511	5,480	2,570	10,240	1,530	2,227	7,540	1,560
11062001	Lytle Creek near Fontana, combined discharge	15,650	15,640	14,910	129,900	51,700	119,600	17,230	[1]39,268	[1]106,569	23,300
11063510	Cajon Creek below Lone Pine Creek, near Keenbrook	3,490	4,890	[8]3,221	[3]71,930	[3]11,200	[3]33,870	[3]7,170	[3]7,590	[8]34,322	7,340
11063680	Devil Canyon Creek near San Bernardino	[11]2,820	[11]4,069	[11]1,690	[11]11,430	[11]3,400	[11]14,542	278	[11]3,203	[11]9,220	[11]2,895
	Total inflow	75,759	78,824	63,215	516,100	236,663	573,692	89,049	162,909	449,774	104,878
	Surface-water outflow from the San Bernardino area										
11065801	Warm Creek near Colton, combined discharge	—	—	—	—	—	—	—	—	—	—
11060400	Warm Creek near San Bernardino	1,330	1,760	1,630	51,820	3,100	19,460	7,940	13,920	24,640	19,550
11059300	Santa Ana River at E Street	19,410	24,000	24,550	153,000	55,630	320,300	27,150	61,260	248,770	44,040
11066050	Santa Ana River at Colton	—	—	—	—	—	—	—	—	—	—
11065000	Lytle Creek at Colton	130	1,060	635	37,360	2,750	29,990	1,200	3,010	[12]13,100	[13]3,117
	Total outflow	20,870	26,820	26,815	242,180	61,480	369,750	36,290	78,190	286,510	66,707
	Inflow minus outflow	54,889	52,004	36,400	273,920	175,183	203,942	52,759	84,719	163,264	38,171

See footnotes at end of table.

Table 1. Annual discharge for gaged streams in the San Bernardino area, California, 1945–98—Continued.

[See fig. 2 for definition of the San Bernardino area; values in acre-feet per calendar year are accurate to no more than three significant figures; greater precision is shown for computation purposes only; measured data from the U.S. Geological Survey (J.A. Huff, U.S. Geological Survey, unpub. data, 2000); —, no data; (), data available but are included in another measurement; average values for 1945–98]

Station no.	Station name	1985	1986	1987	1988	1989	1990	1991	1992	1993	1994
	Surface-water inflow to the San Bernardino area										
11051501	Santa Ana River near Mentone, combined discharge	30,570	43,100	24,020	21,220	17,680	12,770	26,450	39,390	170,100	35,740
11054001	Mill Creek near Yucaipa, combined discharge	16,990	[1]22,449	[1]11,783	[1]10,218	[1]8,239	[1]5,494	[1]13,142	[1]20,375	[1]93,442	[1]18,335
11055501	Plunge Creek near East Highlands, combined discharge	2,580	4,990	2,080	1,960	2,030	1,770	3,450	4,260	24,460	2,670
11055801	City Creek near Highland, combined discharge	3,340	5,740	2,410	2,414	2,320	1,460	4,410	6,050	32,470	3,280
11057000	San Timoteo Creek near Redlands	[1]319	[1]833	[1]51	[1]0	[1]0	[1]0	[1]150	[1]681	[1]6,040	[1]531
11058500	East Twin Creek near Arrowhead Springs	1,940	3,610	1,250	1,300	1,170	1,640	7,680	2,530	16,750	2,060
11058600	Waterman Canyon Creek near Arrowhead Springs	[1]1,050	[1]2,071	[1]629	[1]659	[1]580	[1]867	[1]4,557	[1]1,411	[1]10,099	[1]1,124
11062001	Lytle Creek near Fontana, combined discharge	16,190	26,850	13,780	15,690	11,480	8,210	16,810	45,210	123,000	16,620
11063510	Cajon Creek below Lone Pine Creek, near Keenbrook	5,970	7,250	4,520	4,710	2,820	2,470	4,200	10,150	26,260	7,240
11063680	Devil Canyon Creek near San Bernardino	[11]2,360	[11]2,962	[11]1,994	[11]1,713	[11]1,690	[11]1,200	[11]2,180	[11]2,538	[11]10,374	1,770
	Total inflow	81,309	119,855	62,517	59,884	48,009	35,881	83,029	132,595	512,995	89,370
	Surface-water outflow from the San Bernardino area										
11065801	Warm Creek near Colton, combined discharge	—	—	—	—	—	—	—	—	—	—
11060400	Warm Creek near San Bernardino	19,150	18,590	14,500	11,530	9,340	6,780	5,350	5,860	5,690	2,610
11059300	Santa Ana River at E Street	41,280	56,410	34,700	32,830	30,030	30,290	49,310	55,540	206,900	35,830
11066050	Santa Ana River at Colton	—	—	—	—	—	—	—	—	—	—
11065000	Lytle Creek at Colton	1,390	4,660	1,730	1,910	1,020	1,550	4,160	8,900	18,640	841
	Total outflow	61,820	79,660	50,930	46,270	40,390	38,620	58,820	70,300	231,230	39,281
	Inflow minus outflow	19,489	40,195	11,587	13,614	7,619	–2,739	24,209	62,295	281,765	50,089

See footnotes at end of table.

Table 1. Annual discharge for gaged streams in the San Bernardino area, California, 1945–98—Continued.

[See fig. 2 for definition of the San Bernardino area; values in acre-feet per calendar year are accurate to no more than three significant figures; greater precision is shown for computation purposes only; measured data from the U.S. Geological Survey (J.A. Huff, U.S. Geological Survey, unpub. data, 2000); —, no data; (), data available but are included in another measurement; average values for 1945–98]

Station no.	Station name	1995	1996	1997	1998	Average
	Surface-water inflow to the San Bernardino area					
11051501	Santa Ana River near Mentone, combined discharge	124,400	46,500	35,860	116,500	52,528
11054001	Mill Creek near Yucaipa, combined discharge	[1]67,896	[1]24,350	[1]18,402	[1]63,480	27,702
11055501	Plunge Creek near East Highlands, combined discharge	15,210	3,650	5,610	16,430	6,351
11055801	City Creek near Highland, combined discharge	20,100	5,750	7,490	19,360	8,377
11057000	San Timoteo Creek near Redlands	[1]4,166	[1]972	[1]536	[1]3,842	1,238
11058500	East Twin Creek near Arrowhead Springs	9,480	3,540	4,270	9,990	3,800
11058600	Waterman Canyon Creek near Arrowhead Springs	[1]5,657	[1]2,028	[1]2,474	[1]5,969	2,187
11062001	Lytle Creek near Fontana, combined discharge	64,930	24,440	18,750	59,640	32,630
11063510	Cajon Creek below Lone Pine Creek, near Keenbrook	16,440	7,250	4,820	12,860	9,134
11063680	Devil Canyon Creek near San Bernardino	6,720	2,510	3,030	5,530	2,506
	Total inflow	334,999	120,990	101,242	313,601	146,452
	Surface-water outflow from the San Bernardino area					
11065801	Warm Creek near Colton, combined discharge	—	—	—	—	—
11060400	Warm Creek near San Bernardino	6,820	3,920	3,430	10,910	—
11059300	Santa Ana River at E Street	153,400	28,580	18,580	110,000	—
11066050	Santa Ana River at Colton	—	—	—	—	—
11065000	Lytle Creek at Colton	7,940	3,820	1,290	8,230	—
	Total outflow	168,160	36,320	23,300	129,140	67,931
	Inflow minus outflow	166,839	84,670	77,942	184,461	78,522

See footnotes at end of table.

Table 1. Annual discharge for gaged streams in the San Bernardino area, California, 1945–98—Continued.

[See fig. 2 for definition of the San Bernardino area; values in acre-feet per calendar year are accurate to no more than three significant figures; greater precision is shown for computation purposes only; measured data from the U.S. Geological Survey (J.A. Huff, U.S. Geological Survey, unpub. data, 2000); —, no data; (), data available but are included in another measurement; average values for 1945–98]

[1]Estimated from regression relations shown in figure 13*A-E*.

[2]Estimated from the sum of the measured value for Cajon Creek (station no. 11063000) and the estimated value for Lone Pine Creek (station no. 11063500) determined from regression relation shown in figure 13*F*.

[3]Estimated from the sum of the measured values for Cajon Creek (station no. 11063000) and Lone Pine Creek (station no. 11063500).

[4]Estimated from regression relation shown in figure 13*I*.

[5]Estimated from regression relation shown in figure 13*J*.

[6]Estimated from the sum of measured values for Santa Ana River near San Bernardino (station no. 11056200), San Timoteo Creek near Loma Linda (station no. 11057500), and wastewater discha ge to the Santa Ana River and Warm Creek.

[7]Estimated from the sum of measured values for Santa Ana River near San Bernardino (station no. 11056200), San Timoteo Creek near Loma Linda (station no. 11057500), Warm Creek Floodway at San Bernardino (station no. 11059000), and wastewater discharge into the Santa Ana River and Warm Creek.

[8]Estimated from the sum of the measured value for Lone Pine Creek (station no. 11063500) and the estimated value for Cajon Creek (station no. 11063000) determined from regression relation shown in figure 13*G*.

[9]Estimated from the sum of measured values for Santa Ana River near San Bernardino (station no. 11056200), Warm Creek Floodway at San Bernardino (station no. 11059000), and wastewater discharge into the Santa Ana River and Warm Creek, and from the estimated value for San Timoteo Creek near Loma Linda (station no. 11057500) determined from regression relation shown in figure 7*K*.

[10]Estimated from total outflow determined from regression relation shown in figure 13*H* minus the sum of Santa Ana River at E Street (station no. 11059300) and Lytle Creek at Colton (station no. 11065000).

[11]Combined discharge (upstream diversion by City of San Bernardino begun in 1975 and ceased in 1993).

[12]Estimated from 11 months of daily values.

[13]Estimated from regression relation shown in figure 13*L*.

Table 3. Average annual runoff from ungaged areas bordering the Bunker Hill and Lytle Creek basins in the San Bernardino area, California, 1928–98.

[Values for calendar years; ungaged areas adapted from Webb and Hanson (1972, plate 26); estimated average ungaged runoff for calendar years 1945–98 is 0.946 times the values listed below]

Ungaged area no. (fig.17)	Ungaged area	Acreage (acres)	Average annual precipitation (inches per year)	Potential ungaged runoff (acre-feet per year)	Estimated runoff rate (percent)	Estimated average ungaged runoff (acre-feet per year)
1	West of Lytle Creek	1,134	25.00	2,363	25	591
2	Lytle Creek to Cajon Creek	5,085	[1]24.44	10,355	25	2,589
3	Cajon Creek to Devil Canyon Creek	7,625	24.54	15,594	25	3,898
4	Devil Canyon Creek to Waterman Creek	2,497	20.99	4,367	25	1,092
5	East Twin Creek to City Creek	5,582	20.87	9,709	25	2,427
6	City Creek to Plunge Creek	2,575	19.51	4,186	25	1,047
7	Plunge Creek to Santa Ana River	2,195	19.02	3,479	25	870
8	Santa Ana River to Mill Creek	2,550	18.70	3,973	25	993
9	Mill Creek to Crafton Hills	2,195	19.72	3,608	25	902
10	Crafton Hills and adjacent older alluvium	5,295	15.14	6,681	20	1,336
11	Badlands	1,593	15.00	1,992	20	398
12	Bedrock outcrops within the valley fill	1,281	[1]18.90	2,018	25	504
	Total	39,609	[1]20.70	68,324	24.4	16,647

[1]Calculated for multiple areas.

Table 4. Precipitation components for the San Bernardino area, California, 1945–98.

[Precipitation at San Bernardino County Flood Control District station 47723 (fig. 6); precipitation components calculated for the Bunker Hill and Lytle Creek basins; average for calendar years 1945–98]

Calendar year	Precipitation at San Bernardino, in inches per year	Precipitation component								
		Percent of precipitation				Acre-feet of water				
		Direct recharge	Local runoff	Evapotrans-piration	Recharge from local runoff	Direct recharge	Local runoff	Evapo-transpiration	Recharge from local runoff	Total
1945	18.61	0	15	80	5	0	19,155	102,161	6,385	127,702
1946	17.90	0	15	80	5	0	18,424	98,264	6,141	122,830
1947	5.95	0	15	80	5	0	6,124	32,663	2,041	40,829
1948	13.24	0	15	80	5	0	13,628	72,682	4,543	90,853
1949	14.84	0	15	80	5	0	15,275	81,466	5,092	101,832
1950	9.00	0	15	80	5	0	9,264	49,406	3,088	61,758
1951	17.03	0	15	80	5	0	17,529	93,488	5,843	116,860
1952	21.69	0	15	80	5	0	22,326	119,069	7,442	148,837
1953	7.37	0	15	80	5	0	7,586	40,458	2,529	50,573
1954	19.35	0	15	80	5	0	19,917	106,224	6,639	132,780
1955	12.23	0	15	80	5	0	12,588	67,138	4,196	83,922
1956	11.21	0	15	80	5	0	11,538	61,538	3,846	76,923
1957	20.45	0	15	80	5	0	21,049	112,262	7,016	140,328
1958	20.83	0	15	80	5	0	21,440	114,348	7,147	142,935
1959	9.70	0	15	80	5	0	9,984	53,249	3,328	66,561
1960	12.69	0	15	80	5	0	13,062	69,663	4,354	87,079
1961	6.54	0	15	80	5	0	6,732	35,902	2,244	44,877
1962	10.53	0	15	80	5	0	10,839	57,805	3,613	72,257
1963	17.35	0	15	80	5	0	17,858	95,245	5,953	119,056
1964	9.88	0	15	80	5	0	10,169	54,237	3,390	67,797
1965	21.71	0	15	80	5	0	22,346	119,179	7,449	148,974
1966	14.76	0	15	80	5	0	15,192	81,026	5,064	101,283
1967	17.13	0	15	80	5	0	17,632	94,037	5,877	117,546
1968	7.50	0	15	80	5	0	7,720	41,172	2,573	51,465
1969	31.96	5	15	75	5	10,965	32,896	164,482	10,965	219,310
1970	14.54	0	15	80	5	0	14,966	79,819	4,989	99,773
1971	13.46	0	15	80	5	0	13,854	73,890	4,618	92,363
1972	6.85	0	15	80	5	0	7,051	37,604	2,350	47,005
1973	14.50	0	15	80	5	0	14,925	79,599	4,975	99,499
1974	15.06	0	15	80	5	0	15,501	82,673	5,167	103,342

Table 4. Precipitation components for the San Bernardino area, California, 1945–98—Continued.

[Precipitation at San Bernardino County Flood Control District station 47723 (fig. 6); precipitation components calculated for the Bunker Hill and Lytle Creek basins; average for calendar years 1945–98]

Calendar year	Precipitation at San Bernardino, in inches per year	Precipitation component								
		Percent of precipitation				Acre-feet of water				
		Direct recharge	Local runoff	Evapotrans-piration	Recharge from local runoff	Direct recharge	Local runoff	Evapo-transpiration	Recharge from local runoff	Total
1975	11.93	0	15	80	5	0	12,280	65,491	4,093	81,864
1976	15.07	0	15	80	5	0	15,512	82,728	5,171	103,410
1977	14.91	0	15	80	5	0	15,347	81,850	5,116	102,312
1978	30.83	5	15	75	5	10,578	31,733	158,667	10,578	211,555
1979	16.46	0	15	80	5	0	16,942	90,359	5,647	112,949
1980	26.57	5	15	75	5	9,116	27,349	136,743	9,116	182,323
1981	11.51	0	15	80	5	0	11,847	63,185	3,949	78,982
1982	22.92	0	15	80	5	0	23,592	125,822	7,864	157,277
1983	34.37	5	15	75	5	11,792	35,377	176,885	11,792	235,847
1984	8.93	0	15	80	5	0	9,192	49,022	3,064	61,278
1985	10.90	0	15	80	5	0	11,219	59,837	3,740	74,796
1986	15.52	0	15	80	5	0	15,975	85,199	5,325	106,498
1987	13.03	0	15	80	5	0	13,412	71,529	4,471	89,412
1988	12.18	0	15	80	5	0	12,537	66,863	4,179	83,579
1989	7.23	0	15	80	5	0	7,442	39,690	2,481	49,612
1990	8.19	0	15	80	5	0	8,430	44,960	2,810	56,200
1991	17.84	0	15	80	5	0	18,363	97,934	6,121	122,418
1992	20.10	0	15	80	5	0	20,689	110,341	6,896	137,926
1993	26.29	5	15	75	5	9,020	27,060	135,301	9,020	180,402
1994	13.87	0	15	80	5	0	14,276	76,141	4,759	95,176
1995	21.64	0	15	80	5	0	22,274	118,795	7,425	148,494
1996	18.70	0	15	80	5	0	19,248	102,656	6,416	128,319
1997	17.36	0	15	80	5	0	17,869	95,299	5,956	119,124
1998	29.00	5	15	75	5	9,950	29,850	149,249	9,950	198,998
Average	15.91	1	15	79	5	1,137	16,377	86,209	5,459	109,183

Table 5. Distribution, quantity, and use of water imported into the San Bernardino area, California, 1972–98.

[Values, in acre-feet per calendar year, obtained from the San Bernardino Valley Municipal Water District (written commun., 2000), the local agency entitled to import water via the California State Water Project. Maximum annual entitlement is defined by state contract (Santa Ana Watershed Protection Agency, 1980, table 1, p. 33). No water was imported prior to 1972. na, not applicable]

Site no. (fig. 11)	Delivery point	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982
	Deliveries inside the Bunker Hill and Lytle Creek basins											
	Artificial recharge											
1	Lytle Creek gravel pit	0	0	0	0	0	0	0	0	0	0	2,912
2	Sweetwater to Devil Canyon	98	5,316	3,242	4,606	3,075	7,352	7,243	4,000	3,969	6,177	2,899
3	Badger	22	3,991	3,204	2,684	2,166	3,134	624	294	294	895	186
4	Waterman	1,155	22,918	9,747	6,821	7,045	3,637	181	48	48	7	0
5	Patton	0	0	0	0	1	0	12	0	0	0	0
6	City Creek	0	0	0	0	8	2	67	0	0	0	0
7	Santa Ana	0	0	0	0	260	10,438	5,103	126	126	8,494	3,171
	Subtotal	1,275	32,225	16,193	14,111	12,555	24,563	13,230	4,468	4,437	15,573	9,168
	Agricultural use											
	North Fork	0	0	0	0	0	0	20	35	35	682	66
	Bear Valley canal	0	0	0	0	0	0	0	0	0	0	0
	Greenspot pump station	0	0	0	0	0	0	0	0	0	0	0
	Edwards pump line	na	na	na	na	na	na	na	na	na	na	na
	Seven Oaks Dam construction	na	na	na	na	na	na	na	na	na	na	na
	Subtotal	0	0	0	0	0	0	20	35	35	682	66
	Municipal use											
	East Valley Water District	0	0	0	0	0	0	0	0	0	0	0
	Subtotal	0	0	0	0	0	0	0	0	0	0	0
	Deliveries outside the Bunker Hill and Lytle Creek basins											
	Linden ponds	0	0	0	0	0	0	0	0	0	0	3,514
	Havasu Water Company	na	na	na	na	na	na	na	na	na	na	na
	Subtotal	0	0	0	0	0	0	0	0	0	0	3,514
Total imported		1,275	32,225	16,193	14,111	12,555	24,563	13,250	4,503	4,472	16,255	12,748
Maximum entitlement		1,677	48,000	50,000	52,500	55,000	57,500	60,000	62,500	62,500	68,500	71,500

Table 5. Distribution, quantity, and use of water imported into the San Bernardino area, California, 1972–98—Continued.

[Values, in acre-feet per calendar year, obtained from the San Bernardino Valley Municipal Water District (written commun., 2000), the local agency entitled to import water via the California State Water Project. Maximum annual entitlement is defined by state contract (Santa Ana Watershed Protection Agency, 1980, table 1, p. 33). No water was imported prior to 1972. na, not applicable]

Site no. (fig. 11)	Delivery point	1983	1984	1985	1986	1987	1988	1989	1990	1991	1992
	Deliveries inside the Bunker Hill and Lytle Creek basins										
	Artificial recharge										
1	Lytle Creek gravel pit	1,424	215	0	0	85	0	0	0	0	0
2	Sweetwater to Devil Canyon	54	169	1,161	0	0	0	3,840	6,834	982	128
3	Badger	0	0	0	0	0	0	0	10	0	0
4	Waterman	0	0	0	0	0	0	0	0	0	0
5	Patton	0	0	0	0	0	0	0	0	0	0
6	City Creek	0	0	0	0	0	0	0	0	0	0
7	Santa Ana	0	0	0	0	0	0	545	0	82	0
	Subtotal	1,478	384	1,161	0	85	0	4,385	6,844	1,064	128
	Agricultural use										
	North Fork	0	573	1,417	282	2,406	2,509	2,136	2,488	922	0
	Bear Valley canal	0	618	1,947	626	3,022	4,367	7,861	7,848	3,428	2,538
	Greenspot pump station	0	0	0	0	551	27	68	0	25	5
	Edwards pump line	na	na	na	na	131	460	579	443	390	0
	Seven Oaks Dam construction	na	na	na	na	na	na	na	na	na	na
	Subtotal	0	1,191	3,364	908	6,110	7,363	10,644	10,779	4,765	2,543
	Municipal use										
	East Valley Water District	98	222	395	558	364	0	871	687	353	0
	Subtotal	98	222	395	558	364	0	871	687	353	0
	Deliveries outside the Bunker Hill and Lytle Creek basins										
	Linden ponds	4,736	3,471	3,879	5,345	3,030	4,601	4,522	65	435	1,559
	Havasu Water Company	na	na	na	na	78	58	63	52	55	52
	Subtotal	4,736	3,471	3,879	5,345	3,108	4,659	4,585	117	490	1,611
Total imported		6,312	5,268	8,799	6,811	9,667	12,022	20,485	18,427	6,672	4,282
Maximum entitlement		74,500	78,000	81,500	85,000	89,000	93,000	97,000	102,600	102,600	102,600

Table 5. Distribution, quantity, and use of water imported into the San Bernardino area, California, 1972–98—Continued.

[Values, in acre-feet per calendar year, obtained from the San Bernardino Valley Municipal Water District (written commun., 2000), the local agency entitled to import water via the California State Water Project. Maximum annual entitlement is defined by state contract (Santa Ana Watershed Protection Agency, 1980, table 1, p. 33). No water was imported prior to 1972. na, not applicable]

Site no. (fig. 11)	Delivery point	1993	1994	1995	1996	1997	1998	Average 1972–98	Average 1945–98
Deliveries inside the Bunker Hill and Lytle Creek basins									
Artificial recharge									
1	Lytle Creek gravel pit	0	0	0	2	0	0	172	86
2	Sweetwater to Devil Canyon	2,370	2,674	0	96	0	4	2,455	1,228
3	Badger	0	0	0	0	0	0	648	324
4	Waterman	0	0	0	0	0	0	1,911	956
5	Patton	0	0	0	0	0	0	0	0
6	City Creek	0	0	0	0	0	0	77	39
7	Santa Ana	0	1,223	0	0	0	163	1,027	513
	Subtotal	2,370	3,897	0	98	0	167	6,291	3,146
Agricultural use									
	North Fork	16	53	0	130	1,113	0	551	276
	Bear Valley canal	515	5,080	601	4,541	7,013	589	1,874	937
	Greenspot pump station	17	0	24	0	0	12	27	14
	Edwards pump line	0	18	0	174	423	0	97	48
	Seven Oaks Dam construction	na	na	49	547	627	638	69	34
	Subtotal	548	5,151	674	5,392	9,176	1,239	2,618	1,309
Municipal use									
	East Valley Water District	0	10	0	690	2,934	380	280	140
	Subtotal	0	10	0	690	2,934	380	280	140
Deliveries outside the Bunker Hill and Lytle Creek basins									
	Linden ponds	1,502	261	0	78	0	0	1,370	685
	Havasu Water Company	0	0	0	0	0	0	13	7
	Subtotal	1,502	261	0	78	0	0	1,384	692
Total imported		4,420	9,319	674	6,258	12,110	1,786	10,573	5,286
Maximum entitlement		102,600	102,600	102,600	102,600	102,600	102,600	78,188	39,094

Table 6. Major wastewater treatment facilities in the San Bernardino area, California.

[Modified and updated from Hill (1979, table 2) and J.C. Hanson (Consulting Civil Engineer, unpub. data, 1984); present is 1998; +, greater than; (?), uncertain]

Site no. (fig. 20)	Name of facility	Type of treatment	Disposal of effluent	Period of operation	Range of discharge (acre-feet per year)
1	City of San Bernardino, plant 1	Sewage treatment plant	Direct discharge into Warm Creek	1928–1972	5,000–10,000
2	City of San Bernardino, plant 2	Sewage treatment plant	Direct discharge into Santa Ana River, irrigation	1959–present	5,000–50,000
3	City of Redlands	Sewage treatment plant	Percolation ponds	1930s–present	1,500–10,700
4	Mountainview Power Company, previously Southern California Edison Company, San Bernardino plant	Cooling tower disposal	Direct discharge into Santa Ana River, export	1957–present	100–1500+
5	Santa Fe Railway	Industrial treatment plant	Evaporation ponds and city sewer system	Prior to 1962–1994	300–700
6	Loma Linda Sanitation District	Sewage treatment plant	Direct discharge into Santa Ana River	1945–1967	200–400
7	Patton State Hospital	Sewage discharge	Irrigation (?) and city sewer system	Prior to 1959–present	200–400
8	Culligan, Inc.	Industrial brine disposal	Percolation and evaporation ponds	1942–1986	100–300
9	Universal Rundle Co.	Industrial treatment plant	Percolation ponds	1946–unknown	10–150
10	Lockheed Propulsion	Sewage treatment plant	Percolation ponds	1955–1976	50–100
11	Norton Air Force base, now San Bernardino Airport Authority	Industrial treatment plant	Percolation ponds, discharge to Santa Ana River, and city sewer system	1954–1995	86
12	Campus Crusade for Christ	Sewage treatment plant	Irrigation, percolation ponds	1938–present	30–50
13	Glen Helen Rehabilitation Center	Sewage treatment plant	Percolation ponds	1959–present	10–20

Table 7. Annual wastewater discharge to the Santa Ana River and Warm Creek in the San Bernardino area, California, 1945–98.

[Refer table 6 for names and descriptions and figure 20 for locations of wastewater facilities. Data from Hill (1979); J.C. Hanson (Consulting Civil Engineer, unpub. data, 1984); S.H. Fuller (San Bernardino Valley Municipal Water District, unpub. data, 1999); H.E. Wade (Southern California Edison, unpub. data, 1993); Western–San Bernardino Watermaster (2000); and U.S. Geological Survey (surface-water station 11059100). Values in acre-feet; [w] all discharge to Warm Creek; [si] most discharge to the Santa Ana River and some to irrigation; [sc] discharge to Santa Ana River, Gage Canal, or Riverside Water Company Canal; [e] estimated; [p] partial record; nio, not in operation; [RIX] discharge to Rapid Infiltration/Extraction facility begun April 1996; average for 1945–98, excluding RIX discharge]

Calendar year	City of San Bernardino		Southern California Edison, San Bernardino plant[sc]	Loma Linda Sanitation District	Total
	Plant 1[w]	Plant 2[si]			
1945	6,999	nio	nio	[e,p]126	[e,p]7,125
1946	6,828	nio	nio	[e]169	[e]6,997
1947	7,063	nio	nio	[e]175	[e]7,238
1948	7,062	nio	nio	[e]180	[e]7,242
1949	7,925	nio	nio	[e]186	[e]8,111
1950	8,116	nio	nio	[e]192	[e]8,308
1951	8,506	nio	nio	[e]197	[e]8,703
1952	9,325	nio	nio	[e]203	[e]9,528
1953	7,865	nio	nio	[e]209	[e]8,074
1954	8,715	nio	nio	[e]214	[e]8,929
1955	9,914	nio	nio	[e]220	[e]10,134
1956	9,512	nio	nio	[e]226	[e]9,738
1957	9,792	nio	[e]232	[e]231	[e]10,255
1958	10,013	nio	[e]310	[e]237	[e]10,560
1959	8,845	1,736	271	[e]242	[e]11,094
1960	7,641	4,462	227	[e]249	[e]12,579
1961	7,718	6,071	315	[e]238	[e]14,342
1962	6,239	7,054	242	265	13,800
1963	5,577	7,438	253	233	13,501
1964	6,278	7,624	227	328	14,457
1965	6,761	8,212	203	[p]295	[p]15,471
1966	6,994	8,316	163	nio	15,473
1967	6,968	9,053	232	nio	16,253
1968	6,700	9,214	342	nio	16,256
1969	5,318	9,124	324	nio	14,766
1970	4,377	9,865	473	nio	14,715
1971	5,705	8,925	600	nio	15,230
1972	6,784	9,963	623	nio	17,370
1973	nio	17,640	515	nio	18,660
1974	nio	17,070	1,004	nio	18,074
1975	nio	16,820	1,099	nio	17,919

Table 7. Annual wastewater discharge to the Santa Ana River and Warm Creek in the San Bernardino area, California, 1945–98—Continued.

[Refer table 6 for names and descriptions and figure 20 for locations of wastewater facilities. Data from Hill (1979); J.C. Hanson (Consulting Civil Engineer, unpub. data, 1984); S.H. Fuller (San Bernardino Valley Municipal Water District, unpub. data, 1999); H.E. Wade (Southern California Edison, unpub. data, 1993); Western–San Bernardino Watermaster (2000); and U.S. Geological Survey (surface-water station 11059100). Values in acre-feet; [w] all discharge to Warm Creek; [si] most discharge to the Santa Ana River and some to irrigation; [sc] discharge to Santa Ana River, Gage Canal, or Riverside Water Company Canal; [e] estimated; [p] partial record; nio, not in operation; [RIX] discharge to Rapid Infiltration/Extraction facility begun April 1996; average for 1945–98, excluding RIX discharge]

Calendar year	City of San Bernardino		Southern California Edison, San Bernardino plant[sc]	Loma Linda Sanitation District	Total
	Plant 1[w]	Plant 2[si]			
1976	nio	17,530	1,137	nio	18,667
1977	nio	17,720	1,338	nio	19,058
1978	nio	18,690	1,350	nio	20,040
1979	nio	19,350	1,057	nio	20,407
1980	nio	20,670	1,463	nio	22,123
1981	nio	21,120	1,518	nio	22,638
1982	nio	24,493	843	nio	25,336
1983	nio	23,170	464	nio	23,634
1984	nio	21,759	183	nio	21,942
1985	nio	23,814	122	nio	23,936
1986	nio	25,442	365	nio	25,807
1987	nio	27,154	127	nio	27,281
1988	nio	27,290	49	nio	27,339
1989	nio	28,300	38	nio	28,338
1990	nio	27,770	57	nio	27,827
1991	nio	26,980	35	nio	27,015
1992	nio	25,510	38	nio	25,551
1993	nio	24,660	5	nio	24,665
1994	nio	24,790	14	nio	24,804
1995	nio	26,260	30	nio	26,290
1996	nio	0 [[RIX]31,084]	22	nio	31,106
1997	nio	0 [[RIX]44,326]	73	nio	44,399
1998	nio	0 [[RIX]48,709]	14	nio	48,723
Average	7,484	15,776	394	220	15,873

Table 9. Annual underflow for the San Bernardino area, California, 1945–98.

All values in acre-feet; –, outflow; net equals total inflow minus total outflow]

Calendar year	Inflow						Outflow			Net
	San Timoteo Canyon	Sand Canyon	Reservoir Canyon	Redlands Heights	Badlands	Total	Near Barrier J	Near Santa Ana River	Total	
1945	4,605	1,165	450	300	280	6,800	10,038	12,054	22,092	–15,292
1946	4,536	1,158	450	300	280	6,724	10,439	11,544	21,983	–15,259
1947	4,468	1,150	450	300	280	6,648	8,935	10,976	19,911	–13,263
1948	4,402	1,143	450	300	280	6,574	5,342	10,768	16,110	–9,536
1949	4,336	1,135	450	300	280	6,501	4,360	10,776	15,136	–8,635
1950	4,271	1,128	450	300	280	6,429	3,449	9,419	12,868	–6,439
1951	4,208	1,120	450	300	280	6,358	2,505	7,215	9,720	–3,362
1952	4,145	1,113	450	300	280	6,287	11,190	7,046	18,236	–11,949
1953	4,083	1,105	450	300	280	6,218	5,229	6,204	11,433	–5,215
1954	4,022	1,098	450	300	280	6,150	6,988	6,212	13,200	–7,050
1955	3,963	1,090	450	300	280	6,083	5,280	5,450	10,730	–4,647
1956	3,904	1,083	450	300	280	6,016	4,811	4,610	9,421	–3,405
1957	3,846	1,075	450	300	280	5,951	5,161	3,864	9,025	–3,074
1958	3,788	1,068	450	300	280	5,886	13,445	3,929	17,374	–11,488
1959	3,732	1,060	450	300	280	5,822	6,508	3,465	9,973	–4,151
1960	3,677	1,053	450	300	280	5,759	3,785	3,091	6,876	–1,117
1961	3,622	1,045	450	300	280	5,697	2,140	2,474	4,614	1,083
1962	3,569	1,038	450	300	280	5,636	7,258	2,091	9,349	–3,713
1963	3,516	1,030	450	300	280	5,576	4,497	1,647	6,144	–568
1964	3,464	1,023	450	300	280	5,516	2,777	1,410	4,187	1,329
1965	3,412	1,015	450	300	280	5,457	10,501	1,249	11,750	–6,293
1966	3,362	1,008	450	300	280	5,399	11,777	1,047	12,824	–7,425
1967	3,312	1,000	450	300	280	5,342	12,217	988	13,205	–7,863
1968	3,263	993	450	300	280	5,286	6,695	871	7,566	–2,280

Table 9. Annual underflow for the San Bernardino area, California, 1945-98—Continued.

All values in acre-feet; –, outflow; net equals total inflow minus total outflow]

Calendar year	Inflow						Outflow			Net
	San Timoteo Canyon	Sand Canyon	Reservoir Canyon	Redlands Heights	Badlands	Total	Near Barrier J	Near Santa Ana River	Total	
1969	3,215	985	450	300	280	5,230	18,938	886	19,824	–14,594
1970	3,167	978	450	300	280	5,175	7,870	1,018	8,888	–3,713
1971	3,121	970	450	300	280	5,121	6,491	1,207	7,698	–2,577
1972	3,075	963	450	300	280	5,067	5,229	1,330	6,559	–1,492
1973	3,029	955	450	300	280	5,014	9,667	1,500	11,167	–6,153
1974	2,985	948	450	300	280	4,962	8,223	1,717	9,940	–4,978
1975	2,941	940	450	300	280	4,911	6,018	1,764	7,782	–2,871
1976	2,897	933	450	300	280	4,860	6,014	1,752	7,766	–2,906
1977	2,855	925	450	300	280	4,810	5,737	1,729	7,466	–2,656
1978	2,813	918	450	300	280	4,760	18,292	1,862	20,154	–15,394
1979	2,771	910	450	300	280	4,711	12,949	2,524	15,473	–10,762
1980	2,731	903	450	300	280	4,663	17,813	3,796	21,609	–16,946
1981	2,691	895	450	300	280	4,616	6,576	5,160	11,736	–7,120
1982	2,651	888	450	300	280	4,569	11,353	6,509	17,862	–13,293
1983	2,612	880	450	300	280	4,522	17,144	7,626	24,770	–20,248
1984	2,574	873	450	300	280	4,476	8,326	8,060	16,386	–11,910
1985	2,536	865	450	300	280	4,431	6,214	7,995	14,209	–9,778
1986	2,499	858	450	300	280	4,387	9,149	7,070	16,219	–11,832
1987	2,462	850	450	300	280	4,342	5,280	6,159	11,439	–7,097
1988	2,426	843	450	300	280	4,299	6,033	5,189	11,222	–6,923
1989	2,391	835	450	300	280	4,256	4,220	4,164	8,384	–4,128
1990	2,356	828	450	300	280	4,213	2,276	3,804	6,080	–1,867

Table 9. Annual underflow for the San Bernardino area, California, 1945-98—Continued.

[All values in acre-feet; –, outflow; net equals total inflow minus total outflow]

Calendar year	Inflow						Outflow			Net
	San Timoteo Canyon	Sand Canyon	Reservoir Canyon	Redlands Heights	Badlands	Total	Near Barrier J	Near Santa Ana River	Total	
1991	2,322	820	450	300	280	4,172	6,432	2,712	9,144	–4,972
1992	2,288	813	450	300	280	4,130	12,171	2,112	14,283	–10,153
1993	2,254	805	450	300	280	4,089	17,976	2,057	20,033	–15,944
1994	2,221	798	450	300	280	4,049	6,366	1,882	8,248	–4,199
1995	2,189	790	450	300	280	4,009	14,270	1,727	15,997	–11,988
1996	2,157	783	450	300	280	3,970	8,603	1,871	10,474	–6,504
1997	2,126	775	450	300	280	3,931	7,066	1,750	8,816	–4,885
1998	2,095	768	450	300	280	3,892	15,117	2,343	17,460	–13,568
Average	3,184	966	450	300	280	5,181	8,391	4,216	12,608	–7,427

Table 10. Annual ground-water pumpage in the San Bernardino area, California, 1945–98.

[Values in acre-feet; na, not applicable; source, refer bottom of table for description of source of annual pumpage values]

Calendar year	Plantiff		Non-plaintiff				Total pumpage	Gross pumpage per model layer		Return flow	Total net pumpage
	Filed		Filed		Non-filed						
	Pumpage	Source	Pumpage	Source	Pumpage	Source		Upper	Lower		
1945	39,112	E1	70,943	E2	12,824	E3	122,879	70,296	52,583	20,083	102,796
1946	43,771	E1	81,807	E2	12,824	E3	138,402	79,300	59,102	22,557	115,845
1947	44,735	P-NV	86,486	NP-NV	12,824	E3	144,045	84,334	59,711	21,020	123,025
1948	51,540	P-NV	95,182	NP-NV	12,824	E3	159,546	93,776	65,770	24,675	134,871
1949	44,539	P-NV	92,227	NP-NV	12,824	E3	149,590	84,638	64,952	24,407	125,183
1950	52,523	P-NV	100,210	NP-NV	12,824	E3	165,557	96,822	68,735	27,020	138,538
1951	54,793	P-NV	103,356	NP-NV	12,824	E3	170,973	95,928	75,045	29,335	141,638
1952	43,444	P-NV	78,117	NP-NV	12,824	E3	134,385	74,642	59,743	23,636	110,749
1953	64,668	P-NV	106,949	NP-NV	12,824	E3	184,441	104,667	79,774	30,418	154,023
1954	51,827	P-NV	97,890	NP-NV	12,824	E3	162,541	88,907	73,634	28,310	134,231
1955	65,395	P-NV	105,637	NP-NV	12,824	E3	183,856	98,574	85,282	30,588	153,268
1956	70,771	P-NV	119,610	NP-NV	12,824	E3	203,205	108,812	94,393	34,745	168,460
1957	57,628	P-NV	100,878	NP-NV	12,824	E3	171,330	91,159	80,171	30,044	141,286
1958	60,571	P-NV	95,588	NP-NV	12,824	E3	168,983	87,413	81,570	28,398	140,585
1959	70,353	P-V	120,459	NP-V	12,824	NP-NFV	203,636	111,087	92,549	33,220	170,416
1960	64,865	P-V	116,019	NP-V	11,899	NP-NFV	192,783	105,163	87,620	32,518	160,265
1961	70,944	P-V	130,419	NP-V	13,680	NP-NFV	215,043	116,489	98,554	36,992	178,051
1962	65,102	P-V	109,181	NP-V	13,121	NP-NFV	187,404	100,304	87,100	32,301	155,103
1963	60,983	P-V	103,718	NP-V	12,599	NP-NFV	177,300	93,725	83,575	30,653	146,647
1964	65,892	P-NV	113,630	NP-NV	12,001	E4	191,523	99,967	91,557	33,287	158,237
1965	54,941	P-NV	104,133	NP-NV	11,404	E4	170,478	86,890	83,588	30,896	139,582
1966	57,337	P-NV	104,152	NP-NV	10,806	E4	172,295	84,886	87,410	29,838	142,457
1967	53,652	P-NV	91,970	NP-NV	10,209	E4	155,831	75,559	80,272	26,722	129,108
1968	63,210	P-NV	104,459	NP-NV	9,611	E4	177,280	87,453	89,828	29,034	148,246
1969	48,943	P-NV	89,131	NP-NV	9,014	E4	147,088	73,227	73,861	25,156	121,931

Table 10. Annual ground-water pumpage in the San Bernardino area, California, 1945–98—Continued.

[Values in acre-feet; na, not applicable; source, refer bottom of table for description of source of annual pumpage values]

Calendar year	Plantiff		Non-plaintiff				Total pumpage	Gross pumpage per model layer		Return flow	Total net pumpage
	Filed		Filed		Non-filed						
	Pumpage	Source	Pumpage	Source	Pumpage	Source		Upper	Lower		
1970	55,900	P-V	111,146	NP-V	8,416	NP-NFV	175,462	86,958	88,504	30,614	144,848
1971	61,015	P-V	115,077	NP-V	6,169	NP-NFV	182,261	93,158	89,103	31,212	151,049
1972	53,992	P-V	117,650	NP-V	6,735	NP-NFV	178,377	88,231	90,146	31,031	147,346
1973	49,752	P-V	101,447	NP-V	5,690	NP-NFV	156,889	75,886	81,003	26,803	130,086
1974	52,226	P-V	100,264	NP-V	5,531	NP-NFV	158,021	73,678	84,343	26,923	131,098
1975	56,800	P-V	97,365	NP-V	4,511	NP-NFV	158,676	77,326	81,350	24,652	134,024
1976	61,532	P-V	93,049	NP-V	4,047	NP-NFV	158,628	81,221	77,407	23,300	135,328
1977	58,889	P-V	96,783	NP-V	4,864	NP-NFV	160,536	82,405	78,131	25,085	135,452
1978	49,172	P-V	87,754	NP-V	3,928	NP-NFV	140,854	68,827	72,027	21,953	118,901
1979	59,759	P-V	91,385	NP-V	3,825	NP-NFV	154,969	77,305	77,664	22,997	131,972
1980	56,773	P-V	94,244	NP-V	4,797	NP-NFV	155,814	80,864	74,950	24,009	131,805
1981	58,996	P-V	112,431	NP-V	4,431	NP-NFV	175,858	89,976	85,882	27,214	148,644
1982	56,158	P-V	93,659	NP-V	4,128	NP-NFV	153,945	77,894	76,051	22,679	131,266
1983	49,629	P-V	84,122	NP-V	3,785	NP-NFV	137,536	64,394	73,142	20,844	116,692
1984	68,830	P-V	113,030	NP-V	4,574	NP-NFV	186,434	93,308	93,126	26,872	159,562
1985	71,770	P-V	119,117	NP-V	4,325	NP-NFV	195,212	91,107	104,105	27,942	167,270
1986	70,341	P-V	115,718	NP-V	4,487	NP-NFV	190,546	92,164	98,382	27,969	162,577
1987	66,869	P-V	124,000	NP-V	4,176	NP-NFV	195,045	94,955	100,090	28,642	166,403
1988	74,432	P-V	131,297	NP-V	4,685	NP-NFV	210,414	102,932	107,482	31,558	178,856
1989	60,919	P-V	132,986	NP-V	4,929	NP-NFV	198,834	94,964	103,870	32,277	166,557
1990	55,675	P-V	134,355	NP-V	4,573	NP-NFV	194,603	89,501	105,102	32,085	162,518
1991	60,379	P-V	124,138	NP-V	4,352	NP-NFV	188,869	82,593	106,276	30,348	158,521
1992	58,423	P-V	121,767	NP-V	4,223	NP-NFV	184,413	82,366	102,047	30,232	154,181
1993	60,796	P-V	122,509	NP-V	4,022	NP-NFV	187,327	88,308	99,019	29,373	157,954
1994	69,889	P-V	129,185	NP-V	4,167	NP-NFV	203,241	95,490	107,751	29,951	173,290

Table 10. Annual ground-water pumpage in the San Bernardino area, California, 1945–98—Continued.

[Values in acre-feet; na, not applicable; source, refer bottom of table for description of source of annual pumpage values]

Calendar year	Plantiff		Non-plaintiff				Total pumpage	Gross pumpage per model layer		Return flow	Total net pumpage
	Filed		Filed		Non-filed						
	Pumpage	Source	Pumpage	Source	Pumpage	Source		Upper	Lower		
1995	63,495	P-V	126,213	NP-V	4,155	NP-NFV	193,863	90,731	103,132	28,900	164,963
1996	72,790	P-V	138,295	NP-V	3,614	NP-NFV	214,699	102,389	112,310	32,001	182,698
1997	69,547	P-V	137,510	NP-V	3,475	NP-NFV	210,532	97,324	113,208	31,624	178,908
1998	62,084	P-NV	116,988	NP-NV	3,475	E5	182,547	85,455	97,092	26,390	156,157
Average	59,044	na	107,512	na	8,163	na	174,719	88,810	85,909	28,173	146,546

DESCRIPTION OF SOURCE OF PUMPAGE DATA

Western–San Bernardino Watermaster known pumpage

P-NV Plaintiff, filed, non-verified

P-V Plaintiff, filed, verified

NP-NV Non-plaintiff, filed, non-verified

NP-V Non-plaintiff, filed, verified

NP-NFV Non-plaintiff, non-filed, verified

Estimated pumpage

E1 Average plaintiff, non-verified pumpage for 6-year period (1947–52), scaled to percent runoff in the Santa Ana River.

E2 Average non-plaintiff, non-verified pumpage for 6-year period (1947–52), scaled to percent runoff in the Santa Ana River.

E3 Non-plaintiff, non-filed, verified values for 1959.

E4 Average non-plaintiff, non-filed, verified pumpage for 4-year period (1962–63, 1970–71), scaled to declining linear trend from 1963 to 1970.

E5 Non-plaintiff, non-filed, verified values for 1997.

Table 12. Comparison between past and present ground-water flow models of the San Bernardino area, California.

[ASF, annual specified flux; CSF, constant specified flux; HDF, head-dependent flux]

Item	Past ground-water flow model (Hardt and Hutchinson, 1980)	Present ground-water flow model (this study)
Major benefits	First three-dimensional simulation of ground-water flow in the San Bernardino area.	Simulation of individual recharge and discharge components; revised pumpage and boundary conditions; improved simulation of faults; longer simulation period.
Major limitations	Most recharge and discharge components lumped into a single array; fixed areal distribution of recharge; time invariant underflow; non-filed pumpage missing.	Time invariant transmissivity; idealized aquifer geometry; lack of ground-water quality data to confirm model results.
	General characteristics	
Areal extent	Most of the valley-fill aquifer within the Bunker Hill and Lytle Creek basins.	All of the valley-fill aquifer within the Bunker Hill and Lytle Creek basins.
Computer code	Finite-element code (Durbin, 1978).	Modular finite-difference code (McDonald and Harbaugh, 1988).
Size and number of model cells	296 variably-sized triangles per layer, each triangle with an average area of about 0.60 square miles.	5,337 identical squares per layer, 250 meters on a side, each square with an area of about 0.025 square miles
Number of layers	Two transmissive layers separated by a confining layer.	Two transmissive layers separated by a confining layer.
Simulated period	Calendar years 1945–72.	Calendar years 1945–98.
Length of uniform stress	1 year.	1 year.
	Aquifer parameters	
Transmissivity	Variable per cell; time invariant.	Variable per cell; time invariant; values interpolated from model values used by Hardt and Hutchinson (1980).
Storage	Variable per cell.	Variable per cell in the upper model layer using values interpolated from Eckis (1934); constant (0.0001) for cells in the lower model layer.
Confining layer	Vertical leakage through a confining layer; restriction to vertical flow occurs only in the middle part of the basin, designated as the confining area (fig. 28C).	Same methology (fig. 40D)
Faults	Reduced value of transmissivity in model cells coincident with faults.	Simulated with the horizontal-flow barrier package between model cells (Hsieh and Freckleton, 1993).
	Recharge and discharge	
Recharge from precipitation	None.	CSF; small amount based on the areal distribution of average precipitation.

Table 12. Comparison between past and present ground-water flow models of the San Bernardino area, California—Continued.

[ASF, annual specified flux; CSF, constant specified flux; HDF, head-dependent flux]

Item	Past ground-water flow model (Hardt and Hutchinson, 1980)	Present ground-water flow model (this study)
Recharge from gaged runoff	ASF; single recharge array scaled, with some variations, to annual discharge in the Santa Ana River.	HDF; simulates interaction between surface and ground water (Prudic, 1989).
Recharge from ungaged runoff	ASF; added to recharge array.	ASF; linearly related to discharge in the Santa Ana River.
Recharge from local runoff	None.	ASF; uses the areal distribution of average precipitation.
Artificial recharge from imported water	ASF; added to recharge array.	ASF.
Underflow	CSF; added to recharge array.	ASF; HDF used for underflow beneath the Santa Ana River.
Evapotranspiration	HDF; includes rising ground water.	HDF; simulates evapotranspiration only.
Pumpage	ASF; measured and estimated values using non-verified data from the local watermaster; omits non-filed pumpage.	ASF; measured and estimated values using verified and non-verified data from the local watermaster; includes non-filed pumpage.
Return flow	ASF; subtracted from pumpage.	ASF; simulated separately.

Table 15. Simulated ground-water budget for the valley-fill aquifer in the San Bernardino area, California, 1945–98.

[All values in acre-feet. Method of calculation and source of recharge and discharge are listed in headings; (negative) change in storage indicates loss of water from the aquifer. Area, shown in figure 22, includes the Bunker Hill and Lytle Creek ground-water basins; (e), estimated; (s), simulated; (c), calculated]

Calendar year	Recharge								Discharge					Change in storage (s)	Residual (c)
	Precipi-tation (e)	Gaged runoff (s)	Ungaged runoff (e)	Local runoff (e)	Im-ported water (e)	Under-flow (e)	Pumpage return flow (e)	Total sum	Gross pump-age (e)	Evapo-transpi-ration (s)	Rising ground water (s)	Under-flow (e)	Total sum		
1945	1,137	120,127	20,139	6,387	0	6,800	19,967	174,558	122,106	23,231	15,152	14,054	174,544	1	13
1946	1,137	120,025	15,460	6,140	0	6,724	22,423	171,910	137,510	20,641	13,021	13,544	184,716	(12,799)	–6
1947	1,137	94,687	10,291	2,042	0	6,648	20,851	135,656	142,900	15,024	10,271	12,976	181,171	(45,508)	–7
1948	1,137	70,673	9,846	4,543	0	6,574	24,537	117,310	158,502	10,666	6,842	12,768	188,778	(71,465)	–4
1949	1,137	75,292	10,526	5,095	0	6,501	24,257	122,808	148,518	9,284	5,831	12,776	176,409	(53,267)	–335
1950	1,137	60,561	8,175	3,088	0	6,429	26,894	106,285	164,585	6,140	3,397	11,419	185,540	(79,247)	–8
1951	1,137	52,582	7,495	5,844	0	6,358	29,187	102,603	169,932	4,518	2,252	9,215	185,918	(83,324)	9
1952	1,137	158,743	17,707	7,442	0	6,287	23,461	214,777	133,607	6,602	3,461	9,046	152,717	62,058	1
1953	1,137	64,246	8,320	2,529	0	6,218	30,280	112,731	183,872	3,295	1,240	8,204	196,611	(83,880)	–1
1954	1,137	106,321	13,303	6,639	0	6,150	28,205	161,755	162,058	3,201	1,189	8,212	174,660	(12,905)	1
1955	1,137	65,643	8,460	4,197	0	6,083	30,464	115,984	183,343	2,265	936	7,450	193,994	(78,021)	12
1956	1,137	60,343	8,320	3,847	0	6,016	34,580	114,243	202,488	1,917	840	6,610	211,855	(97,605)	–7
1957	1,137	69,672	7,966	7,016	0	5,951	29,949	121,691	170,952	1,902	860	5,864	179,579	(57,886)	–2
1958	1,137	231,624	21,300	7,146	0	5,886	28,267	295,360	168,519	8,155	6,452	5,929	189,055	106,310	–5
1959	1,137	68,874	8,175	3,329	0	5,822	33,037	120,374	203,029	3,000	923	5,465	212,417	(92,036)	–8
1960	1,137	55,112	7,633	4,355	0	5,759	32,358	106,355	191,980	2,233	837	5,091	200,141	(93,773)	–14
1961	1,137	37,310	4,811	2,244	0	5,697	36,776	87,976	214,041	1,588	706	4,474	220,809	(132,835)	1
1962	1,137	90,394	10,291	3,613	0	5,636	32,103	143,175	186,380	1,669	864	4,091	193,003	(49,828)	–0
1963	1,137	46,471	5,348	5,952	0	5,576	30,463	94,947	176,386	1,371	724	3,647	182,128	(87,169)	–12
1964	1,137	42,963	5,467	3,390	0	5,516	33,085	91,559	190,534	1,168	633	3,410	195,745	(104,187)	0
1965	1,137	134,743	12,653	7,451	0	5,457	30,752	192,194	169,664	1,499	905	3,249	175,318	16,858	18
1966	1,137	171,347	20,139	5,063	0	5,399	29,685	232,771	171,620	2,017	1,215	3,047	177,900	54,866	5
1967	1,137	186,326	18,871	5,880	0	5,342	26,564	244,120	155,196	5,831	5,381	2,988	169,396	74,715	9
1968	1,137	80,113	9,173	2,574	0	5,286	28,865	127,148	176,712	2,862	969	2,871	183,414	(56,268)	2
1969	1,137	400,198	66,748	10,965	0	5,230	25,013	509,291	146,606	11,308	15,129	2,886	175,928	333,359	4

Table 15. SImulated ground-water budget for the valley-fill aquifer in the San Bernardino area, California, 1945–98—Continued.

[All values in acre-feet. Method of calculation and source of recharge and discharge are listed in headings; (negative) change in storage indicates loss of water from the aquifer. Area, shown in figure 22, includes the Bunker Hill and Lytle Creek ground-water basins; (e), estimated; (s), simulated; (c), calculated]

Calendar year	Recharge								Discharge					Change in storage (s)	Residual (c)
	Precipi-tation (e)	Gaged runoff (s)	Ungaged runoff (e)	Local runoff (e)	Im-ported water (e)	Under-flow (e)	Pumpage return flow (e)	Total sum	Gross pump-age (e)	Evapo-transpi-ration (s)	Rising ground water (s)	Under-flow (e)	Total sum		
1970	1,137	104,569	11,583	4,991	0	5,175	30,438	157,893	174,878	4,672	1,906	3,018	184,474	(26,551)	–30
1971	1,137	80,164	9,605	4,619	0	5,121	30,985	131,631	181,503	3,069	1,224	3,207	189,004	(57,374)	2
1972	1,137	59,995	7,170	2,351	1,208	5,067	30,831	107,759	177,714	2,346	948	3,330	184,338	(76,586)	7
1973	1,137	142,164	17,518	4,978	30,419	5,014	26,683	227,914	156,494	3,118	1,355	3,500	164,468	63,459	–13
1974	1,137	97,351	11,448	5,166	15,222	4,962	26,765	162,052	157,498	2,735	1,199	3,717	165,149	(3,112)	15
1975	1,137	76,841	9,605	4,094	13,275	4,911	24,468	134,331	158,061	2,426	1,079	3,764	165,330	(30,995)	–4
1976	1,137	79,867	9,480	5,171	11,794	4,860	23,170	135,480	158,200	2,292	1,047	3,752	165,292	(29,805)	–6
1977	1,137	64,689	7,170	5,117	22,653	4,810	24,966	130,543	160,145	2,395	1,472	3,729	167,741	(37,203)	6
1978	1,137	367,931	33,981	10,580	12,279	4,760	21,903	452,571	140,691	12,045	11,305	3,862	167,902	284,680	–11
1979	1,137	230,248	31,633	5,646	4,230	4,711	22,865	300,471	154,528	15,946	13,773	4,524	188,772	111,704	–4
1980	1,137	233,890	67,711	9,116	4,191	4,663	23,862	344,571	155,325	23,703	28,253	5,796	213,076	131,487	8
1981	1,137	94,289	10,291	3,950	14,321	4,616	27,097	155,700	175,469	10,543	6,951	7,160	200,123	(44,425)	3
1982	1,137	135,714	18,943	7,864	8,544	4,569	22,551	199,322	153,520	11,545	5,886	8,509	179,460	19,859	3
1983	1,137	204,569	50,283	11,796	1,404	4,522	20,723	294,436	137,137	26,578	29,462	9,626	202,803	91,615	18
1984	1,137	108,536	11,986	3,065	361	4,476	26,742	156,303	186,002	11,559	5,534	10,060	213,155	(56,849)	–3
1985	1,137	83,675	9,480	3,740	1,108	4,431	27,813	131,385	194,783	6,701	2,369	9,995	213,848	(82,363)	–100
1986	1,137	121,958	13,303	5,328	0	4,387	27,836	173,950	190,105	6,097	2,107	9,070	207,379	(33,421)	–8
1987	1,137	63,520	7,495	4,471	81	4,342	28,515	109,562	194,627	3,570	1,006	8,159	207,362	(97,804)	4
1988	1,137	60,606	6,474	4,180	0	4,299	31,437	108,133	210,010	2,588	720	7,189	220,507	(112,357)	–17
1989	1,137	48,661	5,467	2,481	4,138	4,256	32,155	98,295	198,429	1,972	655	6,164	207,220	(108,920)	–4

Table 15. SImulated ground-water budget for the valley-fill aquifer in the San Bernardino area, California, 1945–98—Continued.

[All values in acre-feet. Method of calculation and source of recharge and discharge are listed in headings; (negative) change in storage indicates loss of water from the aquifer. Area, shown in figure 22, includes the Bunker Hill and Lytle Creek ground-water basins; (e), estimated; (s), simulated; (c), calculated]

Calendar year	Recharge								Discharge					Change in storage (s)	Residual (c)
	Precipi-tation (e)	Gaged runoff (s)	Ungaged runoff (e)	Local runoff (e)	Im-ported water (e)	Under-flow (e)	Pumpage return flow (e)	Total sum	Gross pump-age (e)	Evapo-transpi-ration (s)	Rising ground water (s)	Under-flow (e)	Total sum		
1990	1,137	36,506	3,977	2,810	6,497	4,213	32,083	87,224	194,598	1,235	620	5,804	202,257	(115,036)	3
1991	1,137	83,704	8,175	6,123	1,014	4,172	30,346	134,671	188,864	1,594	679	4,712	195,849	(61,189)	11
1992	1,137	120,025	12,181	6,899	117	4,130	30,230	174,720	184,409	1,607	605	4,112	190,733	(16,002)	–12
1993	1,137	379,934	52,482	9,022	2,254	4,089	29,371	478,290	187,323	11,708	9,920	4,057	213,007	265,269	14
1994	1,137	90,814	11,042	4,758	3,633	4,049	29,949	145,381	203,236	4,324	1,442	3,882	212,884	(67,501)	–2
1995	1,137	266,055	38,408	7,424	0	4,009	28,898	345,931	193,858	14,767	10,534	3,727	222,885	123,039	7
1996	1,137	123,131	14,265	6,418	93	3,970	31,999	181,013	214,695	5,939	2,153	3,871	226,658	(45,635)	–10
1997	1,137	102,672	11,042	5,956	0	3,931	31,622	156,360	210,527	4,334	1,419	3,750	220,031	(63,669)	–1
1998	1,137	239,066	35,918	9,951	151	3,892	26,388	316,504	182,543	16,485	12,105	4,343	215,475	101,028	1
1945–98 Maxi-mum	1,137	400,198	67,711	11,796	30,419	6,800	36,776	509,291	214,695	26,578	29,462	14,054	226,658	333,359	18
1945–98 Average	1,137	121,584	16,199	5,460	2,944	5,181	28,050	180,555	174,189	6,839	4,773	6,216	192,017	(11,454)	–8
1983–98 Average	1,137	133,340	18,249	5,901	1,303	4,198	29,132	193,260	191,947	7,566	5,083	6,158	210,753	(17,487)	–6
1945–98 Mini-mum	1,137	36,506	3,977	2,042	0	3,892	19,967	87,224	122,106	1,168	605	2,871	152,717	(132,835)	–335

Table 19. Description of water-management scenarios simulated by the ground-water flow and constrained optimization models for the San Bernardino area, California.

Scenario	Purpose	Description	Result
Scenario 1—Average recharge and pumpage, 1999–2030.	Show the general trend of future surface-water flows and ground-water levels if present (1998) water-management operations are continued.	This scenario is the constant version of scenario 2. Runoff, recharge, and pumpage equal the average of annual values for the period 1983–98. No water is imported from Northern California.	In nearly all areas, ground-water levels flatten within a couple of years, indicating that the aquifer is highly transmissive and that recharge and discharge are approximately in balance (fig. 56). During the 32-year period, ground-water levels tend to rise in the former marshland and decline in the outlying areas.
Scenario 2—Annual variations in recharge and pumpage, 1999–2030.	Show the range of effects of continuing present (1998) water-management operations, assuming recent (1983–98) annual variations in recharge and pumpage.	This scenario is the base case for scenarios 1, 3, 4, and 6. Runoff, recharge, and pumpage equal annual values for the period 1983–98, repeated twice. No water is imported from Northern California.	In most areas, ground-water levels rise and fall within an acceptable range that prevents damage from elevated hydrostatic presssure, liquefaction, or land subsidence (fig. 57). High ground-water-level conditions, however, occur in the former marshland during wetter periods.
Scenario 3—Increase artificial recharge, 1999–2030.	Show the effects of increasing artificial recharge of either imported water or local runoff made available for recharge by construction of Seven Oaks Dam.	Uses values of runoff, recharge, and pumpage from scenario 2, plus additional artifical recharge ranging from 3,500 to 25,000 acre-feet per year in the Santa Ana River recharge basins.	Ground-water levels show little change compared to scenario 2 in most areas (figs. 57–58). Much of the additional recharge is discharged to the streams or increaes underflow out of the aquifer, decreasing the effect of the extra recharge.
Scenario 4—Increase ground-water pumpage, 1999–2030.	Show the effects of increasing ground-water pumpage using recent (1994–98) excess well capacity.	Uses values of runoff, recharge, and pumpage from scenario 2. Extra pumpage is added equal to the difference between maximum and average pumpage for each well during 1994–98. No water is imported from Northern California. Four test cases (a–d) (fig. 60) are evaluated, each using wells located progessively further from the basin outflow.	In most areas, ground-water levels fall significantly if net pumpage exceeds about 15,000 acre-feet per year, which can be achieved using wells within 10,000 feet of basin outflow (fig. 59). Net pumpage of more than about 35,000 acre-feet per year, which can be obtained within the former marshland, causes precipitious declines likely to prompt additional land subsidence.
Scenario 5—Optimal hydraulic containment of contaminated ground water in the Newmark area.	Determine the minimum pumpage needed to prevent further migration of trichloroethylene (TCE) and perchloroethylene (PCE) contamination associated with the Newmark U.S. Environmental Protection Agency Superfund site.	Uses values of runoff, recharge, and pumpage from scenario 2 except all pumpage within the plume is set to zero. Identifies the minimum pumpage needed to prevent further migration of the contamination by determining optimal pumpage at five planned and four hypothetical barrier well sites located along the leading (south) edge of the contamination (fig. 61). Constraints on hydraulic gradients around the edge of the plume are used to assure that the plume does not spread laterally or bypass the nine barrier wells.	Maximum pumpage from the five planned wells appears to be insufficient to prevent further migration of the contamination (figs. 61–62). If all nine well sites are used, the leading edge of the contamination can be contained. No realistic quantity of pumpage from the nine barrier well sites will restrict lateral, east-west migration of the contamination.

Table 19. Description of water-management scenarios simulated by the ground-water flow and constrained optimization models for the San Bernardino area, California—Continued.

Scenario	Purpose	Description	Result
Scenario 6—Optimal management of ground-water levels in the former marshland.	Determine the minimum total pumpage from hypothetical well sites along proposed extensions of the Baseline feeder pipeline in order to maintain safe ground-water levels within the vicinity of the former marshland.	Uses values of runoff, recharge, and pumpage from scenario 2. Pumpage is determined at 29 possible well sites along three proposed extensions of the Baseline feeder pipeline (fig. 63). Ground-water levels are constrained at 58 locations within the former marshland to be no higher than 30 feet below land surface and no lower than the minimum simulated value during 1945–98.	Results indicate that the problem is infeasible with the assumed distribution of wells, even if the constraint is changed to 10 feet below land surface (figs. 64–65). About 20,000 acre-feet of additional pumpage is needed to satisfy the constraints at most locations during most years. Nearly all infeasiblities are caused by ground-water levels being too high during the occasional wet years.

Table 20. Recharge and discharge components for scenarios 1–6 for the San Bernardino area, California, 1999–2030.

[See table 19 for definition of scenarios. All values vary annually, unless noted as constant. Calibration is calibration of the ground-water flow model. Included, additional recharge or discharge component noted as included in selected scenarios; —, not included]

Recharge or discharge component	Calibration	Scenario					
		1	2	3	4	5	6
Recharge from direct precipitation	Small contant value (table 4)	Calibration	Calibration	Calibration	Calibration	Calibration	Calibration
Recharge from gaged runoff	Measured annual discharge for gaged streams used in a stream-flow-routing model linked to the ground-water flow model (table 1)	Average runoff values from scenario 2	Annual runoff values for 1983–98, repeated twice	Same runoff values as scenario 2	Same runoff values as scenario 2	Same runoff values as scenario 2	Same runoff values as scenario 2
Recharge from ungaged runoff	Estimates based on drainage area, precipitation, and runoff percentage, then scaled by percent annual runoff in the Santa Ana River (tables 2 and 3)	Average values from scenario 2	Annual values for 1983–98, repeated twice	Scenario 2	Scenario 2	Scenario 2	Scenario 2
Recharge from local runoff	Estimates based on annual precipitation (table 4)	Average values from scenario 2	Annual values for 1983–98, repeated twice	Scenario 2	Scenario 2	Scenario 2	Scenario 2
Recharge from imported water	Estimated from measured annual values of imported water (table 5)	Zero	Zero	Zero	Zero	Zero	Zero
Underflow from the Badlands, Redlands Heights, and Reservoir Canyon	Small constant values (table 9; figure 59)	Calibration	Calibration	Calibration	Calibration	Calibration	Calibration
Underflow from San Timoteo Canyon	Estimated from regression equation (3) (table 9, figs. 26-27)	Estimated from regression equation (3) (fig. 53)	Scenario 1	Scenario 1	Scenario 1	Scenario 1	Scenario 1
Underflow from Sand Canyon	Estimated from regression equation (4) (table 9, figs. 26-27)	Estimated from regression equation (4) fig.27)	Scenario 1	Scenario 1	Scenario 1	Scenario 1	Scenario 1
Underflow across the San Jacinto fault near Barrier J	Constant value (table 9)	Calibration	Calibration	Calibration	Calibration	Calibration	Calibration

Table 20. Recharge and discharge components for scenarios 1–6 for the San Bernardino area, California, 1999–2030—Continued.

[See table 19 for definition of scenarios. All values vary annually, unless noted as constant. Calibration is calibration of the ground-water flow model. Included, additional recharge or discharge component noted as included in selected scenarios; —, not included]

Recharge or discharge component	Calibration	Scenario					
		1	2	3	4	5	6
Underflow across the San Jacinto fault near the Santa Ana River	Head-dependent discharge calculated from measured head in the Heap well (table 9, fig.28)	Head-dependent discharge calculated from simulated head in the model cell for the Heap well, equation 5 (fig. 28)	Same head-dependent relation used in scenario 1	Same head-dependent relation used in scenario 1	Same head-dependent relation used in scenario 1	Same head-dependent relation used in scenario 1	Same head-dependent relation used in scenario 1
Pumpage	Reported and estimated annual values (table 10)	Average values from scenario 2	Annual values for 1983–98, repeated twice	Scenario 2	Scenario 2	Scenario 2	Scenario 2
Return flow	Estimated annual values based on constant percentage of pumpage per well	Average values from scenario 2	Annual values for 1983–98, repeated twice	Scenario 2	Scenario 2	Scenario 2	Scenario 2
Evapotranspiration	Head-dependent value calculated from estimated maximum evapotranspiration	Same head-dependent relation used in calibration	Same head-dependent relation used in calibration	Same head-dependent relation used in calibration	Same head-dependent relation used in calibration	Same head-dependent relation used in calibration	Same head-dependent relation used in calibration
Recharge of additional native water	Not included in calbration	—	—	Included	—	—	—
Recharge of additional native or imported water	ditto	—	—	—	—	—	—
Additional pumpage using excess well capacity	ditto	—	—	—	Included	—	—
Additional pumpage using barrier wells near the leading edge of Newmark plume	ditto	—	—	—	—	Included	—
Additional pumpage using possible wells along extensions of the Baseline feeder pipeline	ditto	—	—	—	—	—	Included