Hydraulic Properties of a Fractured-Rock Aquifer, Lee Valley, San Diego County, California

United States Geological Survey Water-Supply Paper 2394

Prepared in cooperation with San Diego County Department of Planning and Land Use

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By CHARLES A. KAEHLER and PAUL A. HSIEH

Prepared in cooperation with San Diego County Department of Planning and Land Use

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CONVERSION FACTORS AND DEFINITIONS OF TERMS

Conversion Factors

Multiply	By	To obtain
foot (ft)	0.3048	meter
foot per second (ft/s)	0.3048	mèter per second
foot per minute (ft/min)	0.3048	meter per minute
foot per day (ft/d)	0.3048	meter per day
foot squared per day (ft^2/d)	0.09290	meter squared per day
foot per mile (ft/mi)	0.1894	meter per kilometer
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
acre-foot (acre-ft)	0.001233	cubic hectometer
acre-foot per year (acre-ft/yr)	0.001233	cubic hectometer per annum
gallon per minute (gal/min)	0.06309	liter per second
gallon per day (gal/d)	3.785	liter per day
inch (in.)	25.4	millimeter
mile (mi)	1.609	kilometer
square mile (mi^2)	2.590	square kilometer

Air temperature is given in degrees Fahrenheit (°F), which can be converted to degrees Celsius (°C) using the equation:

$$\text{Temp. °C} = (\text{temp. °F} - 32)/1.8$$

Definitions of Terms

In this report, **rain year** refers to the 12-month period that begins July 1 and ends June 30. It is designated by the two calendar years of the period (for example, rain year 1986–87).

The **water year** starts October 1 and ends September 30. It is designated by the calendar year in which it ends and which includes 9 of the 12 months.

In this report, **sea level** refers to the National Geodetic Vertical Datum of 1929—a geodetic datum derived from a general adjustment of the first-order level nets of the United States and Canada, formerly called Sea Level Datum of 1929.

Hydraulic Properties of a Fractured-Rock Aquifer, Lee Valley, San Diego County, California

By Charles A. Kaehler *and* Paul A. Hsieh

Abstract

Lee Valley is a 2.25-square-mile basin in mountainous granitic terrane about 18 miles east of San Diego, California. Water supply in Lee Valley, whose setting is typical of much of eastern San Diego County, is derived entirely from ground water in a fractured-rock aquifer.

The aquifer in Lee Valley consists of three zones. From top to bottom, they are (1) regolith, composed primarily of weathered granodiorite and fine-grained gabbro; (2) a transition zone of highly fractured and partly weathered rock; and (3) unweathered bedrock that is fractured to various degrees. The fluctuation of water levels in wells penetrating the three zones indicates that they form a dynamic, interactive, ground-water flow system.

The reported yields of wells drilled in Lee Valley are characteristic of fractured-rock aquifers with a high degree of heterogeneity. Yields generally are low (median is 15 gallons per minute), but a few high-yielding wells (greater than 100 gallons per minute) have been reported. Areal distribution of well yields does not show any discernible pattern, and wells located near one another can have large differences in yield. There is some evidence that wells drilled near lineaments are likely to yield larger quantities of water than wells drilled at some distance from lineaments.

Aquifer tests were done at five sites to determine the hydraulic properties of the fractured-rock aquifer. These tests were designed to take into account the heterogeneous nature of the aquifer—in particular, the possible presence of regions of low or high hydraulic conductivity surrounding the pumped or observation wells. On the basis of these tests, the horizontal hydraulic conductivity of the regolith was determined to be about 1×10^{-4} feet per second, and its vertical hydraulic conductivity was about one-fifth of the horizontal. For the transition zone, horizontal hydraulic conductivity ranged from 3×10^{-6} to 4×10^{-5} feet per second. Where the transition zone was locally confined, the vertical hydraulic conductivity seemed to be lower than the horizontal hydraulic conductivity. The reverse was found where the transition zone showed water-table drainage. This observation may be explained by the requirement that a large number of vertical fractures be present in order for a water table to exist. The hydraulic conductivity of the unweathered bedrock seems to be highly variable, ranging from near zero (at wells with nonproductive bedrock) to 7×10^{-7} feet per second. However, no tests were done in wells drilled near lineaments. Bedrock penetrated by wells near or on lineaments probably would show substantially higher hydraulic conductivities.

Specific yields determined from aquifer tests were about 1×10^{-2} for the regolith and 6×10^{-3} to 2×10^{-2} for the transition zone. Specific storage ranged from 1×10^{-6} to 4×10^{-5} per foot for the transition zone and was 2×10^{-7} per foot for the bedrock.

Recharge to the aquifer primarily is from infiltration of precipitation. Ground water flows from the north, west, and east sides of the valley toward Jamul Creek, which drains the valley floor. Discharge of ground water occurs by pumping, as stream base flow, and as evapotranspiration, principally by oak trees that tap the water

table along Jamul Creek. For water year 1988 (October 1, 1987, to September 30, 1988), recharge to the ground-water system is estimated to have been 160 acre-feet, and the volume of water in storage in the aquifer to have been 640 acre-feet. Because of annual variations in precipitation, recharge may vary significantly from year to year. Thus, recharge commonly is greater or less than discharge for a given water year; the difference is made up by change in storage. If below-average recharge were to occur over a period of several years, especially in combination with increased use of ground water, a substantial depletion of ground water in storage could result within about a decade.

INTRODUCTION

Residents of eastern San Diego County rely almost entirely on the local ground water for their water supplies. Most of the water is pumped from individual wells that tap the underlying fractured-rock aquifer. As a result of a growing population in this area, an increasing demand is being placed on the ground-water resources. Current knowledge of the area's fractured-rock hydrology is minimal, and additional knowledge would be useful in making future management and regulatory decisions that could help prevent depletion of ground-water resources.

As a result of these considerations, the U.S. Geological Survey (USGS) entered into a cooperative agreement with the primary management agency, the San Diego County Department of Planning and Land Use, to study the hydraulics of a fractured-rock area and to help determine the long-term availability of water.

Since 1983, the USGS has been cooperating with the Department of Planning and Land Use on a hydrologic study of limited scope in Lee Valley. This 2.25-mi^2 basin is in granitic terrane about 18 mi east of San Diego, California (fig. 1). Because the mountainous topography and crystalline-rock geologic setting of the area are typical of most of the eastern part of the county, this area was selected for further study. From 1983 to 1986, the focus of work in Lee Valley was on monitoring of water levels and on defining elements of the water budget, which include precipitation, stream runoff, ground-water recharge, pumpage, and evapotranspiration. The focus of the current (1986–88) study, using existing and new water-level and water-budget information as well as aquifer-test data collected during the current phase of the study, was on ground-water hydraulics of the fractured-rock aquifer. The study results are presented in this report.

Purpose and Scope

This report presents the results of a study whose purpose was to define the ground-water hydraulic properties of the fractured-rock aquifer in Lee Valley. The study concentrated specifically on the following:

1. The development of appropriate aquifer-test and analysis methods needed to define the hydraulic properties (including the horizontal and vertical hydraulic conductivities and storage properties), both areally and with depth; and
2. The sources and quantities of ground-water recharge and discharge.

Several methods were used in this study to define the ground-water hydrology of Lee Valley. The nature of the fractured-rock aquifer was examined by test drilling, geophysical logging, and mapping of fractures and lineaments. Movement of ground water was determined using water-level measurements and chemical analyses of water samples collected from wells at different locations and depths. Data from aquifer tests at five sites were analyzed using existing and newly developed analytical methods to determine hydraulic conductivities and storage coefficients. The ground-water budget of the valley was estimated by monitoring rainfall at three locations, streamflow draining the valley, domestic consumption of ground water at six households, and water levels at about 60 wells.

Description of the Study Area

Lee Valley is about 18 mi east of San Diego in the western part of the Peninsular Ranges batholith. The surface-drainage boundary, which has a minimum altitude of 1,480 ft and a maximum altitude of 2,840 ft, encloses an area of 2.25 mi^2. The valley floor trends northwesterly, forming an inverted "Y" shape, and has an area of about 1 mi^2 (figs. 1, 2). The valley is drained by a small ephemeral stream, Jamul Creek, which has a main branch on the west side of the valley and tributaries that enter primarily from the east. Generally, the last 1,000-ft reach of Jamul Creek at the lower (southern) end of the basin contains flow from November or December to May or June. Other sections farther upstream may contain flow for several days after heavy rains.

Granitic rock is exposed on the hillsides surrounding the valley floor. The exposures consist of large rounded boulders and massive faces of fractured rock (fig. 2*A*, *C*). The uplands are densely covered with brush (chamise, Ramona lilac, and flat-topped buckwheat) between the rock outcrops. Outcrops constitute about 50 percent of the steep hillsides in the southeast part of the basin and about 25 percent of the remaining upland areas. The northeast-facing slope on the western side of the basin has especially dense vegetation, including areas of Engleman oak.

The valley floor has flat to gently rolling topography (fig. 2*A*, *B*). Holly-leaf oak (*Quercus agrifolia*) is the main type of vegetation, in addition to grasses, on the valley floor. The oaks are scattered in the central part of the valley but grow densely along the main branch of Jamul Creek in the western and southwestern parts of the valley floor. The gentle slopes at the north end of the valley are brush covered (fig. 2*B*).

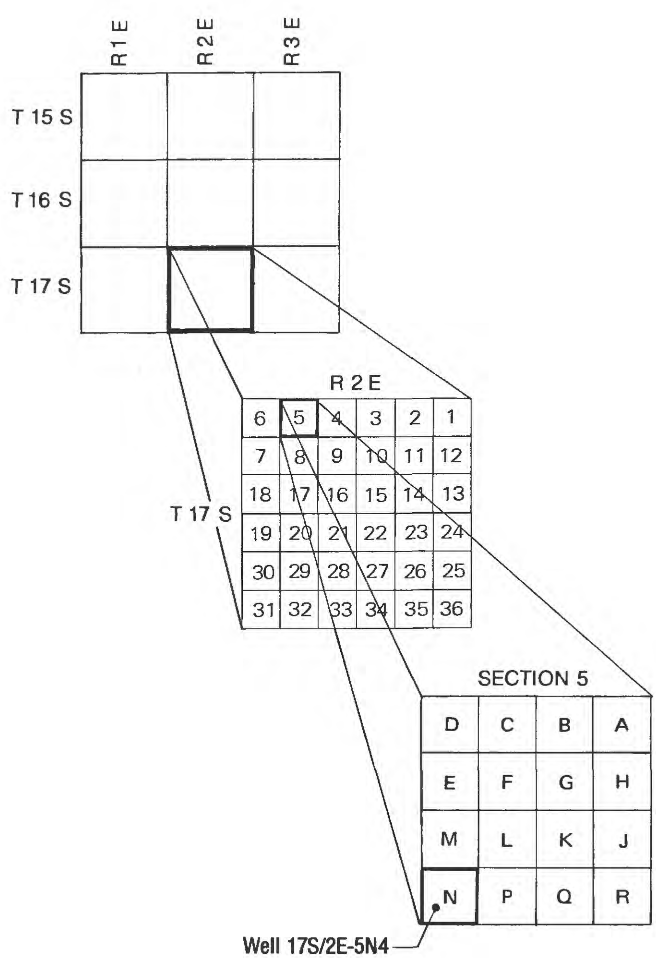
About 50 residences are in Lee Valley, each with property consisting of several acres. In addition, a private campground is in the southwest part of the valley. Ground water is pumped from privately owned wells and is used for household consumption, irrigation of fruit trees and landscape vegetation, and watering of horses and other livestock. Two or three property owners occasionally pump water to fill swimming pools, ponds, or fish-farm tanks.

Well-Numbering System

Wells are numbered in this report according to their location in the rectangular system for the subdivision of public land. The study area lies entirely in the southeast quadrant of the San Bernardino base line and meridian. The part of the number preceding the slash (as in 17S/2E-5N4) indicates the township (T. 17 S.); the number and letter after the slash indicates the range (R. 2 E.); the number following the dash indicates the section (sec. 5); and the letter following the section number indicates the 40-acre subdivision of the section according to the lettered diagram below. The final digit is a serial number for wells in each 40-acre subdivision.

Acknowledgments

The study was done in cooperation with the San Diego County Department of Planning and Land Use. The assistance of John Peterson, Hydrologist, San Diego County, is gratefully acknowledged. Acknowledgment also is extended to Steven Pierce, San Diego State University graduate student and San Diego County Department of Planning and Land Use employee, for his help in data collection, for preparation of a map of lineament and fracture-trace locations, and for compilation of well-yield and borehole fracture-orientation data. In addition, we thank Matt Whedlan, Chris Diercks, and Phil Brown of the San Diego County Department of Planning and Land Use for their help with data collection during 1984–88; and William Batten, Devin Galloway, and Yu Chunming of the USGS for their help in conducting aquifer tests. Special thanks are given to the many property owners in Lee Valley who graciously allowed access to their property and wells. Without their cooperation, data collection would not have been possible.



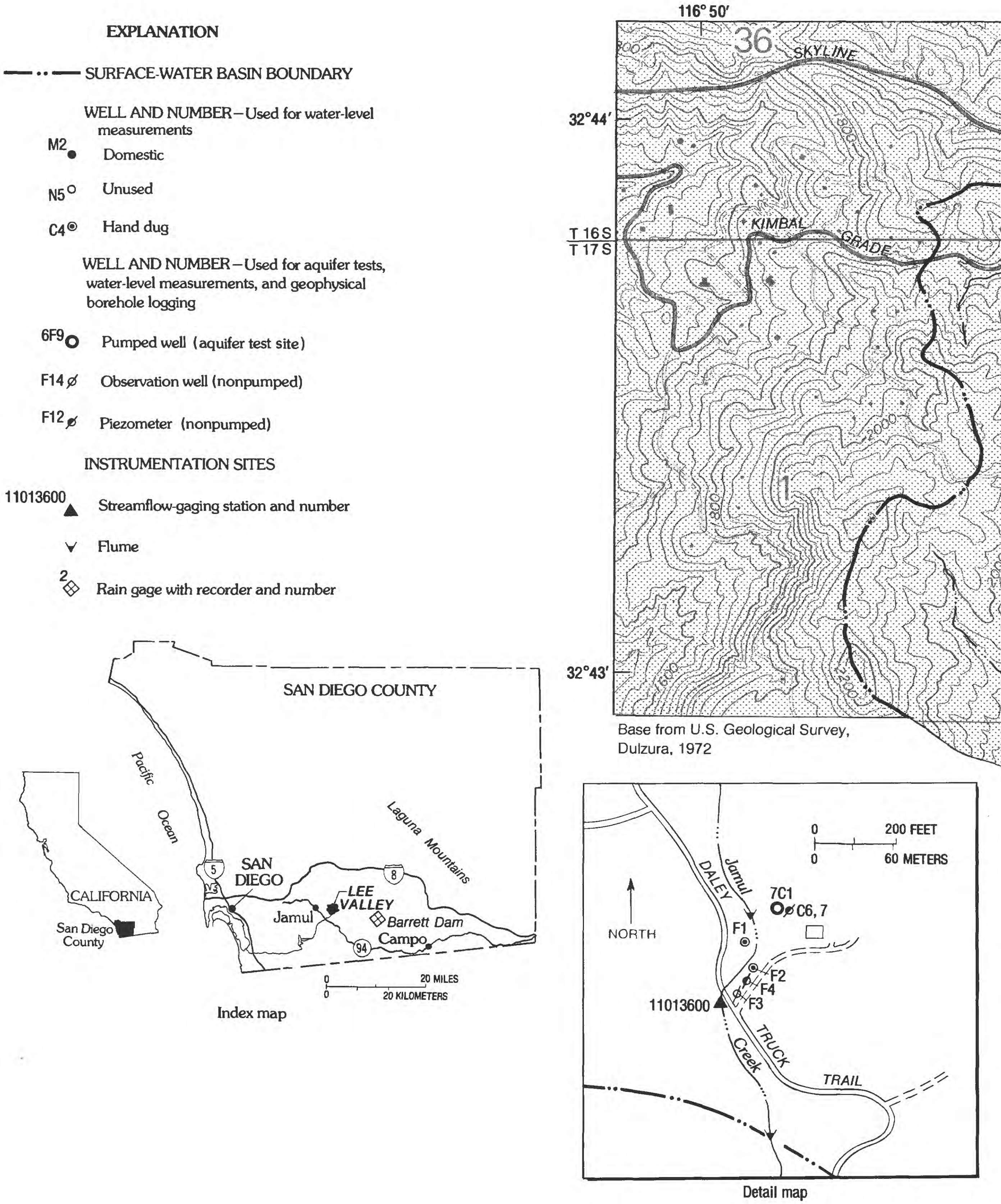


Figure 1. Study area, selected wells, and instrumentation sites.

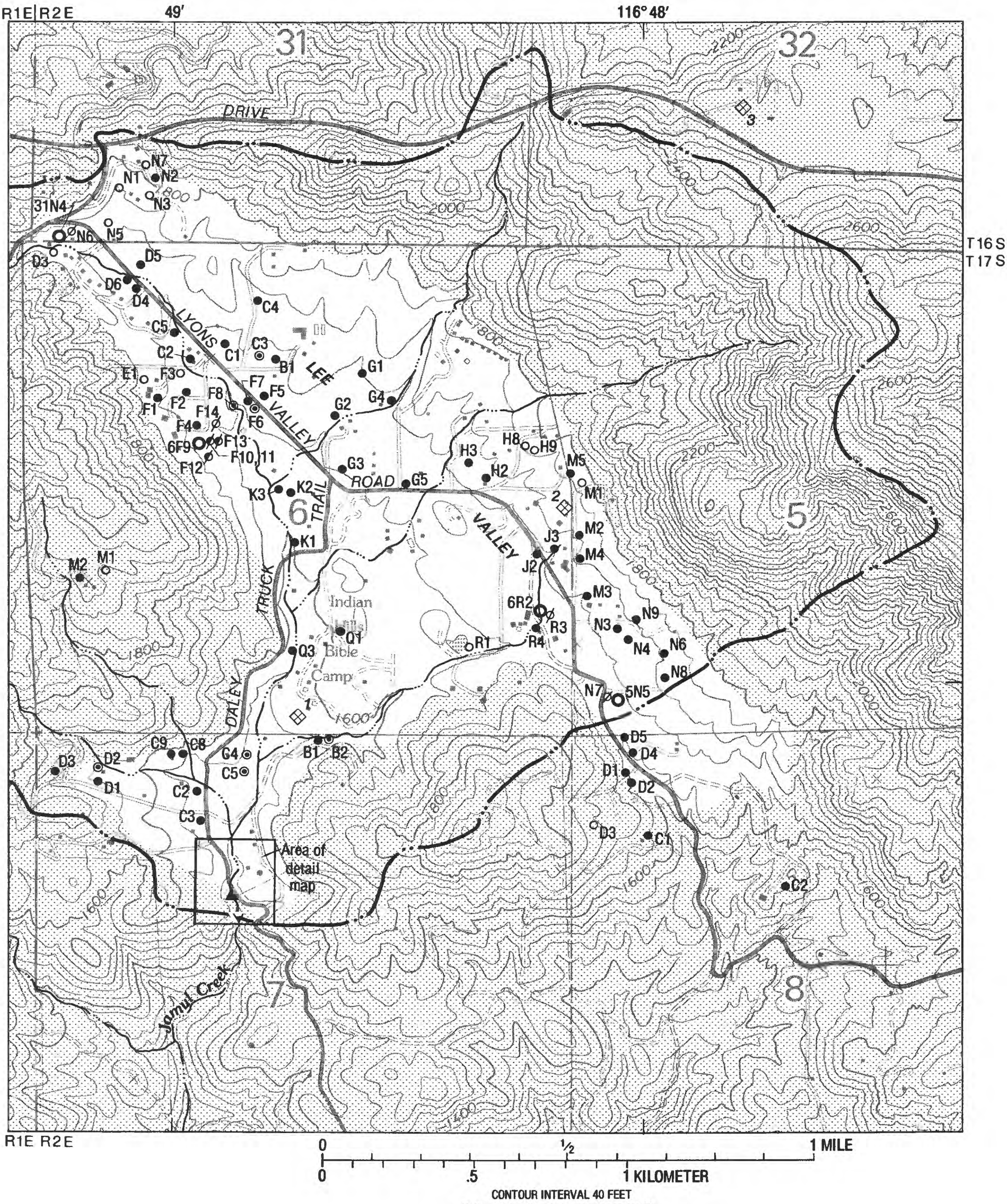


Figure 1. Continued.

A

Figure 2. Landforms, vegetation, and bedrock outcrops in the study area. *A*, Oblique aerial view from above western ridge, looking northeast. Note scattered Engleman oaks on ridge in foreground and holly-leaf oaks along Jamul Creek.

B

B, View from Skyline Drive, looking south-southeast toward discharge end of the basin. Brush and scattered Engleman oaks cover the hillsides, and holly-leaf oaks are present along Jamul Creek (right center) and on the valley flat.

C

C, View from well 17S/2E-5N7, looking east-northeast. The outcrops and boulders are composed of granodiorite. The subvertical faces are fractured granodiorite.

GEOHYDROLOGIC SETTING

Lee Valley is in the western part of the Peninsular Ranges batholith, which extends more than 500 mi from the southern part of Baja California in Mexico north to the Riverside area in southern California. Lee Valley is one of many small valleys within the batholith. Lee Valley trends northwest, parallel to one of the major structural trends of the region. The valley floor, which has an altitude of about 1,600 ft, is higher than those of the surrounding valleys.

Lithology

The Peninsular Ranges batholith primarily consists of granitic crystalline rocks. No detailed geologic mapping has been done in Lee Valley, but other, nearby parts of the batholith, especially to the north and east (Todd, 1978, 1980), have been mapped. A generalized geologic map (fig. 3) was compiled for this study using existing information and field observations. In upland areas, the bedrock is exposed as cliff faces and large boulders (fig. 2*A*, *C*), and mapping was accomplished by limited field observations and examination of aerial photographs. On the valley floor, much of the bedrock is obscured by a thin layer of soil and weathered-rock material commonly known as regolith, and mapping was based on sparse outcrops and near-surface lithologies described in drillers' reports. Thus, the soil cover has not been mapped, but the inferred composition of the regolith has.

Three rock types were mapped (fig. 3). The light-colored, coarse-textured crystalline rock is mapped as granodiorite; the dark-colored, coarse-textured crystalline rock as coarse-grained gabbro; and the dark-colored, fine-textured crystalline rock as fine-grained gabbro. Granodiorite predominates in most of the upland part of the Lee Valley drainage basin, as well as in the western and northern parts of the valley floor. Coarse-grained gabbro is exposed at the edges of the basin: on a roadcut on Skyline Drive, and along Jamul Creek about 0.25 mi south of the basin boundary. Fine-grained gabbro mainly is in the eastern part of the valley floor. The fine-grained gabbro occurs in smaller bodies and is less resistant to weathering than the granodiorite or coarse-grained gabbro (V.R. Todd, U.S. Geological Survey, oral commun., 1988) and therefore is less common in upland areas.

Structure

The structural relations between the various rocks in Lee Valley are not well known. Drillers' records and a television log of a borehole (17S/2E-6H9) show that, at depth, lithologies commonly alternate. On exposures (surface and subsurface) where contacts can be seen, granodiorite appears to have intruded fine-grained gabbro. However, the intrusive relation is complex, suggesting contemporaneous emplacement of the two rock types. Large-scale contacts between granodiorite and fine-grained gabbro occur along the eastern and northern sides of the valley (fig. 3). The relative straightness of the contacts and the presence of possible breccia and offset-intersecting fractures indicate that these may be fault contacts.

Fractures

In the analysis of ground-water resources in bedrock terranes, fractures are by far the most important structural feature. They are almost ubiquitous in the Earth's crust and are found at great depths. Open fractures, where water saturated and hydraulically connected, serve to store and transmit water in the underground.

A fracture is defined as any parting across which coherence has been lost. The opposite faces of a fracture may be in partial or complete contact, may gape open, or may represent a parting that has healed. Fractures that have shear-type offset of opposing faces are classified as faults, and fractures with no offset are classified as joints (Trainer, 1987). Because this distinction can be difficult to make, the more general term "fracture" primarily is used in this report.

In Lee Valley, the general nature of fracture pattern can be seen in the area of high relief and altitude on the east and northeast sides of the drainage basin. Viewed from a distance, the fractures appear to fall into three nearly orthogonal sets. Two of the sets are nearly vertical, and the third is nearly horizontal. Most of the fractures are less than 1,000-ft in length. Detailed mapping, however, was not possible because this area is inaccessible by foot. Therefore, analysis of fracture orientations was accomplished by mapping of borehole walls through the use of acoustic-televiewer logs.

The acoustic televiewer (Zemanek, 1969) is a geophysical well-logging tool used to determine the location and orientation of planar features such as

EXPLANATION

INFORMAL LITHOLOGIC UNITS—Soil cover is ignored. Mapping of upland areas is based on field observations of outcrops and on aerial photographs. Mapping of the valley floor is based on sparse outcrops of weathered bedrock and on near-surface lithologies described in drillers' reports

g — Granodiorite

b — Gabbro, fine grained

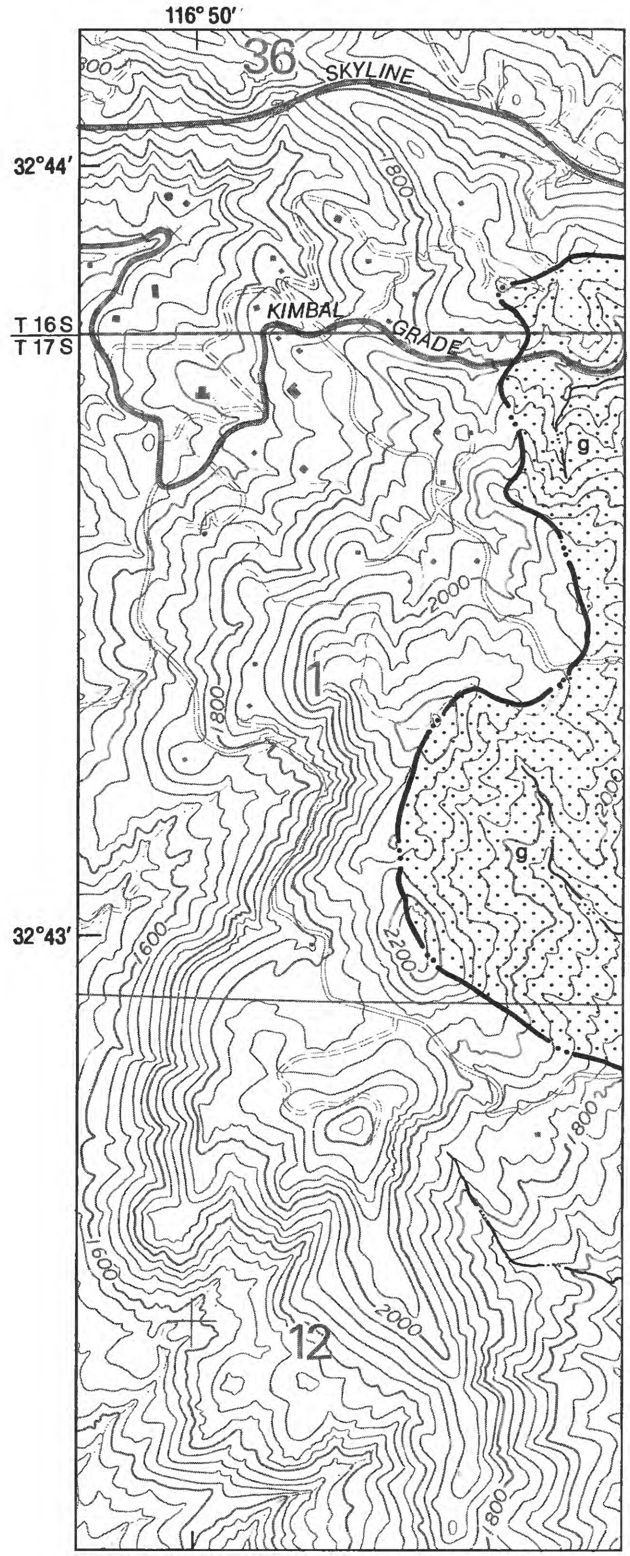
bc — Gabbro, coarse grained

— —?— CONTACT—Dashed where approximately located, queried where probable

SELECTED FAULTS—Apparent geologic contacts are observed at a number of locations along these faults

SURFACE-WATER BASIN BOUNDARY

52● WELL WITH LITHOLOGIC INFORMATION FROM DRILLER'S REPORT—Number, where present, indicates thickness of regolith, in feet



Base from U.S. Geological Survey, Dulzura, 1972

Figure 3. Generalized geology and thickness of regolith.

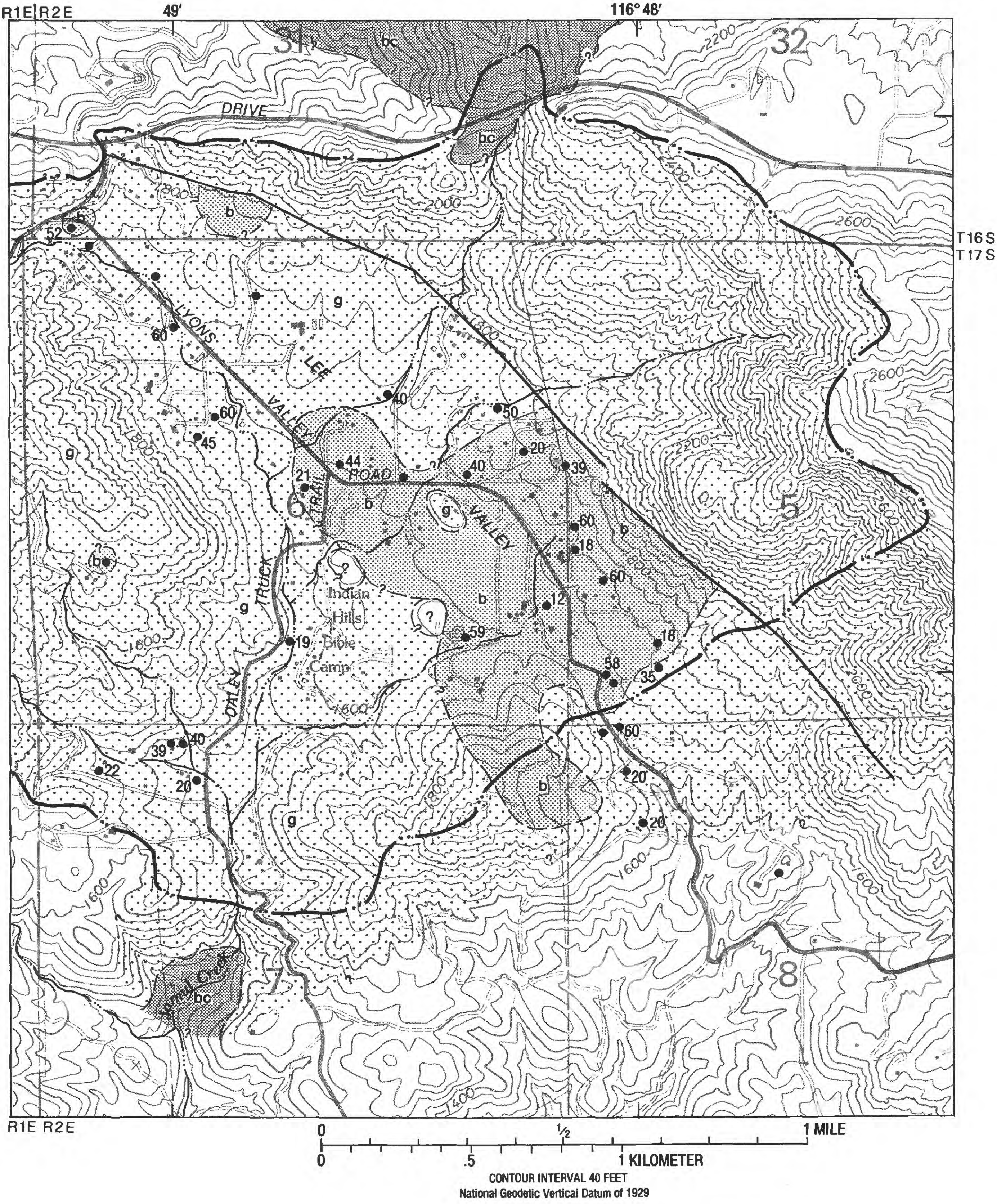


Figure 3. Continued.

fractures that intersect a well. The televiewer produces a two-dimensional image of the borehole wall, on which planar features that are parallel or perpendicular to the borehole appear, respectively, as vertical or horizontal straight lines; features intersecting at other angles appear as sinusoids. The appearance of a fracture trace in the acoustic-televiewer log and the method used to determine the fracture strike and dip are shown in figure 4. Because the televiewer signal can be transmitted only through a liquid medium, the log extends from the water level to the bottom of the well.

Fracture orientations were determined from acoustic-televiewer logs of 14 wells in Lee Valley. All the wells are on the valley floor. A total of 791 fractures were mapped. A lower-hemisphere, Schmidt stereonet plot of poles to fractures for the entire data set is shown in figure 5. (For a discussion of stereographic projection, see Ramsey, 1967.) One can see that there is a large scatter in fracture orientations. The orientations for the two highest concentrations (each representing about 3 percent of total fractures per 1 percent of the area of the hemisphere) are strike N. 35° W., dip 50° NE.; and strike N. 18° E., dip 50° W. A slightly weaker concentration occurs at N. 15° W., dip 65° W. These orientations seem to be inconsistent with those of the near-horizontal and near-vertical fractures seen on rock exposures on the east and northeast sides of the drainage basin. The inconsistency, in part, may be due to sampling bias because fractures mapped in vertical boreholes are underrepresentative of high-angle fractures. The borehole data do not show any significant concentration of near-horizontal fractures, even though sampling bias favors low-angle fractures. This observation suggests that the near-horizontal fractures seen on the hillsides may be surficial features not representative of the subsurface beneath the valley floor.

In figure 6, the predominant fracture orientations mapped in the wells are shown by strike and dip symbols. Secondary concentrations are represented by symbols that have two dip lines. In general, the predominant fracture orientations for the individual wells are somewhat similar to those of the combined data set. However, dispersion of orientations, rather than concentration, probably is more characteristic of fractures in Lee Valley.

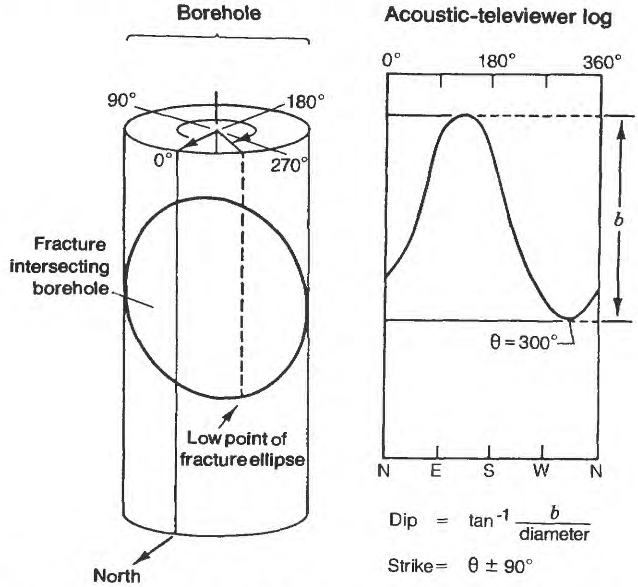


Figure 4. Example of fracture-orientation measurements made from an acoustic-televiewer log (from Paillet and others, 1987, fig. 3).

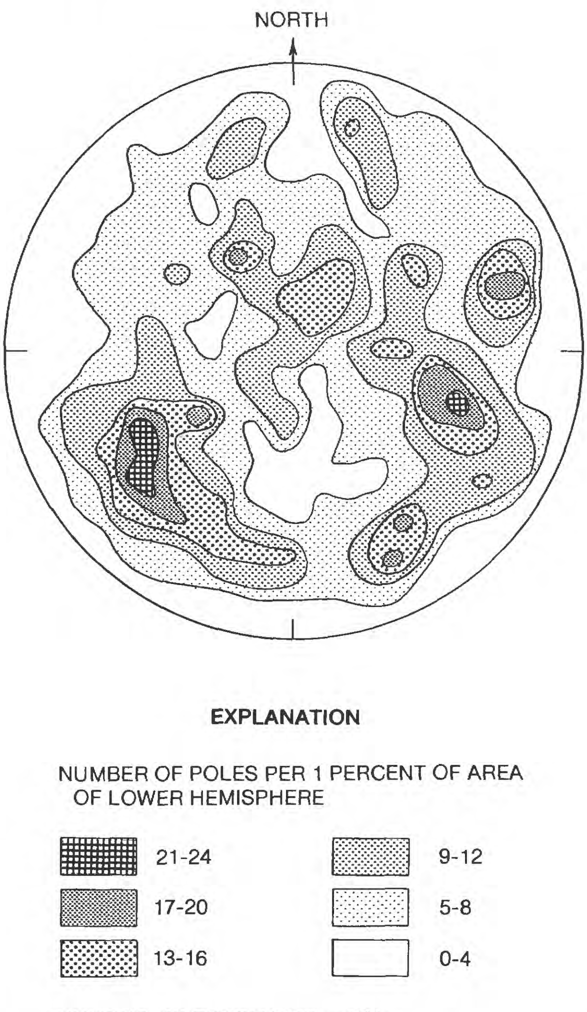


Figure 5. Contoured lower-hemisphere plot of poles to fracture planes determined from acoustic-televiewer logs of 14 wells. Compiled and prepared by Steven Pierce, San Diego State University (written commun., 1989).

Lineaments

According to Allum (1966, p. 31), "A lineament is any line on an aerial photograph that is structurally controlled; it includes, for photogeological purposes, any alignment of separate photographic images such as streambeds, trees, or bushes that are so controlled. The word thus has very wide applications; it can be used to refer to lines representing beds, lithological horizons, mineral banding, veins, faults, joints, unconformities, and rock boundaries." For Lee Valley, lineaments are of interest because they may represent zones of higher fracture intensity, which may result in increased permeability in the subsurface.

Lineaments in and near Lee Valley were mapped from 1:12,000 color stereo aerial photographs (Pierce, 1990). A high-altitude infrared photograph (scale 1:120,000) also was consulted as a check on the large-scale features. Only those fracture traces expressed continuously for at least 1,000 ft were mapped. The photogeologic features used for recognizing lineaments were (1) vegetation lines, (2) visible fractures or fracture zones in exposed outcrops, and (3) topographic features such as linear ridges, notched ridges, or abruptly deflected streamchannel segments. Soil tonal alignments were not useful because of the lack of visible undisturbed soil.

Lineaments mapped from aerial photographs are shown in figure 6. The lineaments are differentiated according to their degree of expression as strong, strong to moderate, and moderate to weak. Probable extensions of lineaments or probable connections between lineament segments also are shown. In general, the lineaments trend northwest and northeast. The northwest-trending lineaments are subparallel to the trend of Lee Valley; both sets may be related to a major structural trend of the region. Most of the lineaments probably are associated with near-vertical planar features such as fractures and faults. However, because of the vertical exaggeration of the stereo photographs used to map them, any feature with a dip of greater than about 50° generally appears to be vertical. In some places, a planar feature with a dip less than 90° is indicated by a curving surface trace as the feature crops out over topographic highs and lows.

Many of the longer, more strongly expressed lineaments that extend beyond the study-area boundaries probably are regional structural features. At some locations, these lineaments appear to be zones of increased density of parallel, near-vertical fractures. At a few other locations, they appear to be contacts between granodiorite and gabbro. Both of these characteristics, along with the presence of cemented angular conglomerate at two places, suggest that these lineaments may be fault zones.

Although lineaments may be local zones of higher fracture intensity, it is difficult to compare them with fractures mapped in boreholes in Lee Valley. In addition to the bias caused by mapping in a vertical borehole, the scale of mapping also creates a problem. Fractures in boreholes are mapped over a depth of several hundred feet, but lineaments are mapped over distances of thousands of feet. As noted before, the lineaments generally are surface expressions of near-vertical fracture zones or faults, but the dominant fractures encountered in boreholes dip approximately 50°. On the other hand, the strikes of the dominant fractures (northwest and northeast) are similar to the dominant trends of the lineaments. Thus, although the subsurface fractures may be related to the lineaments, the nature of the relation is not clear.

Fractured-Rock Aquifer

The aquifer in Lee Valley can be divided, primarily on the basis of degree of weathering, into three zones. Near the surface, the rock has been altered substantially into a layer of loose, fragmental, weathered-rock material commonly known as regolith. Below the regolith is a layer of fractured and partially weathered rock that is referred to as the transition zone in this study. Below the transition zone is fresh, unweathered rock, which may nevertheless contain fractures. Although the vertical extent of each layer cannot be defined precisely, the distinctions are useful for geohydrologic characterization. Similar divisions have been used in other studies of water resources in bedrock terranes (for example, Houston and Lewis, 1988). In effect, the aquifer is composed of three layers: the regolith, the transition zone, and the unweathered bedrock. Together, these three layers are referred to as the fractured-rock aquifer.

The regolith in Lee Valley is derived primarily from in-place weathering of bedrock, and to a lesser extent from deposition of alluvium by debris flow or streams. Although fractures originally in the parent rock may persist in the regolith, permeability and porosity are provided mainly by intergranular pore space. Clay minerals are present as a result of chemical weathering of the parent rock. Therefore, permeability of the regolith, in comparison with other unconsolidated

EXPLANATION

LINEAMENT OR FRACTURE TRACES—Mapped according to degree of expression: strong (solid lines); strong to moderate (long dashed lines); and moderate to weak (short dashed lines). Dotted lines denote probable extensions of lineaments or probable connections between lineament segments

75 STRIKE AND DIP OF PREDOMINANT FRACTURE ORIENTATION FROM ACOUSTIC-TELEVIEWER LOGS—Number is dip, in degrees. Direction of secondary-fracture orientation indicated with two dip lines

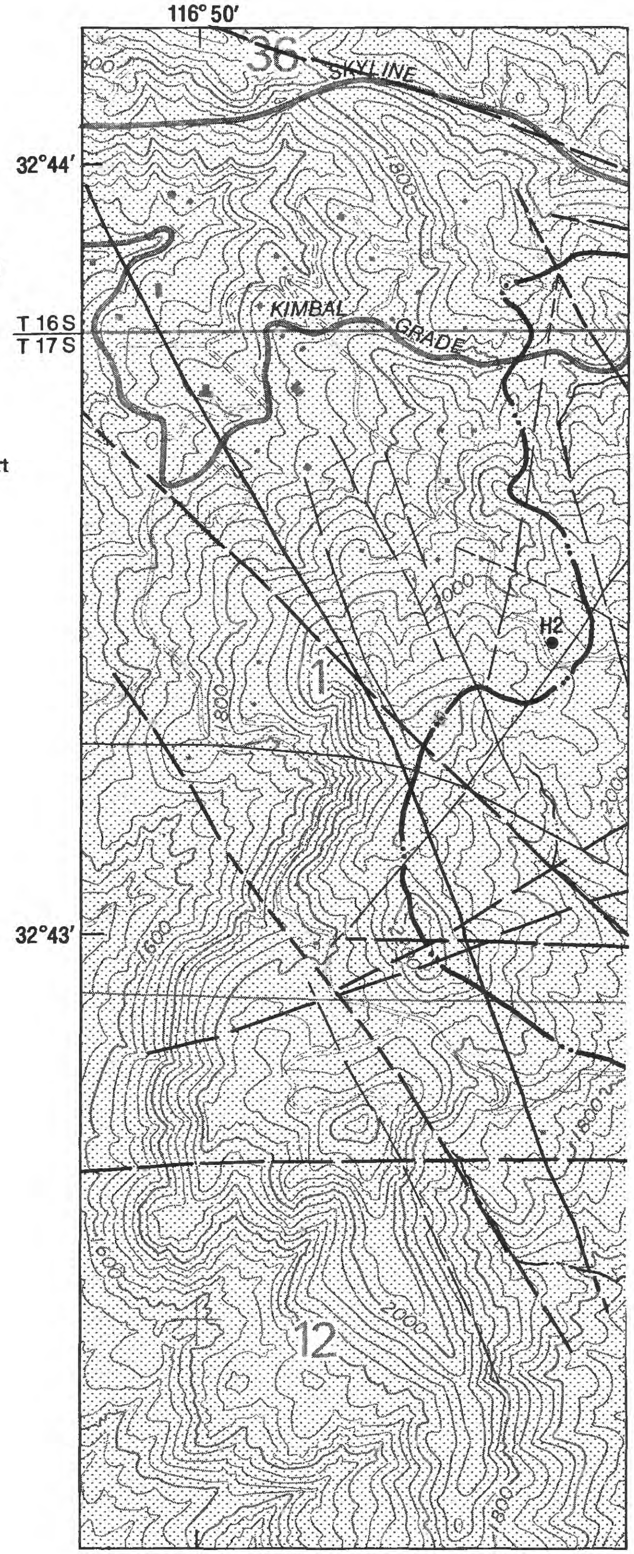
SURFACE-WATER BASIN BOUNDARY

WELL—With reported yield, in gallons per minute, from driller's report

○ M1 0-4

◒ D1 5-49

● C8 Equal to or greater than 50



Base from U.S. Geological Survey, Dulzura, 1972

Figure 6. Lineaments, predominant subsurface-fracture orientations, and reported well yields. Lineament locations and subsurface-fracture orientations mapped by Steven Pierce, San Diego State University (written commun., 1989).

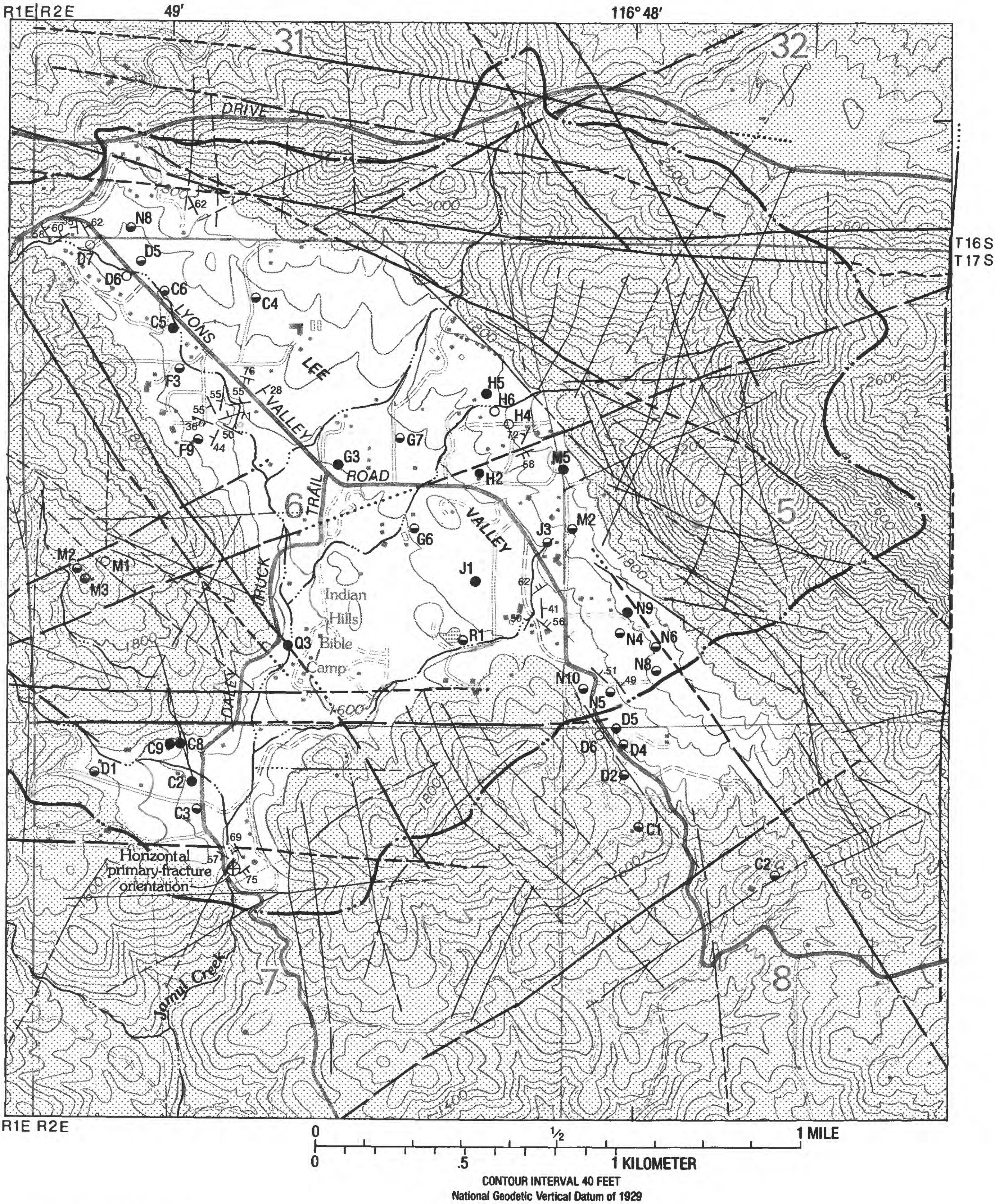


Figure 6. Continued.

materials such as alluvium, is relatively low. Thickness of regolith in Lee Valley (fig. 3) was estimated from lithologic descriptions on drillers' logs. In addition, wells generally are cased through the regolith and left open below. Thus, in areas of regolith, depth of the bottom of well casing can be taken as an approximate estimate of regolith thickness. As shown in figure 3, the regolith in the valley floor ranges from 12 to 60 ft in thickness. In upland areas, where bedrock crops out at land surface, the regolith is entirely absent. In the central part of the valley floor, the water table generally is 10 to 35 ft below land surface, and saturated regolith probably occurs as a laterally continuous layer. Along the edges of the valley floor, where depth to water table may be as much as 70 ft, saturated regolith may be limited to isolated patches separated by regions where the water table lies below the regolith.

Permeability and porosity in the transition zone and in the unweathered bedrock are entirely due to fractures. The boundary between the transition zone and the bedrock cannot be defined precisely. In the present study, this boundary is inferred from caliper logs of borehole diameter. In the transition zone, the borehole wall can be irregular, reflecting partial weathering of the rock. In the unweathered bedrock, the borehole wall is much smoother, with occasional irregularities where the well intersects fractures. An example of borehole geophysical logs is shown in figure 7. In general, the transition zone in Lee Valley extends to depths of less than 100 ft.

Well Yields

As used in this report, "well yield" is an informal term for characterizing the rate at which water can be produced from a well. Well yield usually is determined either during well drilling, by noting the flow of water being channeled away from the well, or shortly after drilling, by pumping water from the completed well. In either case, the amount of drawdown in the well usually is not measured, nor is the duration of water production. This lack of information can lead to problems in interpretation. For example, over a short duration, a low-production well can be pumped at a high rate; however, most of the water is derived from storage in the well bore. Thus, the well yield determined from such a short-duration test will be an overestimation. Continued pumping at the high rate eventually will cause the water level to decline below the pump intake. Therefore, well yield may not necessarily represent the sustained rate at which water can be pumped from a well. On the other hand, a comparison of well yields in different parts of an aquifer may provide insight on the aquifer heterogeneity.

Well yields for 43 wells in Lee Valley were compiled from drillers' records (table 1). As noted previously, reported values should not be viewed individually but rather are meant for comparison with each other. Perhaps the most striking feature in table 1 is the extreme variation in reported well yields, which range from 1 to 300 gal/min. A histogram (fig. 8) shows the distribution of yields. The distribution is skewed strongly to the left, indicating that most of the yields are low. Although the average yield is 40 gal/min, the median yield is only 15 gal/min. In other words, one-half the reported yields are less than or equal to 15 gal/min. This skewness in yield distribution, which is common in igneous-rock terranes (Davis and DeWiest, 1966, p. 232–328), reflects a high degree of heterogeneity in the subsurface.

Areal distribution of the reported well yields is shown in figure 6. For the purpose of illustration, yields are divided into three categories: 0 to 4 gal/min, 5 to 49 gal/min, and equal to or greater than 50 gal/min. Overall, a clear pattern of well yields is not apparent. In fact, neighboring wells may exhibit large differences in yield. For example, wells 17S/2E-6H6 and 17S/2E-6H5 are only about 150 ft apart, but the reported yield of the former is 2.5 gal/min, whereas the reported yield of the latter is 50 gal/min. These differences are further indications of the highly heterogeneous nature of the fractured-rock aquifer.

Studies in some bedrock terranes suggest that well yields may be related to the presence or absence of lineaments. In general, well yield is controlled by the permeability of the aquifer: A well with a high yield reflects high permeability near the well bore. Because a lineament may be the surface expression of an underlying zone of higher fracture intensity (and therefore higher permeability), a well drilled on a lineament may be more productive than a well drilled away from a lineament. This relation was demonstrated for carbonate rocks by Siddiqui and Parizek (1971), who surveyed well yields in central Pennsylvania. Although studies in crystalline rocks are lacking, it would not be unreasonable to expect a similar relation for crystalline rocks.

Well-yield data for Lee Valley were examined to explore the possibility of a relation between well yield and lineament proximity. The locations of lineaments and wells with reported yields are shown in figure 6. Many of the wells were drilled on the valley floor, where soil cover obscures any lineament that otherwise might have been seen on aerial photographs. However, if only those wells drilled at the edge of the valley floor and on the surrounding hillsides are considered, then a relation between well yield and lineament

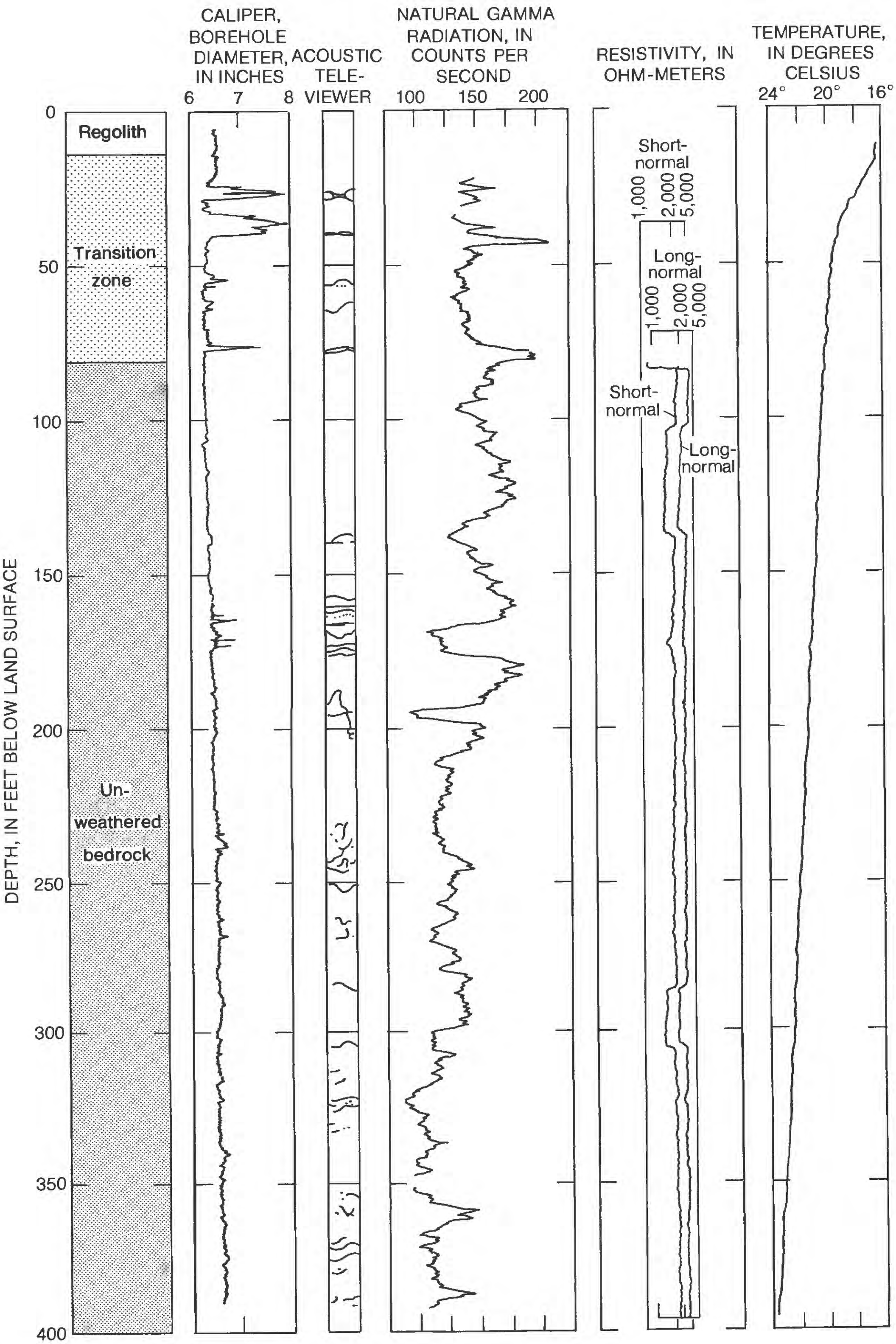


Figure 7. Example of borehole geophysical logs.

Table 1. Well-yield data from drillers' reports

[Well yield in gallons per minute. Depth of well and depth to water in feet below land surface. --, no data]

Well No.		Well yield	Depth of well	Depth to water	Well No.		Well yield	Depth of well	Depth to water
16S/2E-	31N8	15	400	--	17S/2E-	6H4	1.5	--	--
17S/1E-	1H2[1]	50	--	--		6H5	50	300	40
17S/2E-	5M2	37.5	144	30		6H6	2.5	210	45
	5M5	50	239	57		6J1	60	400	35
	5N4	25	240	60		6J3	30	610	--
	5N5	6	320	60		6M1[1]	1	--	--
	5N6	7	470	74		6M2[1]	25	540	120
	5N8	7	400	60		6M3[1]	15	--	--
	5N9	300	470	60		6Q3	120	210	30
	5N10	6.5	320	70		6R1	30	260	70
	6C4	15	260	--		7C2	65	260	40
	6C5	75	285	--		7C3	5.75	467	--
	6C6	12	308	--		7C8	150	245	15
	6D5	5	--	--		7C9	300	200	10
	6D6	2	590	--		7D1	15	140	25
	6D7	4	250	13		8C1	20	520	--
	6F3	6	335	--		8C2	15	525	150
	6F9	20	198	--		8D2	10	240	--
	6G3	50	190	25		8D4	5	605	55
	6G6	7	--	--		8D5	12	620	45
	6G7	15	230	40		8D6	2	350	83
	6H2	75	300	--					

[1]Includes information from oral communication with well owner.

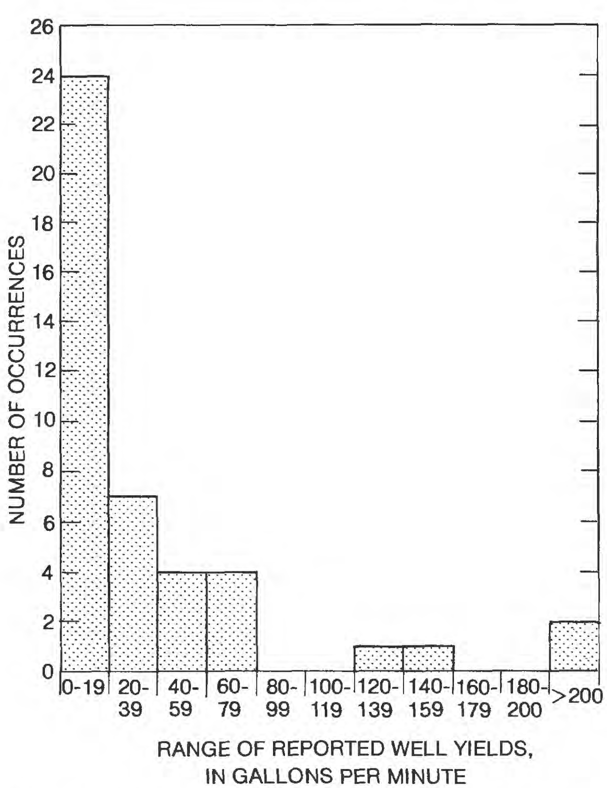


Figure 8. Frequency distribution of reported well yields.

proximity does seem to emerge. All valley-edge and hillside wells that have yields greater than or equal to 50 gal/min are near (within 300 ft of) lineaments. For example, wells 17S/2E-5N9, 17S/2E-6Q3, and 17S/2E-7C8 (figs. 1, 6), all with reported yields exceeding 100 gal/min, are near northwesterly trending lineaments. In addition, well 17S/2E-5M5, with a reported yield of 50 gal/min, is near a northeasterly trending lineament. Although the data are scant, they nevertheless suggest that most wells with high yields are near lineaments.

The converse is not supported by the data—it cannot be concluded that all wells drilled near lineaments have high yields. For example, wells 17S/2E-5N9 and 17S/2E-5N6 are about the same distance from a lineament. The former has a reported yield of 300 gal/min, but the latter has a reported yield of 7 gal/min. Thus, although lineament proximity may serve as a helpful guide for locating well sites, it is only one of many factors that control well yields. Drilling a well close to a lineament does not guarantee high yield.

The fact that high-yield wells are close to lineaments suggests two possibilities. First, a lineament may be flanked on both sides by zones of higher permeability. A well drilled on the flank may have as high a yield as one drilled on the lineament. Alternatively, a lineament may represent a steeply dipping, rather than vertical, fracture zone. Consequently, a well drilled near the lineament at land surface may intercept it at depth.

Ground-Water Movement

To determine the directions of horizontal ground-water movement, a water-level-contour map (fig. 9) was drawn on the basis of water levels measured in monitoring wells. Because some wells are completed in the regolith and others are completed in the transition zone or unweathered bedrock, the water levels shown in figure 9 are best viewed as composite hydraulic heads in the fractured-rock aquifer. The map shows that, for most parts of Lee Valley, ground water flows from the north, west, and the east sides of the valley toward Jamul Creek. Discharge of ground water along Jamul Creek occurs in the form of streamflow (generally limited to the southern reach of the creek, and to the late autumn, winter, and spring seasons) and evapotranspiration by oak trees that tap the water table. During periods of heavy precipitation, the rise in stream stage may result in local recharge to the regolith along the streambanks. However, the amount of recharge is minor in comparison with discharge during the rest of the year.

To examine the vertical movement of ground water in Lee Valley, inflatable packers were installed in three wells to measure hydraulic heads at different depths. Monitoring over a period of 1 year showed vertical hydraulic gradients (head differences of 2 to 11 ft) that provide driving forces for downward movement at the northern end of the valley (well 16S/2E-31N6) (see fig. 1) and for upward movement near Jamul Creek (well 17S/2E-6F14) and in the southeast part of the basin (well 17S/2E-5N7). These observations are consistent with the general concept of ground-water movement in Lee Valley; that is, downward movement is expected in recharge areas, and upward movement is expected in discharge areas.

Temporal Water-Level Fluctuations

Water levels in Lee Valley have been measured monthly since 1984 at most of the approximately 60 monitored wells. In addition, beginning in 1985, three continuous water-level recorders were in place, and a total of eight recorders have been operating since June 1987. Water-level data from this study are published by the USGS in a summary of water-resources data for California.

Fluctuations in water levels are directly related to rainfall that recharges the fractured-rock aquifer. Rainfall measurements for 1984–88 at three stations in Lee Valley are shown in figure 10. The records for stations 1 and 2 on the valley floor are virtually identical. Station 3 in the upland area shows about 10 percent greater rainfall in comparison with the valley-floor stations. In general, most of the rainfall occurs between October and May; November through January are the wettest months. In this study, a "rain year" is defined as the 12-month period from July 1 of a year to June 30 of the succeeding year. Note that rain years 1985–86 and 1987–88 were considerably wetter than rain years 1984–85 and 1986–87.

Because rainfall fluctuates both seasonally and annually, water levels can be expected to fluctuate accordingly. Water levels that were recorded during 1984–88 at three wells in Lee Valley are shown in figure 11. These wells were selected because they show water-level responses typical of wells in Lee Valley. The first well (17S/2E-6F6) is shallow (depth is 20 ft) and is in the west-central part of Lee Valley. Because the well penetrates only the regolith, the water level responds relatively quickly to recharge. In general, the water level rises during the November-to-April rainy season and declines during May to October. However, the amount of water-level rise varies from year to year. Higher rainfall during the 1985–86 and 1987–88 rain years is reflected by larger water-level rises during the rainy seasons. Conversely, lower rainfall during the 1984–85 and 1986–87 rain years resulted in smaller water-level rises.

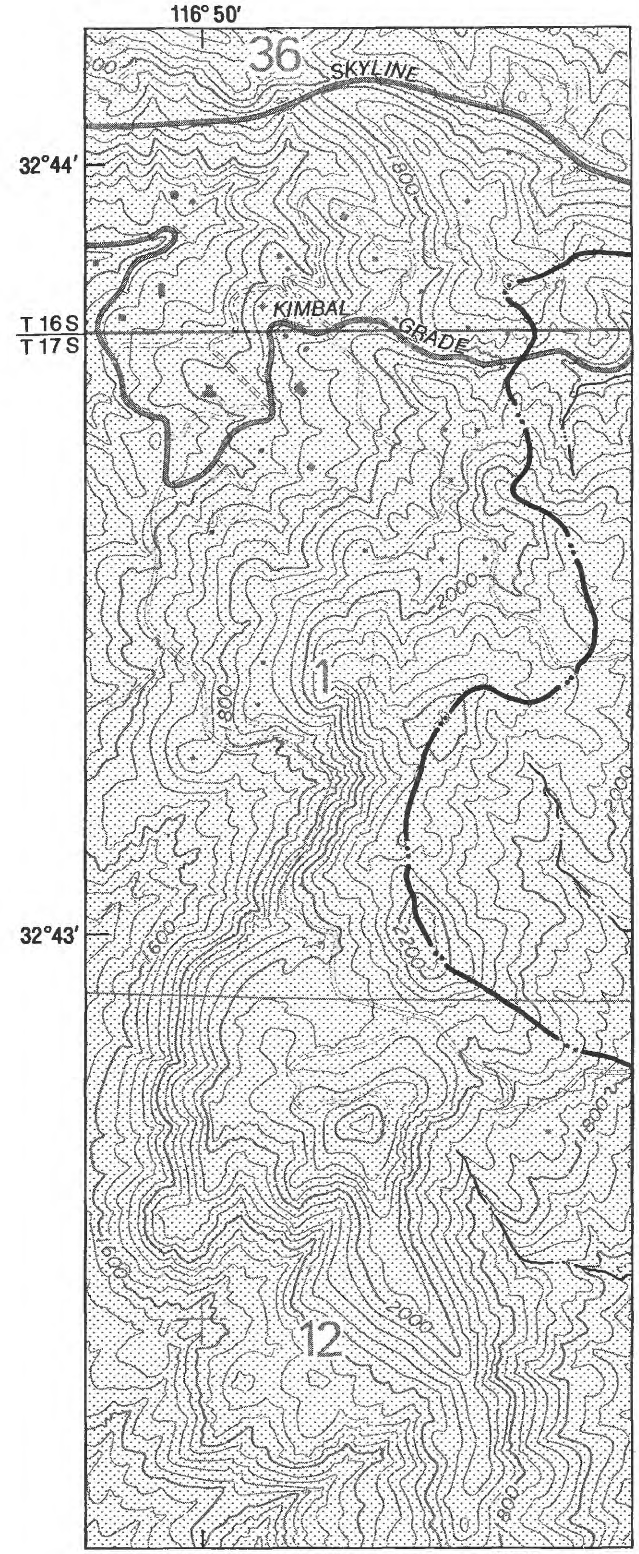
The second hydrograph in figure 11 is from a moderately deep well (203 ft) in the southeastern part of Lee Valley. The well (17S/2E-6R2) is open to both the transition zone and the unweathered bedrock. Hydraulic tests showed that both zones are productive; thus, the water level in this well should represent a composite hydraulic head of the two zones. The hydrograph mimics that of the well (17S/2E-6F6) described above, which is in the regolith. However, the rises and declines in water level are more gradual, and the magnitudes from troughs to peaks are smaller. (The apparent sudden rise in water level in June 1987 probably is the result of a measurement error.) The subdued features in the hydrograph indicate that the

EXPLANATION

——1,700——? WATER-TABLE CONTOUR–Shows altitude of water table. Dashed where approximate; queried where probable. Contour interval 20 feet. Datum is sea level

—— · · —— SURFACE-WATER BASIN BOUNDARY

● WELL



Base from U.S. Geological Survey, Dulzura, 1972

Figure 9. Altitude of the water table, December 31, 1985.

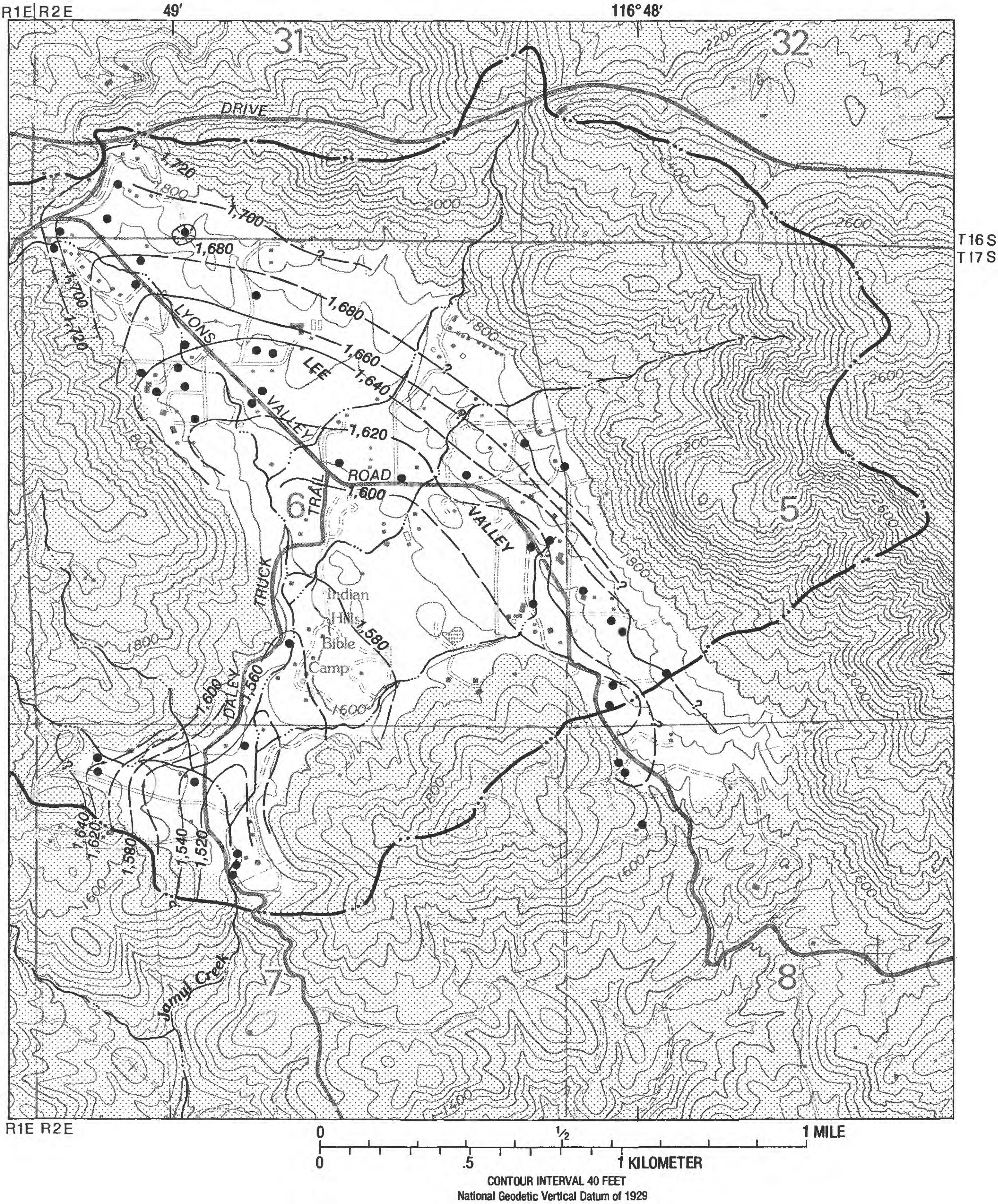


Figure 9. Continued.

deeper parts of the fractured-rock aquifer respond more slowly than does the regolith to ground-water recharge.

The hydrographs for several deep wells in Lee Valley show greatly muted response to recharge. The third hydrograph in figure 11 is such an example. Well 17S/

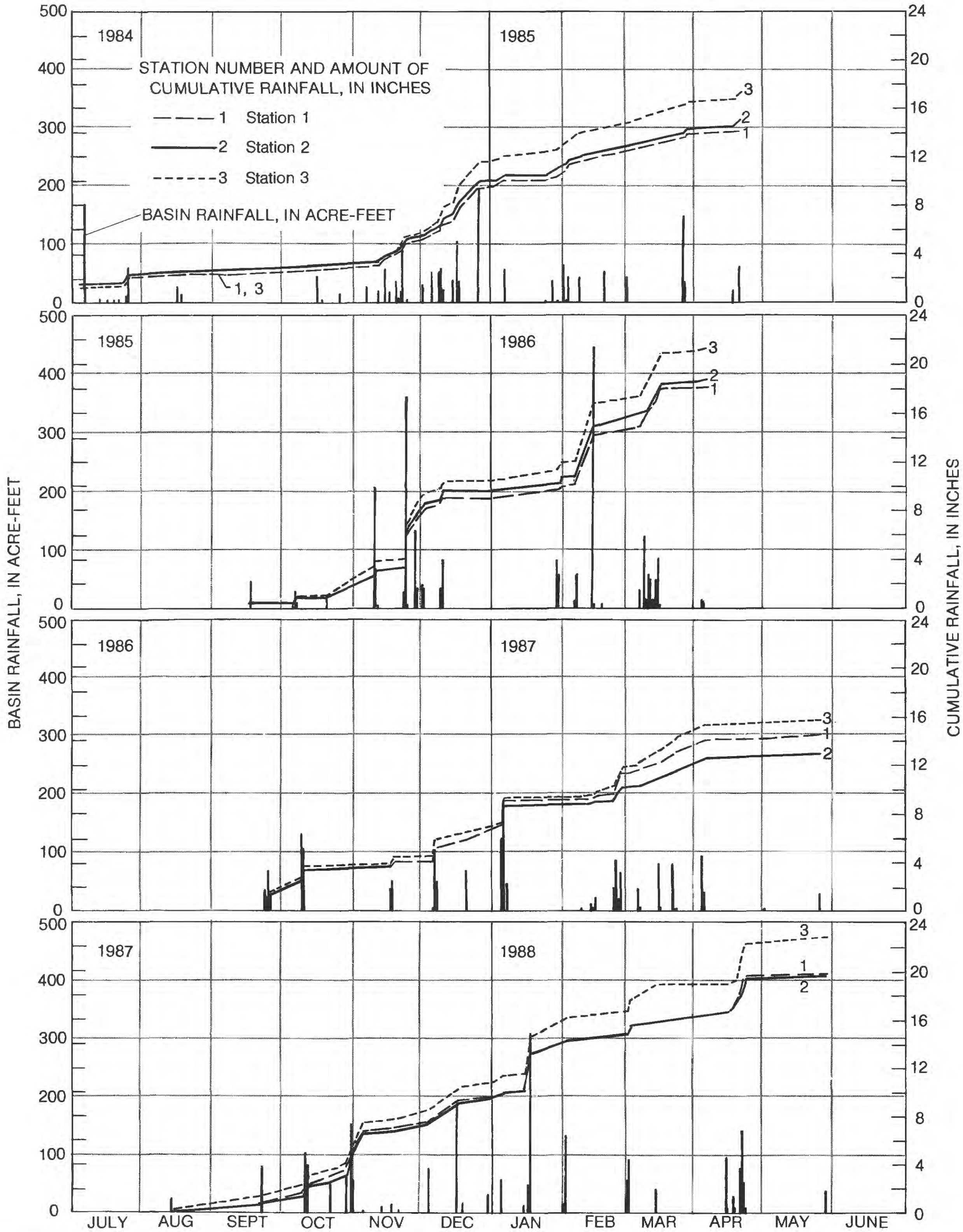


Figure 10. Rainfall in Lee Valley, July 1984–June 1988.

2E-5N8 is a 400-ft-deep well in the southeastern part of Lee Valley. Recharge during the relatively wet winters of 1985–86 and 1987–88 is reflected by gentle upward trends, peaking almost 4 months after the corresponding peak in the hydrograph of the regolith well. Response to recharge during the low-rainfall winters of 1984–85 and 1986–87 is almost imperceptible. The sluggish response indicates that the well probably is in a locally tight part of the subsurface and therefore is poorly connected to the transmissive fractures in the transition zone and unweathered bedrock. The water-level changes in such a well represent a delayed and damped version of the hydraulic-head changes in the aquifer.

A downward trend is evident for the 4½-year period (January 1984 to July 1988) shown in the three hydrographs in figure 11. This trend is most noticeable in the hydrograph for well 17S/2E-5N8, in which the seasonal fluctuations are more or less damped out. Analysis of ground-water recharge and discharge (described later in this report) suggests that this downward trend cannot be attributed entirely to ground-water pumpage. Rather, the trend indicates that the aquifer is extensively recharged only during periods of exceptionally high rainfall. For example, the relatively high water levels at the beginning of 1984 were due to heavy rainfall during the winter of 1982–83. For

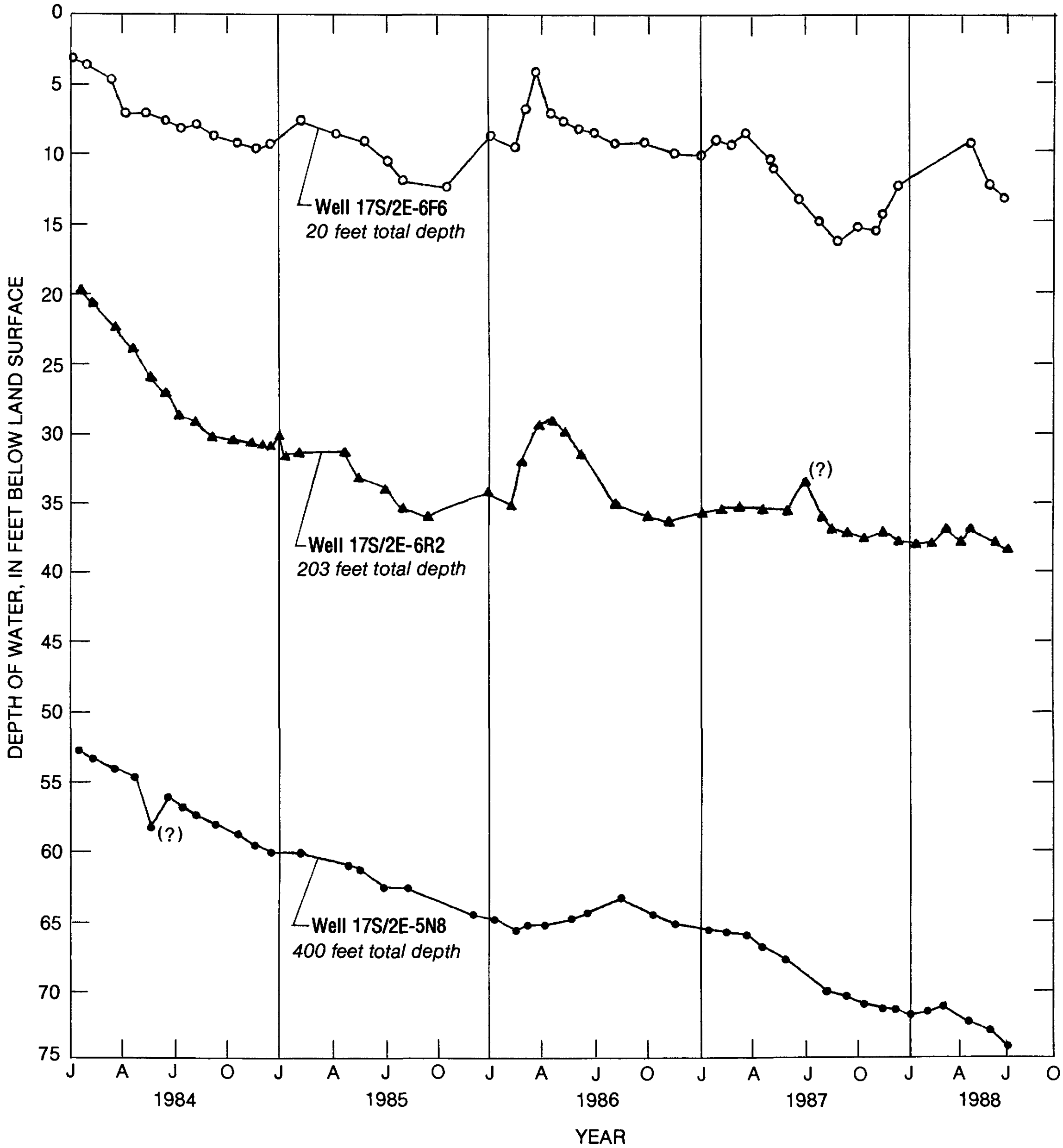


Figure 11. Water levels in three typical wells, January 1984–July 1988.

1984–88, rainfall was closer to average, and a downward trend in water levels is superimposed on the seasonal fluctuations. This decline is expected to continue until the aquifer is recharged again by heavy rainfall, whereupon the water level may rise by tens of feet in some locations.

On a shorter time scale (several days to several weeks), the water-level fluctuations in regolith wells reflect the role of Jamul Creek as a drain in the southern part of Lee Valley. Rainfall during 1987–88 and two hydrographs recorded in regolith wells are shown in figure 12. Well 17S/2E-6F11 is about 400 ft from Jamul Creek in the west-central part of Lee Valley. Well 17S/2E-7C4 is about 150 ft from Jamul Creek in the southern part of the valley. During January and February of 1988, water level in well 17S/2E6-F11 rose by 6 ft, while the water level in well 17S/2E-7C4 rose by less than 2 ft. In general, these short-term responses are typical for regolith wells in Lee Valley. Regolith wells in the northern part of the valley and farther from Jamul Creek show larger responses to winter rainfall; regolith wells in the southern part of the valley and closer to Jamul Creek show smaller responses. These responses suggest that Jamul Creek acts as a drain, receiving inflow of ground water from the saturated regolith in the southern part of Lee Valley. Near the creek, the water table is strongly controlled by the elevation of the creekbed. Therefore, fluctuations in water level are limited to a narrow range.

Water Quality

Ground-water samples were collected from zones isolated by inflatable packers in five test wells to define existing water quality and to serve as a possible aid in recognizing sources of recharge and vertical movement of water between aquifer zones. In test well 17S/2E-7C1, an additional sample was collected from the entire open interval. A sample also was collected from an observation well (17S/2E-5N7). The water samples were collected toward the end of aquifer tests, after the pump had been on for about 5 hours or more. Therefore, the water samples are assumed to be representative of water in the aquifer rather than water in the well. Two additional analyses were reported originally by the California Department of Water Resources (1967, app. D).

Results of laboratory analyses of the samples are shown in table 2. Water quality generally meets U.S. Environmental Protection Agency standards for drinking water. Exceptions are dissolved iron, dissolved manganese, and dissolved solids, which exceeded the recommended limits in most of the wells tested. The anomalously high value for dissolved iron in well 17S/2E-5N7 could have been caused either by naturally occurring reducing conditions, by the steel well casing in the upper part of the hole, or by incomplete development and purging of the well. The recommended limits for dissolved chloride and dissolved sulfate were exceeded in the lower zone of well 17S/2E-6R2. The concentration limits for all these constituents are among those that have been set primarily to provide acceptable esthetics and taste characteristics (Freeze and Cherry, 1979, p. 386; U.S. Environmental Protection Agency, 1977, 1979).

Differences in concentrations of individual constituents in samples collected from different zones in the same well generally were insignificant. The greatest differences were in the concentrations of dissolved calcium, sodium, chloride, sulfate, total nitrate-nitrogen, boron, and solids, and in hardness and specific conductance (table 2). For most of these constituents, water from the deeper part of the well had higher ionic concentrations than water from the transition zone, possibly indicating that water entering the well from deeper fractures either traveled a different flow path than water in the transition zone or had not mixed thoroughly with the shallow part of the flow system.

Some of the samples also were analyzed for oxygen and hydrogen isotopes (table 2). All the hydrogen-isotope-ratio values are virtually the same, as are the oxygen-isotope-ratio values, indicating that water sampled from different wells and different aquifer zones has had a similar history or source area.

The results of analyses for tritium concentration also are shown in table 2. Tritium is produced in large quantities by atmospheric testing of nuclear weapons. Once produced in the atmosphere, the tritium atom is incorporated in the water molecule and is useful as a hydrologic tracer. Tritium also is produced naturally in small amounts by the interaction of cosmic rays and atmospheric molecules. Natural pre-bomb (pre-1953) concentrations of tritium are about 6–13 pCi/L (picocuries per liter) or 2–4 TU (tritium units) (Freeze and Cherry, 1979, p. 136). In the 1950's and early 1960's, tritium concentrations in precipitation increased by 2 to 3 orders of magnitude, with the greatest peak in the early 1960's. The concentrations determined for Lee Valley samples ranged from 12.5 to 22.1 pCi/L (3.9 to 6.9 TU). These values are slightly higher than pre-1953 concentrations but not as high as would be expected if the sampled ground water had been recharged in the early 1960's. Recharge of the sampled water, therefore, probably took place somewhat less than 25 years ago. For well 17S/3E-6R2, the difference between the high and low values of tritium (table 2) and the difference between values from the two aquifer zones probably are not great enough to allow an interpretation of distinct recharge events separated by time or by flow path.

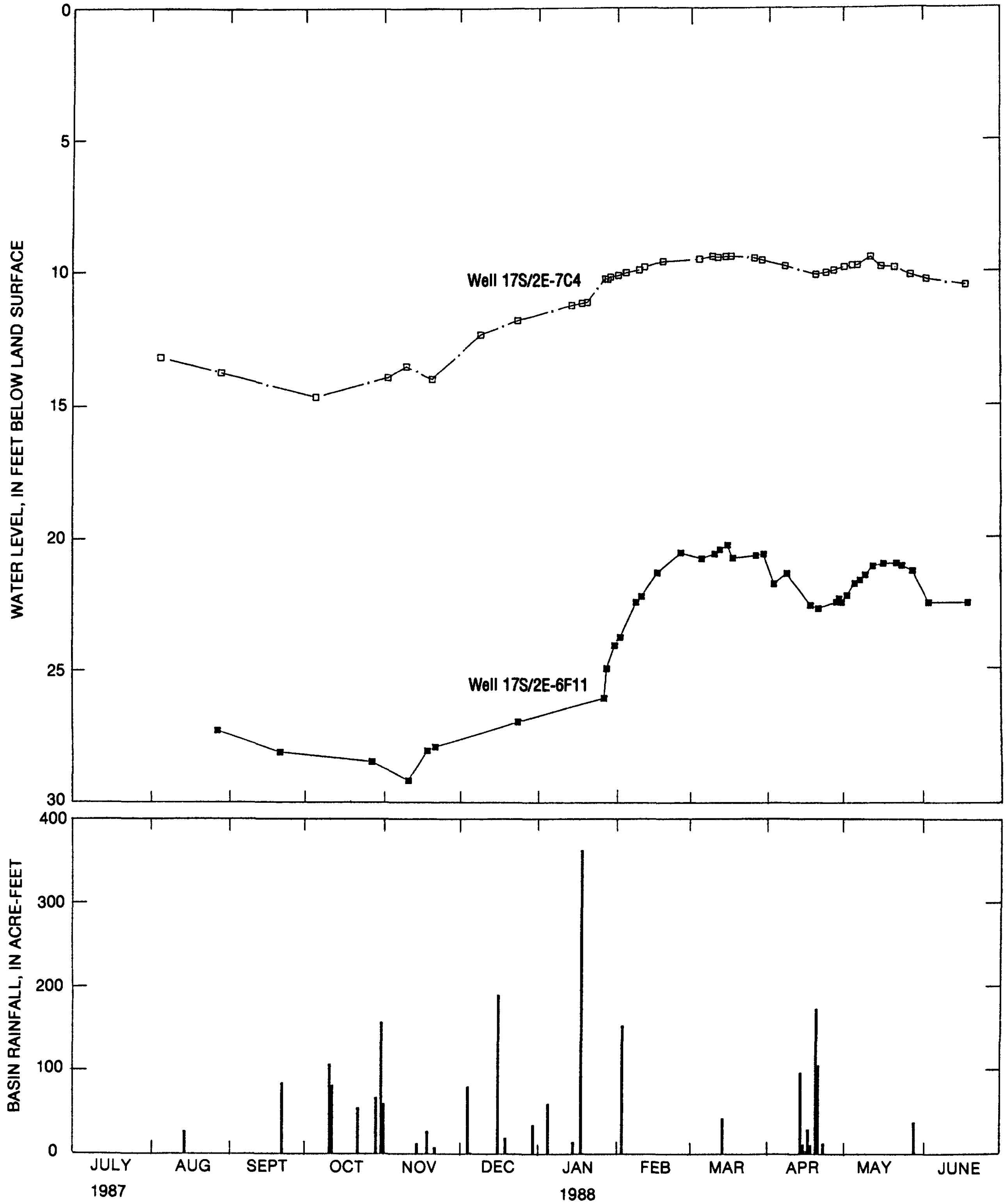


Figure 12. Short-term water-level response in wells 17S/2E-6F11 and 17S/2E-7C4 to rainfall, July 1987–June 1988.

Table 2. Chemical analyses of water from wells, Lee Valley

[Depth of well and interval sampled in feet below land surface. Specific conductance, pH, alkalinity: F, field; L, laboratory. µS/cm, microsiemen per centimeter at 25°C; °C, degree Celsius; mg/L, milligram per liter; µg/L, microgram per liter; pCi/L, picocurie per liter. <, actual value is less than value shown. --, no data]

Well No.		Depth of well	Interval sampled	Date	Specific conductance (µS/cm)	pH (units)	Temperature (°C)	Hardness (mg/L)	Noncarbonate hardness (mg/L)	Calcium, dissolved (mg/L)
16S/2E-	31N4	457	above 100	1-20-88	F934	F6.8	--	250	--	57
17S/2E-	5N5	320	above 118	12-13-87	L581	L7.3	--	170	--	42
	5N7	328	entire well	12-18-87	--	--	22.0	140	0	40
	6F9	198	above 83	1-10-88	F1,010	F7.2	21.0	290	--	80
			below 86	1-13-88	F1,010	F7.1	20.5	300	110	84
			below 153	1-15-88	L1,020	F7.3	--	300	110	84
	6J3[1]	--	--	3-08-63	L620	L7.4	--	185	--	46
	6Q1[1]	--	--	3-08-63	L600	L7.3	--	178	--	40
	6R2	203	above 100	1-22-88	F1,280	F6.8	22.5	500	260	120
			below 100	1-24-88	F1,890	F7.0	22.5	700	440	170
	7C1	451	above 44	12-08-87	L1,230	L7.8	--	340	--	74
			entire well	4-26-87	--	--	--	240	56	57

Well No.		Interval sampled	Date	Magnesium, dissolved (mg/L)	Sodium, dissolved (mg/L)	Sodium (percent)	Sodium adsorption ratio	Potassium, dissolved (mg/L)	Alkalinity (mg/L as $CaCO_3$)	Sulfate, dissolved (mg/L)
16S/2E-	31N4	above 100	1-20-88	26	86	43	2	4.6	F199	51
17S/2E-	5N5	above 118	12-13-87	17	55	41	2	2.5	L144	58
	5N7	entire well	12-18-87	10	71	--	3	3.0	L164	--
	6F9	above 83	1-10-88	23	82	38	2	4.0	F194	42
		below 86	1-13-88	23	83	--	2	4.4	F193	41
		below 153	1-15-88	23	84	--	2	6.1	F188	--
	6J3[1]	--	3-08-63	17	64	--	--	1.0	--	35
	6Q1[1]	--	3-08-63	19	69	--	--	1.0	--	46
	6R2	above 100	1-22-88	49	83	27	2	2.3	F243	180
		below 100	1-24-88	68	110	--	2	2.2	F250	260
	7C1	above 44	12-08-87	37	120	44	3	1.4	L208	97
		entire well	4-26-87	24	95	46	3	2.4	L185	53

Footnote at end of table.

DETERMINATION OF HYDRAULIC PROPERTIES USING AQUIFER TESTS

Although aquifer-testing methods routinely are used to determine aquifer properties, their application to fractured rocks is debatable. Many of the difficulties arise from the extremely heterogeneous nature of the physical features and hydraulic properties of fractured rocks. Fracture characteristics, such as aperture, spatial extent, orientation, spacing, and interconnectivity, can vary considerably from one part of the aquifer to another. In addition, the effects of weathering exert a significant influence on aquifer heterogeneity at shallow depths. Therefore, the hydraulic properties of the regolith, transition zone, and unweathered bedrock of the fractured-rock aquifer in Lee Valley can vary from place to place. Also, faults and fracture zones, which are common in bedrock, can act either as zones

Table 2. Chemical analyses of water from wells, Lee Valley—Continued

Well No.	Interval sampled	Date	Fluoride, dissolved (mg/L)	Chloride, dissolved (mg/L)	Silica, dissolved (mg/L)	Residue on ignition at 180°C (mg/L)	Solids, dissolved, sum (mg/L)	Nitrate, dissolved, total (mg/L as N)	Nitrogen, ammonia, dissolved (mg/L as N)
16S/2E- 31N4	above 100	1-20-88	0.4	160	48	570	560	1.10	0.016
17S/2E- 5N5	above 118	12-13-87	.4	65	48	383	370	2.00	.002
5N7	entire well	12-18-87	.4	59	37	--	--	2.30	.260
6F9	above 83	1-10-88	.6	180	36	--	570	.15	.042
	below 86	1-13-88	.6	180	36	--	--	<.1	.043
	below 153	1-15-88	.6	180	37	587	--	.100	.009
6J3[1]	--	3-08-63	1.0	94	34	416	388	--	--
6Q1[1]	--	3-08-63	.6	74	44	412	410	--	--
6R2	above 100	1-22-88	.3	170	46	--	800	2.60	.072
	below 100	1-24-88	.3	320	31	--	1,100	.100	.060
7C1	above 44	12-08-87	.6	210	55	744	720	.830	.017
	entire well	4-26-87	.6	160	53	--	557	--	--

Well No.	Interval sampled	Date	Boron, dissolved (µg/L)	Iron, dissolved (µg/L)	Lithium, dissolved (µg/L)	Manganese, dissolved (µg/L)	Deuterium/ protium ratio (per mil)	Oxygen-18/ oxygen-16 ratio (per mil)	Tritium in water molecule (pCi/L)
16S/2E- 31N4	above 100	1-20-88	100	7	72	420	-39.4	-6.35	22.1
17S/2E- 5N5	above 118	12-13-87	60	5	25	3	-39.0	-6.15	17.9
5N7	entire well	12-18-87	70	6,000	30	50	--	--	--
6F9	above 83	1-10-88	50	460	70	270	-39.5	-6.20	14.1
	below 86	1-13-88	50	--	63	--	--	--	--
	below 153	1-15-88	60	320	72	270	-39.5	-6.25	12.5
6J3[1]	--	3-08-63	0	--	--	--	--	--	--
6Q1[1]	--	3-08-63	50	--	--	--	--	--	--
6R2	above 100	1-22-88	70	--	39	--	-39.0	-6.30	17.6
	below 100	1-24-88	40	130	50	230	-39.0	-6.25	13.1
7C1	above 44	12-08-87	90	670	36	510	--	--	--
	entire well	4-26-87	80	60	40	590	--	--	--

[1]Data from California Department of Water Resources (1967, app. D). Well 17S/2E-6J3 formerly was numbered 17S/2E-5M1.

of high permeability or as barriers to ground-water flow.

In designing the aquifer tests we idealized the aquifer to consist of three zones—regolith, transition zone, and unweathered bedrock. Each zone was treated as an equivalent porous medium with uniform hydraulic properties. This approach is based on the assumption that the ground-water flow is not controlled by a small number of fractures in each zone. Instead, the fractures were assumed to form a network of interconnected conduits, analogous to the system of connected pore space within a granular medium when the latter is magnified many times. The results of the aquifer tests provide "overall" hydraulic properties that characterize the aquifer on a horizontal length scale approximately equal to the distance between pumped and observation wells (30 to 100 ft in this study).

Special attention must be paid to the local variation of hydraulic conductivity in the immediate vicinity of the pumped and observation wells. If the hydraulic conductivity around a pumped well is lower than the overall hydraulic conductivity of the aquifer, a so-called skin effect exists. The skin effect increases drawdown as the water moves across the zone of lower permeability into the pumped well. Conversely, a zone of enhanced hydraulic conductivity surrounding a pumped well reduces drawdown. For the observation well, a surrounding zone of lower hydraulic conductivity creates a delayed response—that is, the change in water level in the observation well lags behind change in hydraulic head in the aquifer. Although these effects introduce additional complexities in the test analysis, they must be taken into account. Failure to do so can lead to errors in the analysis of the test results.

Studies have shown that the hydraulic conductivity of fractured rocks can be anisotropic (Hsieh and others, 1985; Maslia, 1987). The anisotropy commonly is attributed to the presence of preferred flow directions along subparallel sets of fractures in the rock. To fully characterize the anisotropy, it is necessary to determine the magnitudes of the hydraulic conductivities in three principal directions. In this study, we assumed that one principal direction is vertical and the other two horizontal. We assumed, also, that the hydraulic conductivity is isotropic in the horizontal plane. Although these assumptions may not always hold for fractured rocks, they are adopted here for the sake of simplicity. Because the horizontal and vertical components of water movement are of primary interest in this study, the above assumptions allow us to express Darcy's law in terms of two properties: the horizontal hydraulic conductivity, K_r,and the vertical hydraulic conductivity, K_z. Departure of actual conditions from this simplified characterization would alter somewhat the flow configuration, but the general conclusions of the study, in terms of horizontal and vertical water movement, should remain valid.

Design of Aquifer Tests

Aquifer tests were done at five sites in Lee Valley (fig. 1). Available sites necessarily were limited to those that had an unused, privately owned well 200 ft or more in depth. These wells generally are low yielding, and none of them are close to major lineaments mapped on aerial photographs. Consequently, the results of the aquifer tests are more characteristic of the lower permeability rocks in Lee Valley. The hydraulic characteristics of the lineaments were not investigated.

Construction of Wells and Piezometers

To supplement information available from the existing, privately owned wells, the U.S. Geological Survey constructed additional wells and piezometers for the aquifer tests and for monitoring water levels in Lee Valley. The wells were drilled using a 6-in. airhammer with foam/air circulation. The new wells were cased through the regolith with 6-in. steel casing and left open below (as had been done for the privately owned wells). Piezometers were completed with 2-ft screens at the bottom of 2-in.-diameter PVC (polyvinyl chloride) pipes. The screens were surrounded by sand packs, above which bentonite and concrete seals were placed. The construction of a typical piezometer is shown in figure 13.

Description of Equipment

In the pumped well, packers are placed above and below the pump so that only a part of the well is pumped (fig. 13). In some cases, only one packer is installed in the well, and the section above or below the packer is pumped. In the observation well, a string of packers can be installed in the well to divide it into several intervals. Each interval is hydraulically connected to a drop pipe so that the hydraulic head of the interval can be measured by water level in the pipe (fig. 13). In this way, hydraulic heads at different depths in the aquifer can be monitored in the same well. Water levels are measured by pressure transducers and are recorded in a data logger. The use of drop pipes also allows water-level measurements to be checked using steel tapes or electrical sounders. These occasional check measurements were done before, during, and after the tests.

Test Procedure

Prior to an aquifer test, the delayed response of the piezometers and observation wells must be characterized. This is done by analyzing the well response to an instantaneous rise of water level in the well. (The actual analysis is described in the section entitled "Effect of Delayed Response in Observation Well.") For piezometers, the rise in water level is effected by rapidly lowering a deadweight into the water. For observation wells, the water-level rise is caused by the expansion of the rubber element during packer inflation. Although this water-level rise may not be as rapid as that achieved by submersing a deadweight, it generally can be considered to be instantaneous when compared with the time needed for the water level to return to its static level.

The basic test setup was to pump from one water-transmitting zone (transition zone or bedrock) and to monitor drawdowns in all zones. Water levels were monitored, with one exception, for at least 12 hours prior to the start of the test to determine whether there was a long-term trend. (In the test analysis, drawdowns were corrected to account for any such trend.) The test itself generally lasted 6 to 10 hours. The pumping rate was adjusted so as to cause measurable drawdown in observation wells and piezometers but to not allow development of cascading water, which occurs when the water level in the pumped well declines below the depth at which water enters the well. Toward the end of the test, water samples were collected for chemical analysis. After the pump was turned off, recovery of water levels was monitored for at least 12 hours.

Method of Aquifer-Test Analysis

Generally, the analysis of aquifer tests consists of selecting an appropriate aquifer model and choosing model parameters (for example, aquifer properties such as hydraulic conductivity and specific storage) so that the model-simulated drawdowns match the measured drawdowns. The aquifer model may range from a simple analytical model involving a few parameters to a complex computer model involving many parameters. The choice of model generally is based on the background knowledge of the geohydrologic setting of the site, on the size of the data set available for analysis, and on the degree of complexity that is needed to adequately characterize the aquifer. For simple analytical models involving two or three parameters, graphical procedures based on type curves (for example, the Theis method) can be used to determine the aquifer properties. If the model involves substantially more parameters, then one must resort to either trial-and-error matching or computer methods based on least squares or other procedures.

In this study, analytical models were used in the analysis of the aquifer tests. The tests were done in a fractured-rock aquifer composed of zones (regolith, transition zone, and unweathered bedrock) that may have different hydraulic properties; therefore, the models need to be able to account for these zones. In addition, the models need to take into account the effects of wellbore storage in the pumped well, the possible presence of a zone of altered permeability around the pumped well, and the delayed response in

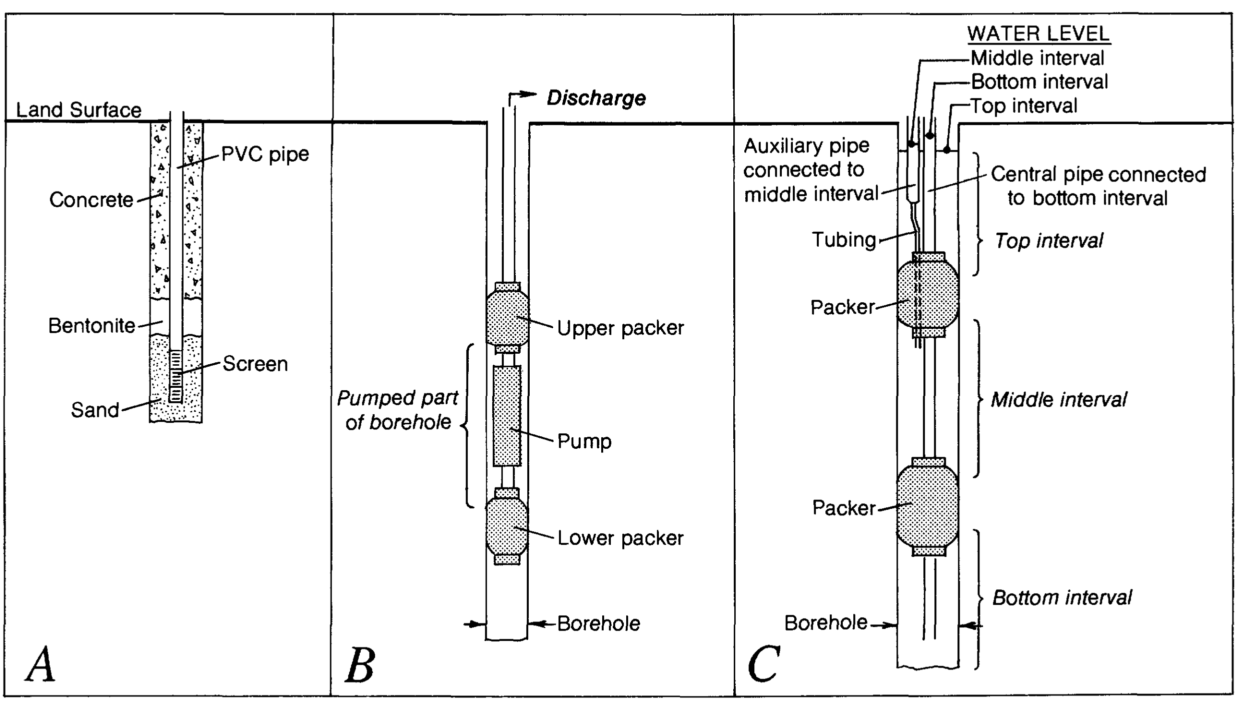


Figure 13. Typical piezometer construction and aquifer-test-equipment configuration. *A*, Piezometer. *B*, Packer/pump assembly. *C*, Packer assembly in an observation well. Two packers are used to separate the well into three observation intervals.

piezometers and observation wells. The models used in this study are summarized below.

Analysis for One-Zone Test

Some of the aquifer tests involve only one zone. For example, where the water table is within the transition zone and the unweathered bedrock has sufficiently low hydraulic conductivity that it can be considered impervious, a one-zone model, representing the transition zone, is used for the test analysis.

The features of the one-zone model are illustrated in figure 14. The zone is bounded above by the water table and below by a horizontal, impervious boundary. Laterally, the zone extends a large distance from the pumped well (that is, there are no lateral boundaries). Hydraulic properties are assumed to be uniform throughout the zone. The pumped and observation wells can fully or partially penetrate the zone. A piezometer is assumed to measure hydraulic head at a point in the zone. An observation well is assumed to measure the average hydraulic head over the screened or open interval. The physical dimensions needed to specify the model, which are listed in figure 14, are assumed to be known from drilling logs and well-construction data. The model parameters to be determined are the hydraulic properties of the zone:

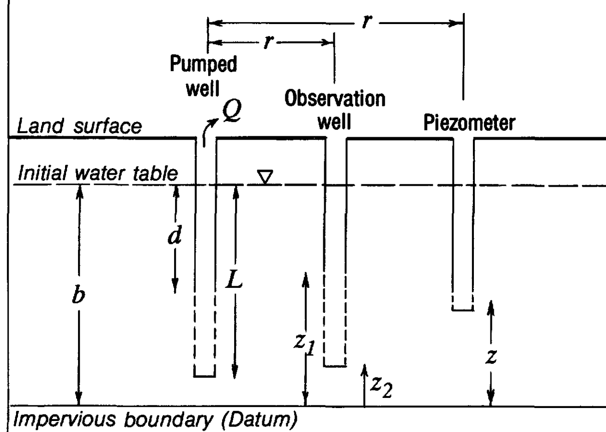
K_r, horizontal hydraulic conductivity;
K_z, vertical hydraulic conductivity;
S_s, specific storage; and (or)
S_y, specific yield.

Mathematical analysis of this model is given by Neuman (1972, 1973, 1974). The model accounts for both horizontal and vertical flow in the zone. During the early stage of the test, water is released primarily from compressive storage, which is characterized by the specific storage, S_s. During the late stage of the test, water is released primarily from lowering of the water table (drainage), which is characterized by the specific yield, S_y. Because drainage does not occur immediately at the start of pumping, the characteristic behavior of a water-table aquifer is known as delayed gravity response. The transition from the early to late stage primarily is determined by the degree of anisotropy in hydraulic conductivity. For further discussion, see Neuman (1972, 1973, 1974).

An important limitation of Neuman's analytical solution is that the drawdown of the water table should be small in comparison with the initial saturated thickness. This criterion was satisfied for all the aquifer tests done in this study.

Analysis for Two- and Three-Zone Tests

Two zones are involved in the aquifer test when (1) the water table is within the regolith and the unweathered bedrock is impervious, or (2) the water table is within the transition zone and the unweathered bedrock is permeable. In the first example, the regolith is the upper zone and the transition zone is the lower zone. In the second, the transition zone is



EXPLANATION

MODEL PARAMETERS

Q = Discharge (L^3/T)

b = Initial saturated thickness of the zone (L)

d = Distance from initial position of the water table to top of the intake interval of the pumped well (L)

L = Distance from initial position of the water table to bottom of the intake interval of the pumped well (L)

r = Distance from the pumped well to the piezometer or observation well (L)

z = Distance from the impervious boundary to the center of the open interval of the piezometer (L)

z_1 = Distance from the impervious boundary to the top of the open interval of the observation well (L)

z_2 = Distance from the impervious boundary to the bottom of the open interval of the observation well (L)

Figure 14. Parameters for one-zone model.

the upper zone and the unweathered bedrock is the lower zone. Both examples are analyzed using a two-zone model developed in this study. The analytical solution is described briefly in appendix 1.

The two-zone model is similar to the one-zone model in all aspects except one—the two-zone model has two distinct zones having different hydraulic properties. There are no restrictions on the properties of either zone; for example, the hydraulic conductivity of the upper zone can be greater or smaller than the hydraulic conductivity of the lower zone. In this respect, the model fully accounts for horizontal and vertical flow in both zones, and therefore is more general than aquifer/confining-unit type models (for example, Cooley and Case, 1973), which assume vertical flow in one layer and horizontal flow in the other.

The essential features of the two-zone model are shown in figure 15. The upper zone is bounded above by the water table, and the lower zone is bounded below by a horizontal, impervious boundary. If the lower zone represents the unweathered bedrock, then one assumes that it becomes impervious below a certain depth. Water may be pumped from either the upper or the lower zone. The model parameters to be determined are

K_{r1}, horizontal hydraulic conductivity of the upper zone;
K_{z1}, vertical hydraulic conductivity of the upper zone;
S_{s1}, specific storage of the upper zone;
S_{y1}, specific yield of the upper zone;
K_{r2}, horizontal hydraulic conductivity of the lower zone;
K_{z2}, vertical hydraulic conductivity of the lower zone; and
S_{s2}, specific storage of the lower zone.

The analytical solution for the two-zone model is subject to the same limitation as the one-zone model—the drawdown of the water table must be small in comparison with the initial saturated thickness of the upper layer. The solution would not apply to cases in which the water table declines into the lower layer, causing the upper zone to become unsaturated during the test. Thus, the specific yield of the lower zone does not enter into the model.

One of the five aquifer-test sites involved all three zones. A three-zone model should be used to analyze the test data. However, an analytical solution for a three-zone model has not been developed owing to mathematical complexity. Therefore, a two-zone model was used for approximate solutions. The justification for using the simpler model is discussed in the section describing aquifer tests at site 6F9.

Effect of Wellbore Storage in Pumped Well

In the models described above, the pumped well is idealized as a line sink of zero radius. This idealization neglects the volume of water that is stored in the wellbore, and it assumes that the rate at which water is pumped from the aquifer is equal to the pump discharge. In reality, when the water level is lowered in the pumped well, a certain volume of water must be removed from the wellbore. The removal of this water from the wellbore causes the actual rate of pumping from the aquifer to be less than the pump discharge. This phenomenon is known as wellbore-storage effect. In general, wellbore-storage effect is negligible if the rate at which water is removed from the wellbore is insignificant in comparison with the rate at which water is pumped from the aquifer. This generally is the case when testing highly permeable aquifers. Crystalline-rock aquifers, on the other hand, generally are characterized by low permeabilities. Aquifer tests in these environments generally require low pumping rates, and a significant portion of the pumped water may be derived from storage in the wellbore. For these cases, neglecting the effect of wellbore storage will result in erroneous interpretation of the aquifer test.

In this study, wellbore storage is accounted for by two different methods, depending on whether one is computing drawdown in the observation well or in the pumped well. To compute drawdown in the observation well, the pumping rate from the aquifer, rather than the pump discharge, is used in the model calculation. The pumping rate from the aquifer is determined in the following manner. On the basis of rate of water-level decline measured in the pumped well, one can determine the rate at which water is removed from the wellbore:

$$Q_w = \pi(r_c)^2 \frac{ds_w}{dt} , \qquad (1)$$

where r_c is the casing radius for the interval of the well in which the water level declines, and s_w is the measured drawdown in the pumped well. If Q_p is the pump discharge and Q_w is the rate at which water is removed from the wellbore, then the rate Q at which water is pumped from the aquifer is

$$Q = Q_p - Q_w . \qquad (2)$$

Note that, in general, Q is not constant. At the start of the test, Q is a small part of Q_p because most of the pumped water is taken from the wellbore (for example, $Q_w \approx Q_p$). As the test progresses, drawdown in the pumped well begins to stabilize, and Q approaches Q_p. These features are illustrated in figure 16, in which Q_p is assumed to be constant. For model calculations, the nonsteady pumping rate Q is approximated by a stairstep-shaped function as shown in figure 16. Drawdown is computed by the superposition of incremental rates starting at different times.

To calculate drawdown in the pumped well, it is necessary to explicitly account for the finite diameter of the well in the mathematical formulation of the aquifer model. For the simple case of a well fully penetrating a confined aquifer of infinite lateral extent,

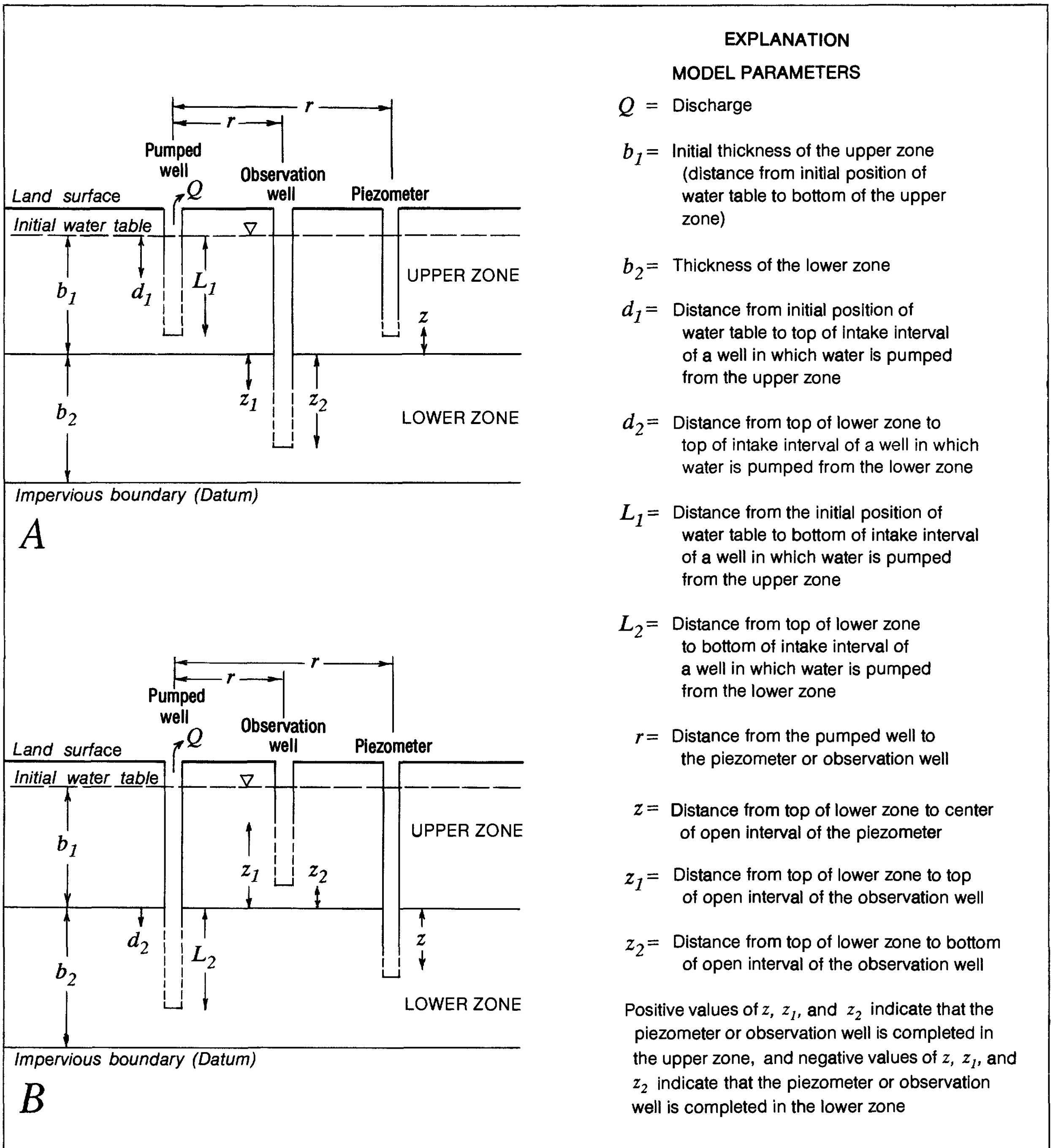


Figure 15. Parameters for two-zone model. *A*, Water pumped from upper zone. *B*, Water pumped from lower zone.

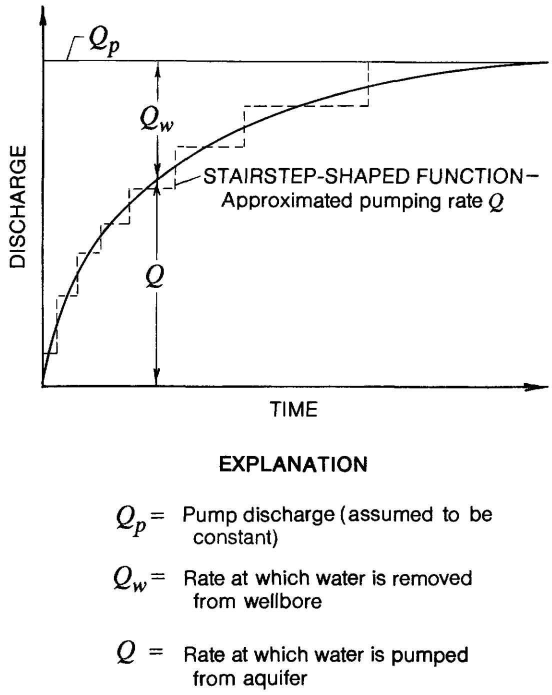


Figure 16. Relation between pump discharge and time, illustrating wellbore-storage effect.

the analytical solution is given by Papadopulos and Cooper (1967). If the well is partially penetrating, the solution of Davis (1987) can be used. For more complicated cases, such as multilayered water-table aquifers, analytical solutions are not available at present. Nevertheless, one can estimate the drawdown at large time, when the effect of wellbore storage becomes unimportant, using the line-sink solutions described above. Hantush (1961, p. 176) suggested that the drawdown in the pumped well can be approximately calculated from the line-sink solution by setting the radial distance, r, equal to the well radius, r_w, and setting z equal to vertical distance from the impervious boundary to the center of the intake interval [for example, $z = (L+d)/2$ for a one-zone model] (see fig. 14). This procedure was adopted for the multizone tests in this study.

Effect of Altered Permeability Around Pumped Well

As discussed earlier, a general characteristic of a fractured-rock aquifer is a high degree of local variation in its hydraulic properties. In analyzing aquifer tests it is neither feasible nor desirable to account for this heterogeneity in full detail. Instead, one attempts to determine the overall properties of the rock on the scale of the test, which is about equal to the distance from the pumped well to the observation wells. Nevertheless, the presence of a local area of altered permeability in the immediate vicinity of the pumped well may exert a strong influence on the drawdown in the pumped well. Although this local area generally can be neglected when simulating drawdowns in observation wells that are far from the pumped well, its influence must be considered when simulating drawdown in the pumped well.

To facilitate a simple analysis, the area of altered permeability is assumed to extend from the wellbore out to a radius r_s (fig. 17). The hydraulic conductivity

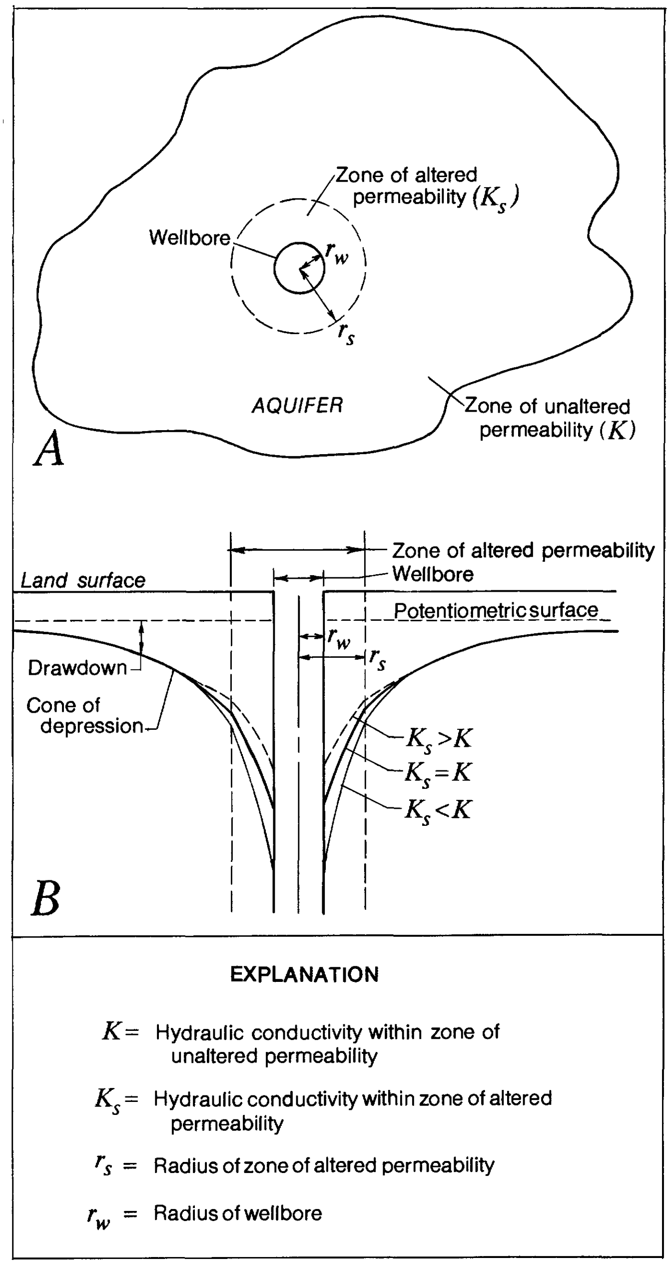


Figure 17. Effects on drawdown of a zone of altered permeability around a pumped well. *A*, Plan view. *B*, Vertical section showing drawdown (cones of depression) for selected hydraulic-conductivity relations.

within this local area is denoted by K_s, and the hydraulic conductivity beyond this local area is K, which is an overall property of the test zone. If K_s is smaller than K (for example, the well is surrounded by a local area of reduced hydraulic conductivity), then the actual drawdown in the pumped well will be greater than the drawdown for the case of $K_s = K$ (fig. 17). To account for the greater drawdown, it is common to introduce the concept of a "skin" around the wellbore. In comparison with the no-skin case ($K_s = K$), the presence of a skin causes an additional drawdown that is proportional to the flow rate into the wellbore. This skin is characterized by a dimensionless quantity, σ, known as the skin factor. Hawkins (1956) shows that

$$\sigma = [(K/K_s) - 1] \ln (r_s/r_w). \qquad (3)$$

An analytical solution accounting for both wellbore storage and skin effect is given by Agarwal and others (1970) for a pumped well that fully penetrates a confined aquifer of infinite lateral extent. For cases involving multilayered aquifers, analytical solutions are not available at present.

If the pumped well is surrounded by a local area of enhanced hydraulic conductivity (that is, $K_s > K$), then the drawdown in the pumped well will be less than the drawdown for $K_s = K$. In effect, the region of enhanced hydraulic conductivity serves to enlarge the wellbore, thus decreasing the drawdown needed for water to flow into the well. To account for the enhanced hydraulic conductivity, the actual wellbore radius r_w can be replaced by an "effective wellbore radius," denoted by r_{we}, which is larger than r_w. The effective wellbore radius can be determined from r_w, r_s, K, and K_s by allowing the skin factor σ to take on negative values. Once σ is computed from equation 3, then r_{we} can be determined (Brons and Miller, 1961) by

$$r_{we} = r_w e^{-\sigma}. \qquad (4)$$

Although the concept of negative skin factor is physically unrealistic, it nevertheless serves a useful purpose because it provides a simple way to determine the effective well radius.

Effect of Delayed Response in Observation Well

An inherent assumption in the aquifer-test models described above is that the observation wells and the piezometers behave in an "ideal" manner. During an aquifer test, the decline of water level in an ideal observation well or piezometer should be identical to the drawdown in the aquifer. In reality, a certain volume of water must flow out of the observation well in order for the water level to decline. If the time needed for the water to exit the well is significantly greater than the time over which the drawdown occurs in the aquifer, then the observation well cannot accurately track the drawdown in the aquifer. In this case, the drawdown in the observation well is delayed in comparison with the drawdown in the aquifer. Although this delay generally is insignificant when testing highly permeable aquifers, its effect must be recognized when testing aquifers with low permeability. Furthermore, if the observation well is surrounded by a local area of reduced permeability, then the delayed effect will be accentuated. In extreme cases, the drawdown in the observation well may differ considerably from the drawdown in the aquifer.

The delayed response of an observation well can be analyzed by considering an instantaneous head decline in the aquifer, as illustrated by the dashed line in figure 18. If the observation well has a fast response, then its water level (solid line in fig. 18) should closely resemble the hydraulic head in the aquifer. If the response is slow, a considerably longer time is needed for the water level to achieve the same head decline (dashed-dotted line in fig. 18). In reality, the well cannot be examined in this way because it is difficult, if not impossible, to create an instantaneous head decline in the aquifer. However, the equivalent effect can be achieved by instantaneously raising the water level in the well and monitoring the subsequent decline, a procedure that is identical to that of a slug test. Thus, although the slug test generally is used as a method to determine aquifer transmissivity (Cooper and others, 1967), it was used in this study to characterize the delayed response of the well.

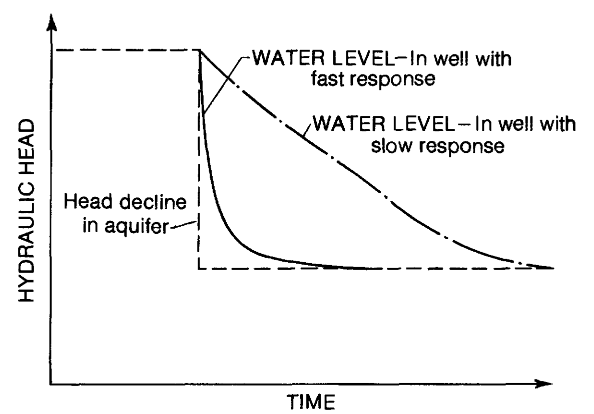


Figure 18. Relation between hydraulic head and time, illustrating delayed response in an observation well.

Past experience and simplified theoretical analysis (Hvorslev, 1951) indicate that the response of a slug test can be approximately described by a negative exponential function. If the initial head increase is H, and after a time, t, the head increase has declined to h, then the normalized head difference, $(H-h)/H$, can be expressed as

$$(H-h)/H = e^{-t/T_o}, \qquad (5)$$

where T_o is known as a characteristic response time. To determine T_o, field data of $(H-h)/H$ are plotted on logarithmic scale against t on arithmetic scale. The data should form a straight line, and T_o is the time corresponding to $(H-h)/H = 0.37$ (fig. 19). Because of the equivalence between the slug test and a stepwise head decline in the aquifer, equation 5 also should describe the water-level response in the observation well, as shown in figure 19. Thus, the characteristic response time, T_o, obtained from a slug test also should characterize the delayed response of the well. For an ideal observation well, T_o is zero.

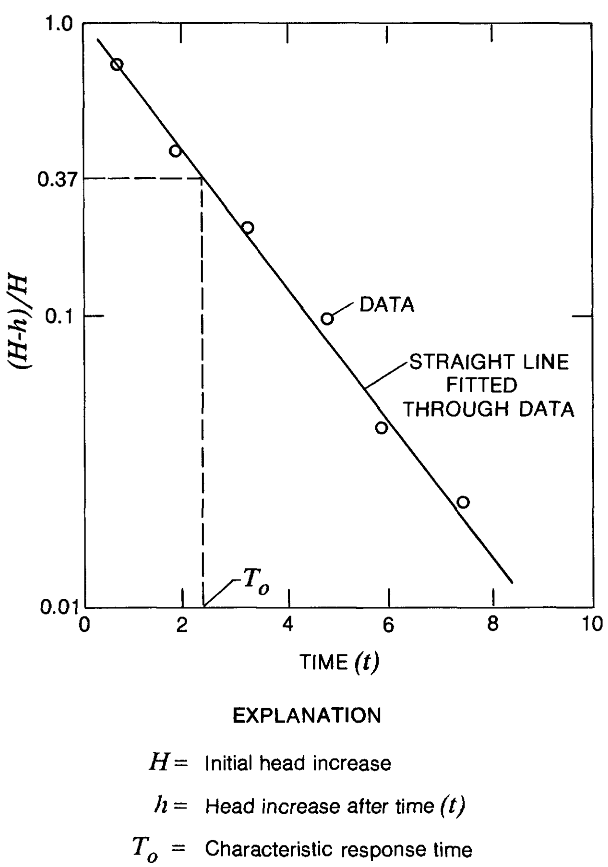


Figure 19. Relation between normalized head difference, $(H-h)/H$, and time for a hypothetical slug test.

Once T_o is known for an observation well, the drawdown in the well during an aquifer test can be determined. In this study, the approach of Black and Kipp (1977) was followed. If $s(t)$ denotes the drawdown in the ideal observation well, then the drawdown in the actual observation well is given by

$$s_{ob}(t) = \frac{1}{T_o} \int_{\tau=0}^{\tau=t} e^{-(t-\tau)/T_o} s(\tau)d\tau, \qquad (6)$$

where τ is the integration variable. [The derivation of this equation is given by Black and Kipp (1977).] Thus, the appropriate analytical solution for the ideal observation well is substituted into the integral in equation 6, and evaluation of the integral yields the drawdown in the actual observation well.

Aquifer Tests at Selected Sites: Test Configurations and Results

Aquifer tests were done at five sites in Lee Valley. The location of test-site pumped wells is shown in figure 1. Each aquifer-test site is identified by the last three characters (number-letter-number) of the well number of the pumped well for the aquifer test. For example, the aquifer-test site at which well 17S/2E-7C1 served as the pumped well is identified as site 7C1.

As noted previously, the purpose of the aquifer tests is to determine the overall hydraulic properties at each site. These properties should be characteristic of the aquifer on a scale that is approximately the distance between the pumped and observation wells (30 to 100 ft). Drawdowns in pumped wells, because they may be greatly influenced by a surrounding zone of altered permeability (wellbore skin), are not used to determine the overall aquifer properties. Instead, aquifer-property values are obtained from analysis of drawdowns in observation wells. After these values are determined, one can ascertain whether a local area of altered permeability (skin) exists around the pumped well. An analysis, accounting for the skin effect, then can be carried out to determine the hydraulic conductivity of the altered area. In the present study, analytical models that account for wellbore storage and skin effect in multilayered aquifers are not available; therefore, skin effects can be analyzed only for sites at which the aquifer consists of one zone.

The data are analyzed using a trial-and-error procedure. First, an appropriate aquifer model is selected. Next, a set of aquifer-property values is selected, and drawdowns are simulated. If the model-simulated drawdowns do not agree with measured drawdowns, the aquifer-property values are changed. This procedure is repeated until the simulated drawdowns match the measured drawdowns to a satisfactory degree.

For each test analysis, a brief sensitivity analysis is done whereby the aquifer properties are varied about the simulated values. The resultant change in simulated drawdown provides a qualitative assessment of the degree to which the various aquifer properties can be identified from the measured data. This assessment is based on subjective judgment, and it carries no statistical implications.

Site 7C1

Site 7C1 is at the southern end of Lee Valley (fig. 1). Except for a thin soil mantle, there is no regolith at the site. The transition-zone thickness, estimated from drillers' logs, is about 40 ft. At the time of the aquifer tests (December 1987), the depth to water was 10 ft below land surface. A privately owned, unused water well (17S/2E-7C1) and two piezometers (17S/2E-7C6 and 17S/2E-7C7) installed by the USGS are at the site. Construction data for the aquifer-test wells and piezometers are given in table 3.

Prior to aquifer testing, slug tests were done in piezometers 17S/2E-7C6 and 17S/2E-7C7 to determine their response characteristics. The characteristic response times are 0.36 minute for 17S/2E-7C6 and 5.22 minutes for 17S/2E-7C7. These results indicate that piezometer 17S/2E-7C6 responds much faster than does piezometer 17S/2E-7C7 to head changes in the aquifer.

One aquifer test was done at this site. A packer was installed in well 17S/2E-7C1 at a depth of 43 ft. The upper interval, from the casing bottom (20 ft) to the top of the packer (42 ft), was pumped for 6 hours 40 minutes. The pumping rate was 2.4 gal/min for the first 5 minutes and 2.0 gal/min for the rest of the test. Measured heads in the pumped well and piezometers are shown in figure 20. The sudden increase in drawdown at an elapsed time of about 2.5 hours is the result of a momentary fluctuation in the pumping rate. Because of a malfunction in the data-logging equipment, hydraulic heads were recorded for only 2 minutes prior to pumping. Thus, it was not possible to determine whether there was a long-term trend (increase or decrease) in hydraulic head.

A second test was attempted by pumping from the interval below the packer (from 44 to 451 ft). However, this interval could not yield enough water to sustain the lowest pumping rate (about 1.2 gal/min) attainable with the equipment. Thus, although the well penetrates 451 ft of rock and the acoustic-televiewer log shows the presence of fractures throughout the open-hole interval, only the upper 20 ft contains fractures that yield enough water for the test. For the purpose of this test analysis, the unweathered bedrock at this site is assumed to be virtually impermeable.

The one-zone model, representing the transition zone, is used for data analysis. The acoustic-televiewer log of well 17S/2E-7C1 shows that fractures in the pumped zone are present between depths of 25 and 40 ft. Thus, the intake interval for the test is assumed to extend over these depths. Physical dimensions (as defined in the section "Analysis for One-Zone Test") used in the model are b = 30 ft, d = 15 ft, L = 30 ft; for piezometer 17S/2E-7C6, r = 24.4 ft and z = 17.5 ft; and for piezometer 17S/2E-7C7, r = 22.2 ft and z = 6 ft. It is assumed that there is no long-term trend in hydraulic head.

The aquifer-property values used to fit the simulated drawdown to the measured drawdown are shown in table 4. The two drawdowns are illustrated in figure 21. One can see that the fit for the early-time data is poorer than the fit for the late-time data. In fact, no combination of aquifer-property values produced drawdown curves that fit all the data, and thus the final fit was selected by subjective judgment. In examining figure 21, one should keep in mind that the log-log plot exaggerates the lack of fit at early time, when the drawdowns are small. When viewed in the context of the entire set of drawdown data, the lack of fit during early time is not severe.

Sensitivity analysis indicates that the horizontal and vertical hydraulic conductivities can be determined to within about 50 percent of the estimated value. Because of the poor fit of the data at early time, subjective judgment is required in selecting the specific-storage value, and it should be considered an order-of-magnitude estimate. The zero value of specific yield indicates that the transition zone at the site is locally confined. This conclusion is supported by the presence of barometric fluctuations in long-term water levels recorded at well 17S/2E-7C1. For a shallow aquifer, the presence of barometric fluctuations generally indicates that the aquifer is confined. Under confined conditions, aquifer material always is saturated; therefore, no water is drained from the pore spaces of the aquifer material, and specific yield cannot be determined using an aquifer test.

Table 3. Construction data for aquifer-test wells and piezometers

[All depths in feet below land surface. Altitudes in feet above sea level. Diameters of casings and holes in inches. --, not applicable or no data]

Piezometer or well No.	Depth of well	Depth of piezometer screen (top-bottom)	Depth of casing	Diameter of casing	Diameter of open hole	Altitude at top of casing	Altitude of land surface
Site 7C1							
17S/2E-7C1	451	--	20	6.5	6.25	1,512.0	1,511.0
17S/2E-7C6	--	21.5-23.5	--	2	--	1,512.31	1,511.55
17S/2E-7C7	--	33-35	--	2	--	1,512.15	1,511.55
Piezometer 7C6 is 24.4 feet east of well 7C1. Piezometer 7C7 is 22.2 feet S. 75° E. of well 7C1.							
Site 5N5							
17S/2E-5N5	320	--	39	6.63	6.63	1,660.7	1,660.0
17S/2E-5N7	328	--	58	6	6	1,660.66	1,659.66
Well 5N7 is 29.2 feet N. 56° W. of well 5N5.							
Site 31N4							
16S/2E-31N4	457	--	42	6	6	1,734.0	1,733.0
16S/2E-31N6	311	--	57	6	6	1,733.69	1,730.52
Well 31N6 is 31.0 feet N. 63° E. of well 31N4.							
Site 6R2							
17S/2E-6R2	203	--	25	6	6	1,650.3	1,649.0
17S/2E-6R3	159	--	18	6	[1]6,4	1,651.1	1,650.0
Well 6R3 is 30.8 feet N. 80° E. of well 6R2.							
Site 6F9							
17S/2E-6F9	198	--	45	6	6	1,637.9	1,637.0
17S/2E-6F14	189	--	58	6	6	1,639.11	1,636.65
17S/2E-6F10	--	30.0-32.0	--	2	--	1,636.26	1,635.44
17S/2E-6F11	--	43.5-45.5	--	2	--	1,635.69	1,635.03
17S/2E-6F12	--	54.0-56.0	--	2	--	1,640.06	1,639.46
17S/2E-6F13	--	35.0-37.0	--	2	--	1,634.12	1,633.32
Well 6F14 is 96.5 feet N. 35° E. of well 6F9. Piezometer 6F10 is 26.7 feet N. 77° E. of well 6F9. Piezometer 6F11 is 32.9 feet N. 77° E. of well 6F9. Piezometer 6F12 is 62.9 feet S. 16° E. of well 6F9. Piezometer 6F13 is 63.2 feet N. 83° E. of well 6F9.							

[1]Diameter of open hole is 6 inches from a depth of 18 to 82 feet and is 4 inches from 82 to 159 feet.

Using aquifer-property values obtained from the above analysis, one can now simulate the drawdown in the pumped interval. The upper curve in figure 21*B*, obtained by setting $r = r_w = 0.25$ ft and $z = 7.5$ ft, shows that the simulated drawdowns are larger than the measured drawdowns for the entire test. The lack of fit during early time results from neglecting wellbore storage. The lack of fit at late time indicates that the wellbore may be surrounded by a local area of higher permeability rocks. Using an effective wellbore radius (r_{we}) of 1.0 ft substantially improves the match between the simulated and measured drawdowns (lower curve of fig. 21*B*). The skin factor is simulated, using equation 4, to be −1.4. If the area of enhanced permeability is assumed to extend between 1 and 5 ft from the pumped well, then according to equation 3, the hydraulic conductivity of rocks that immediately surround the pumped well should be about two to three times the overall value at the site.

Site 5N5

Site 5N5 is near the southeastern end of Lee Valley (fig. 1). Thickness of the regolith at the site is about 60 ft. Well logs indicate that the transition zone extends from the bottom of the regolith to a depth of about 110 ft. At the time of the aquifer test (December 1987), depth to water was about 77 ft below land surface. At the site, there are two wells, 17S/2E-5N5 and 17S/2E-5N7. The former is a privately owned, unused water well. The latter was drilled by the USGS for this study. Construction data for both wells are given in table 3.

Three packers were installed in well 17S/2E-5N7 at depths of 118, 191, and 248 ft. The observation intervals are 77 to 117 ft (top interval), 119 to 190 ft (upper-middle interval), 192 to 247 ft (lower-middle interval), and 249 to 328 ft (bottom interval). Response to packer inflation was not recorded. Thus, the characteristic response times of the intervals are not known.

One aquifer test was done by pumping from well 17S/2E-5N5, in which a packer was installed at a depth of 118 ft. The interval above the packer was pumped for 7 hours at a constant rate of 1.9 gal/min. Measured water levels are shown in figure 22. Note that the water level in only the top observation interval in well 17S/2E-5N7 showed response to pumping. The water levels in the upper-middle and lower-middle intervals continued their gradual rise or decline toward their equilibrium values. The transducer in the bottom interval malfunctioned during the first half of the test, but data recorded during the second half show that this interval also did not respond to pumping.

A second test was attempted by pumping from the interval below the packer in well 17S/2E-5N5. However, the interval could not yield enough water to sustain the lowest pumping rate of the pump. Thus, the bedrock is assumed to be sufficiently impermeable that it can be neglected in the aquifer test.

The one-zone model, representing the transition zone, is used for data analysis. Both the pumped interval in well 17S/2E-5N5 and the top interval in well 17S/2E-5N7 fully penetrate the transition zone. Thus, the physical dimensions (defined in the section "Analysis for One-Zone Test") used in the model are $b = 33$ ft, $d = 0$ ft, $L = 33$ ft, $r = 29.2$ ft, $z_1 = 33$ ft, and $z_2 = 0$ ft. It is assumed that the upper observation interval in well 17S/2E-5N7 has a sufficiently fast response that delay effects may be neglected.

The aquifer-property values used to fit the simulated drawdowns to the measured drawdown are

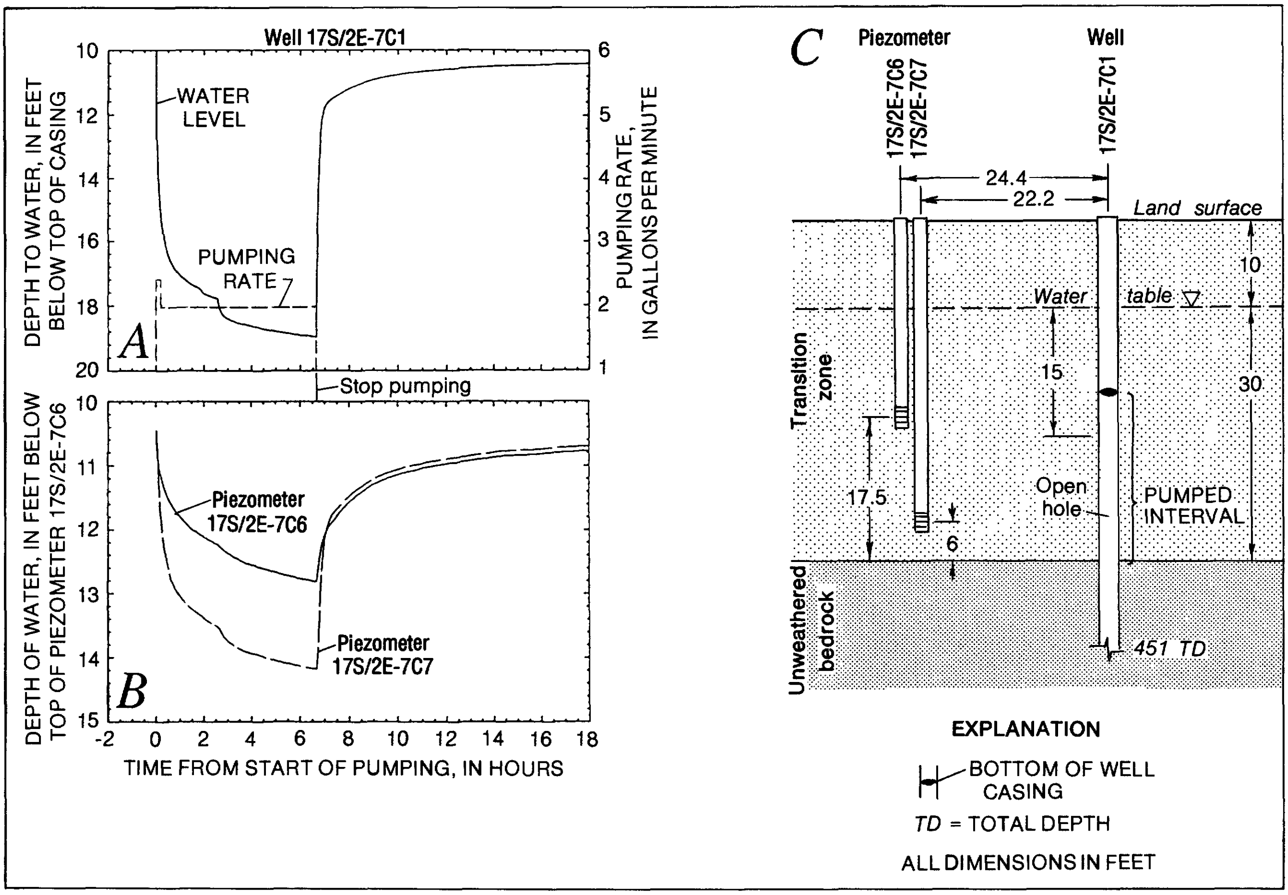


Figure 20. Measured water level at site 7C1 in pumped interval of well 17S/2E-7C1 (*A*) and in piezometers 17S/2E-7C6 and 17S/2E-7C7 (*B*). Schematic diagram (*C*) illustrates configuration of wells and piezometers.

Table 4. Aquifer-property values determined from aquifer tests

[Construction data for test wells and piezometers are given in table 3. For site 6F9, each test has two lines of entries for each of the hydrogeologic zones tested. The regolith aquifer-property values for test 1 are from analysis of data from piezometers 17S/2E-6F10 and 17S/2E-6F13. The transition-zone values for both test 1 and test 2 are from analysis of data from piezometers 17S/2E-6F11 and 17S/2E-6F12. The unweathered bedrock aquifer-property values for test 2 are from analysis of data from the top and bottom intervals of observation well 17S/2E-6F14]

Site No.	Hydrogeologic zone	Horizontal hydraulic conductivity (foot per second)	Vertical hydraulic conductivity (foot per second)	Specific storage (per foot)	Specific yield (unitless)
7C1	regolith[1]				
	transition zone	2.8×10^{-5}	7.5×10^{-7}	5×10^{-6}	[2]0
	unweathered bedrock[3]				
5N5	regolith[4]				
	transition zone	1.8×10^{-5}	2.0×10^{-5}	3.0×10^{-5}	2×10^{-2}
	unweathered bedrock[3]				
31N4	regolith[4]				
	transition zone	1.3×10^{-5}	2×10^{-4}	[5]1×10^{-6}	6×10^{-3}
	unweathered bedrock[3]				
6R2					
Test 1	regolith[1]				
	transition zone	3.8×10^{-5}	2×10^{-5}	2.5×10^{-5}	[2]0
	unweathered bedrock	3×10^{-7}	2×10^{-5}	1×10^{-7}	([6])
Test 2	regolith[1]				
	transition zone	5.0×10^{-5}	1×10^{-5}	5.0×10^{-5}	[2]0
	unweathered bedrock	1.0×10^{-6}	3.0×10^{-6}	1×10^{-7}	([6])
6F9					
Test 1	regolith	2×10^{-4}	8×10^{-5}	[7]1×10^{-5}	1×10^{-2}
		7×10^{-5}	1×10^{-5}	[7]1×10^{-5}	1×10^{-2}
	transition zone	2.5×10^{-6}	7×10^{-7}	1.8×10^{-6}	([6])
		2.5×10^{-6}	4×10^{-7}	3.6×10^{-6}	([6])
	unweathered bedrock[6]				
Test 2	regolith[6]				
	transition zone	4×10^{-6}	3×10^{-7}	1×10^{-6}	([6])
		2×10^{-6}	2×10^{-7}	2×10^{-6}	([6])
	unweathered bedrock	3×10^{-7}	5×10^{-8}	1×10^{-7}	([6])
		2×10^{-7}	1×10^{-5}	3×10^{-7}	([6])

[1]Absent. [2]Confined. [3]Nonproductive. [4]Unsaturated. [5]Unreliable. [6]Not tested. [7]Assumed value.

given in table 4. The model fits the data well (see fig. 23A). The delayed gravity response of the water table is well illustrated by the initial rise of the drawdown curve at early time, the flattening of the curve at intermediate time, and the rise at late time. At early time, water is released primarily from compressive storage, and at late time, water is released from lowering of the water table.

The test results suggest that the transition zone is nearly isotropic. The specific yield of 2×10^{-2} is within the range of fracture porosity of many fractured crystalline rocks. Because the test did not last long enough for the late-time drawdown curve to develop fully, the estimates of horizontal hydraulic conductivity and the specific yield probably have an error range of plus or minus 50 percent. Sensitivity analysis shows that the vertical hydraulic conductivity and the specific storage also are subject to the same error range.

The drawdown measured in the pumped interval and the simulated drawdown that was obtained using the model in conjunction with the aquifer-property values determined from the observation-well data (table 4) are shown in figure 23*B*. The upper curve is simulated by letting $r = r_w = 0.25$ ft and $z = 16.5$ ft. The simulated drawdowns are about 1.5 times the measured drawdowns, indicating that the wellbore may be surrounded by a local area of higher permeability rocks. An improved fit is obtained by assuming an effective wellbore radius of 1.3 ft (lower curve in fig. 23*B*). This value is similar to that obtained for the pumped well at site 7C1 and suggests that the hydraulic conductivity of the rocks surrounding the wellbore is about two to three times the overall hydraulic conductivity of the aquifer.

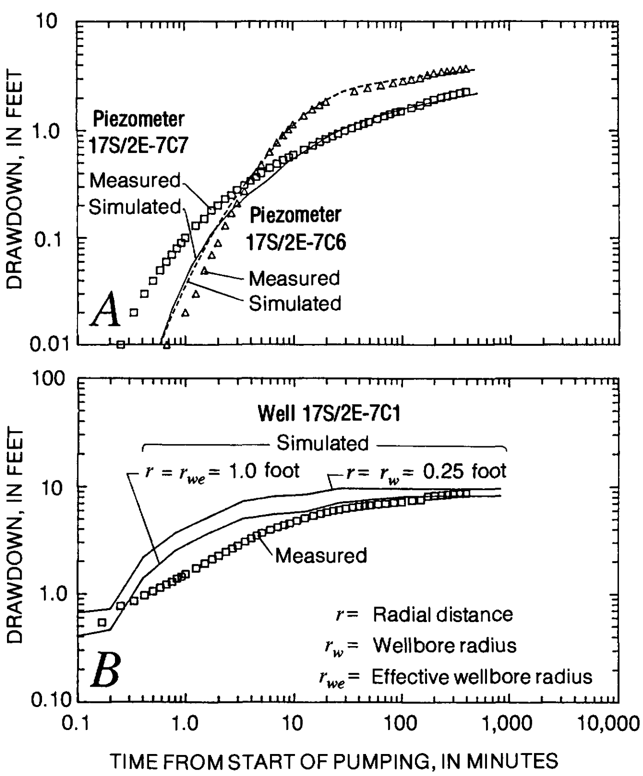


Figure 21. Measured and simulated drawdowns at site 7C1 in piezometers 17S/2E-7C6 and 17S/2E-7C7 (*A*) and in pumped well 17S/2E-7C1 (*B*).

Site 31N4

Site 31N4 is near the northwestern end of Lee Valley (fig. 1). At the site, the regolith extends to a depth of about 40 ft, below which the transition zone extends to about 90 ft below land surface. At the time of the aquifer tests (January 1988), the depth to water was about 46 ft. At the site, there are two wells. Test well 16S/2E-31N4 is a privately owned, unused water well. Observation well 16S/2E-31N6 was drilled by the USGS. Construction data for both wells are given in table 3.

Three observation intervals were created in well 16S/2E-31N6 by installing packers at depths of 97 and 186 ft. The top interval extended from the water table (46 ft) to 96 ft, the middle interval extended from 98 ft to 183 ft, and the bottom interval extended from 187 ft to the bottom of the well (311 ft). Responses of the three intervals to packer inflation are illustrated in figure 24. The characteristic response time of the top interval was 1.87 minutes. Note that the heads in the middle and bottom intervals continued to decline well after the head in the top interval had equilibrated. In fact, these two heads declined below the head in the top interval and did not reach their equilibrium levels until several days after the aquifer test. The final equilibrium heads in the three intervals are reversed from those shown in figure 24 and show a strong downward hydraulic gradient, which provides a driving force for downward movement of water. This test site is in the recharge area of Lee Valley, and thus downward water movement is expected. The strong downward gradient may indicate either a large downward water flux or a poor connection between the transition zone and the unweathered bedrock. Of the two possibilities, the second is supported by the results of the aquifer test.

One test was done at this site. A packer was installed in the pumped well 16S/2E-31N4 at a depth of 100 ft. The top interval, from 46 to 98 ft, was pumped for 7 hours 40 minutes. The pumping rate initially was set at 1.5 gal/min, but soon after the start of the test, water level in the pumped interval declined to near the level of the pump intake. A recirculation system was set up whereby a portion of the pumped water was returned to the well, and the pumping rate eventually was decreased to 0.5 gal/min. During the entire test, the hydraulic heads in the middle and bottom observation intervals of well 16S/2E-31N6 continued their steady decline, and there were no measurable deviations caused by pumping. Shown in figure 25 are the measured head in the top observation interval in well 16S/2E-31N6, the pumped interval in well 16S/2E-31N4, and the pumping rates. Note that the head in the pumped interval began to recover when the pumping rate was decreased to 0.5 gal/min. The pretest head in the top observation interval showed a slight upward trend. This trend was extrapolated into the pumping period to calculate drawdowns.

A second test was attempted by pumping from the interval below the packer in well 16S/2E-31N4. However, the interval could not yield enough water to sustain the lowest pumping rate of the equipment.

Thus, the unweathered bedrock is assumed to have very low permeability at the site.

The one-zone model, representing the transition zone, is used for analysis. The pumped interval in well 16S/2E-31N4 is assumed to penetrate the entire thickness of the transition zone, and the top observation interval in well 16S/2E-31N6 is assumed to monitor the depth-averaged head in the transition zone. The physical dimensions (see the section "Analysis for One-Zone Test") used in the model are $b = 44$ ft, $d = 0$ ft, $L = 44$ ft, $r = 31.0$ ft, $z_1 = 44$ ft, and $z_2 = 0$ ft.

The aquifer-property values determined from the analysis are given in table 4. The measured drawdown

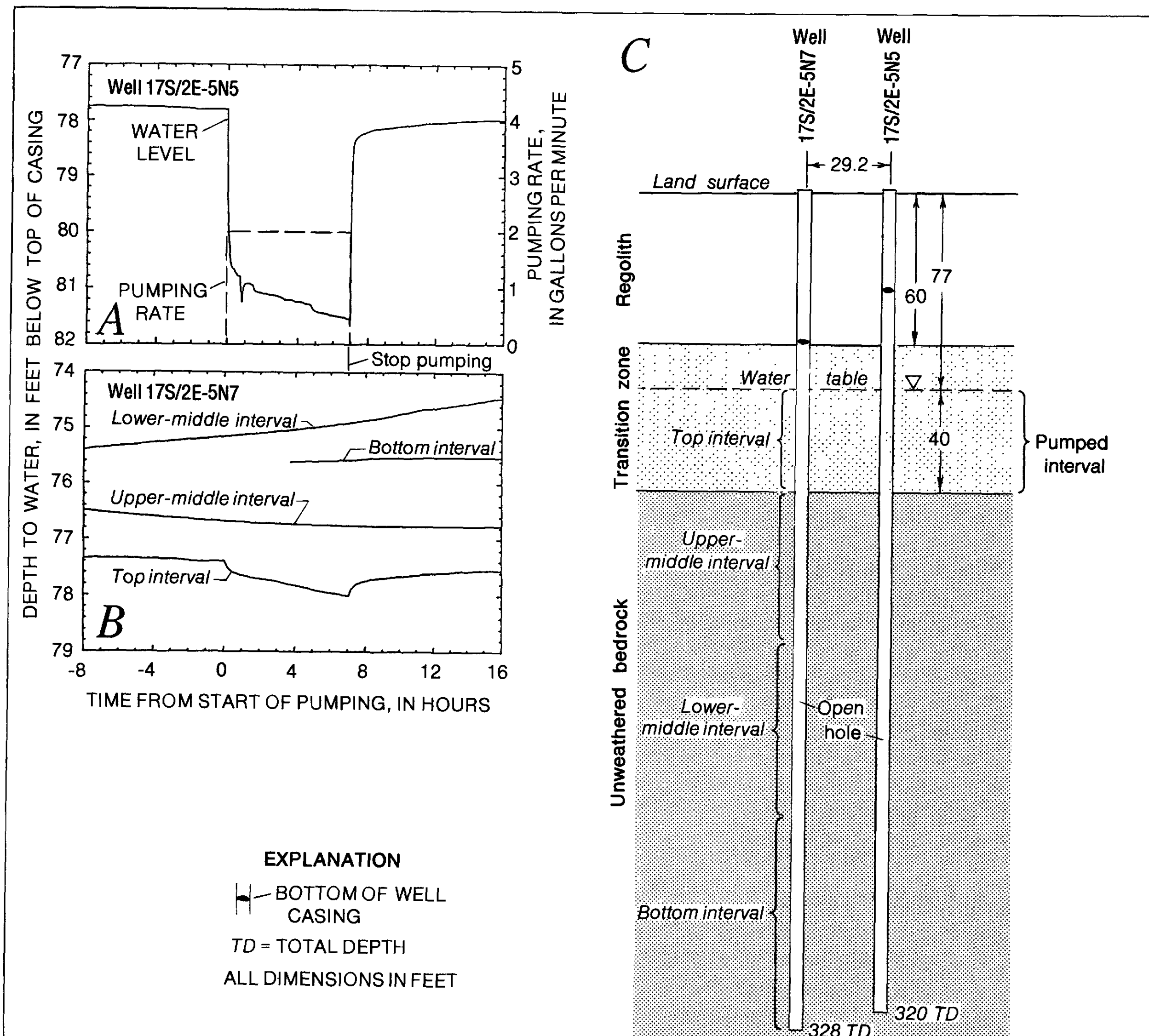


Figure 22. Measured water level at site 5N5 in pumped interval of well 17S/2E-5N5 (*A*) and in well 17S/2E-5N7 (*B*). Schematic diagram (*C*) illustrates configuration of wells.

data are fitted to the "late-time" part of the simulated drawdown curve; that is, water is assumed to be released from lowering of the water table. This assumption is made because fitting the data to the "early-time" part of the simulated drawdown curve would yield a specific storage of about 1×10^{-4}/ft, which is several orders of magnitude larger than what commonly is expected for fractured crystalline rocks. The calculated drawdowns fit the measured drawdowns well (see fig. 26). Sensitivity analysis indicates that the horizontal hydraulic conductivity and the specific yield were determined within an error range of about plus or minus 25 percent. The value of 6×10^{-3} obtained for specific yield is low but within the range of fracture porosity for fractured crystalline rocks. The value obtained for the vertical hydraulic conductivity is a lower bound; higher values do not change the simulated drawdown curve. Thus, the hydraulic conductivity of the transition zone seems to be at least 10 times higher in the vertical direction than in the horizontal direction. Because of the high vertical hydraulic conductivity, drainage of the water table can occur relatively soon after the start of pumping, and the test becomes almost entirely insensitive to the specific storage. Although the value of

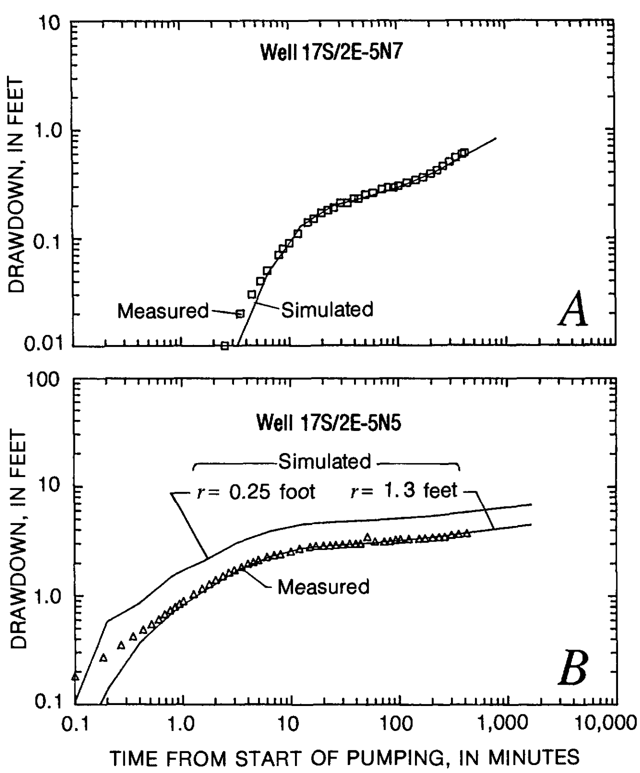


Figure 23. Measured and simulated drawdowns at site 5N5 in top interval of well 17S/2E-5N7 (*A*) and in pumped interval of well 17S/2E-5N5 (*B*).

1×10^{-6}/ft for specific storage was selected as a reasonable value to be used for model calculation, it may not be representative of the transition zone at this site. This value can be varied by several orders of magnitude without affecting the simulated drawdown curve.

Analysis of the drawdown at the pumped interval shows that the effect of wellbore storage persisted over almost the entire test period. Because of this persistence, it would be inappropriate to neglect wellbore storage by setting the radial distance equal to the wellbore radius in the one-zone model. Instead, the wellbore-skin model of Agarwal and others (1970) is used in the analysis. As noted in the section "Effect of Wellbore Storage in Pumped Well," this model accounts for both wellbore storage and skin effects of a pumped well that fully penetrates a confined aquifer. To apply this model to a water-table aquifer, two additional conditions must be met: (1) The drawdown of the water table must be small so that the saturated thickness remains virtually constant, and (2) the vertical hydraulic conductivity must be sufficiently large that there is little delay in water-table drainage. For the study case, both conditions are satisfied.

The measured and simulated drawdowns for various assumed values of skin factor, σ, are shown in figure 26. Note that in the absence of skin effect (σ = 0), the simulated drawdown at late time is only about one-fifteenth as great as the measured drawdown. To obtain a match between the measured and simulated drawdowns, a skin factor of 90 is required. Such a skin factor implies that the pumped well is surrounded by a local area of less permeable rocks. Assuming that this region extends between 1 and 5 ft away from the wellbore, equation 3 yields a local hydraulic conductivity that is about 0.03 times the overall hydraulic

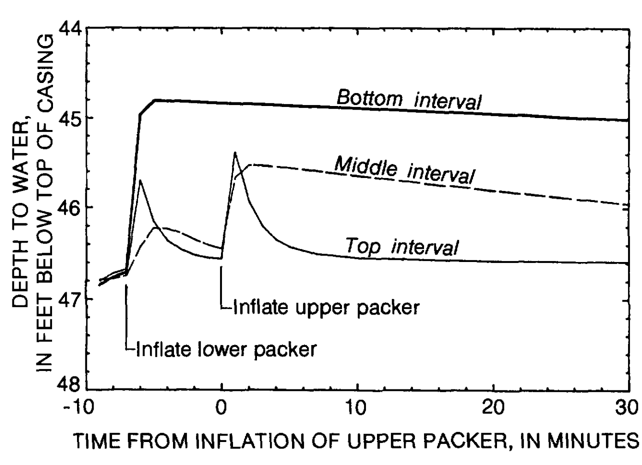


Figure 24. Water-level response to packer inflation in well 16S/2E-31N6 at site 31N4.

conductivity of the aquifer. If this area of lower permeability were not taken into account, then analysis of the pumped-well drawdown would significantly underestimate the overall hydraulic conductivity of the aquifer.

Site 6R2

Site 6R2 is in the east-central part of Lee Valley (fig. 1). Except for a thin mantle of soil, there is no regolith at the site. Well logs indicate that the transition zone extends from land surface to a depth of about 80 ft. At the time of the aquifer tests (January 1988), the depth to water was about 38 ft from land surface. At the site are two wells, 17S/2E-6R2 and 17S/2E-6R3. The former is a privately owned, unused water well. The latter was drilled by the USGS for this study. Construction data for both wells are given in table 3.

To create two observation intervals in well 17S/2E-6R3, a packer was installed at a depth of 86 ft. The top interval extends from the water table to 84 ft, and the bottom interval extends from 88 ft to the bottom of the well (159 ft). The responses of the two intervals to packer inflation are illustrated in figure 27. Note that the bottom interval has a considerably slower response than the top interval. The characteristic response times, as determined by the method

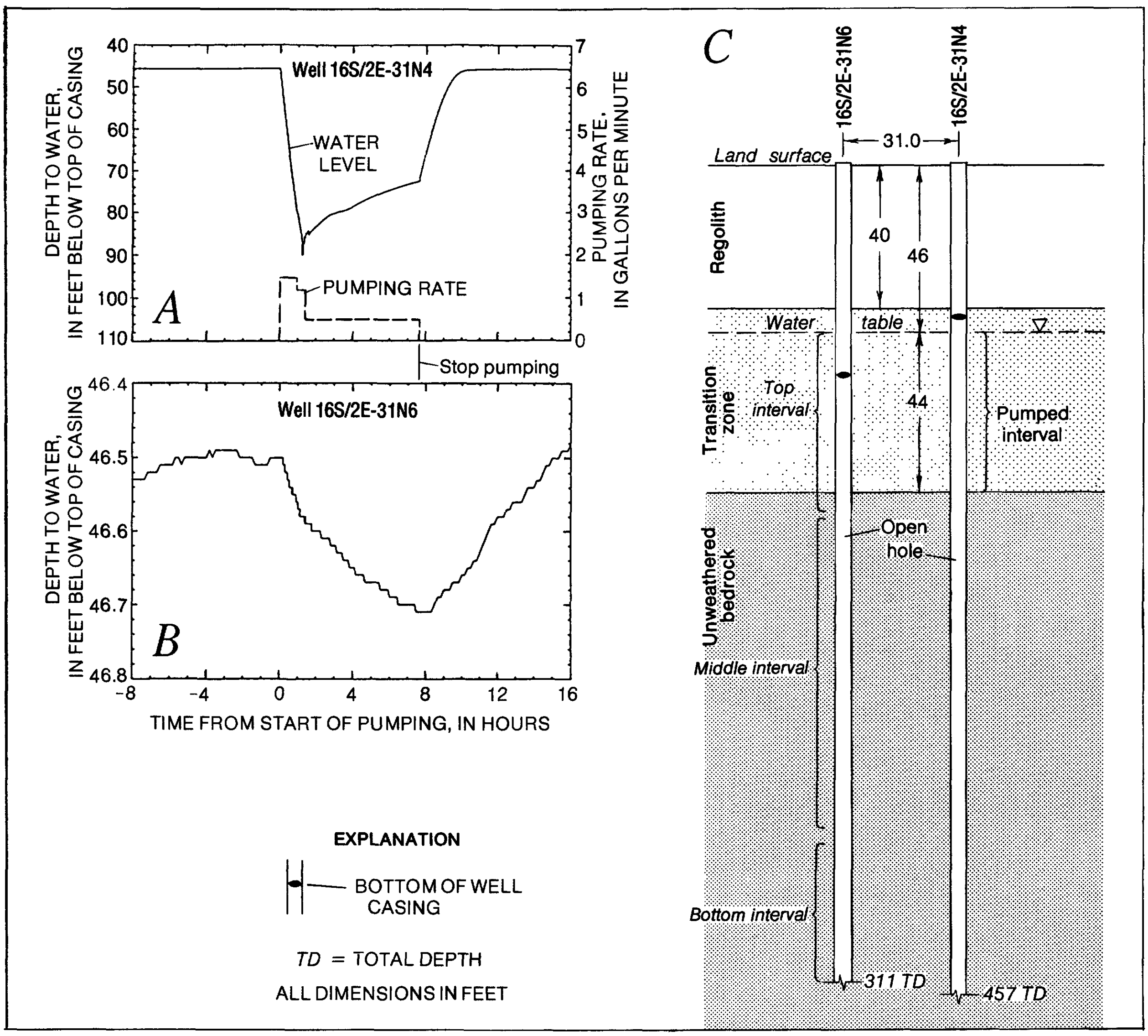


Figure 25. Measured water level and pumping rate at site 31N4 in pumped interval of well 16S/2E-31N4 (*A*), and measured water level at site 31N4 in top interval of well 16S/2E-31N6 (*B*). Schematic diagram (*C*) illustrates configuration of wells.

shown in figure 19, are 4.1 minutes for the top interval and 105 minutes for the bottom interval.

Two tests were done at this site by pumping from well 17S/2E-6R2. In test 1, a packer was installed at a depth of 100 ft in well 17S/2E-6R2. The top interval, from the water table (38 ft) to the top of the packer (84 ft), was pumped for 3 hours 20 minutes. The initial pumping rate was 3 gal/min, but as the water level in the pumped interval declined to near the pump intake, the pumping rate was decreased several times, eventually to 1.5 gal/min. The pumping rate and the measured hydraulic heads in the pumped (top interval) and observation intervals (both top and bottom intervals) are shown in figure 28. Recovery of hydraulic head in the pumped interval can be seen when the pumping rate was decreased to 1.5 gal/min. In test 2, a packer was installed at a depth of 87 ft in well 17S/2E-6R2. The bottom interval, from 89 ft to the bottom of the well (203 ft), was pumped at a constant rate of 1.3 gal/min for 4 hours. Measured hydraulic heads in the pumped (bottom interval) and observation (top and bottom) intervals are shown in figure 29.

The two-zone model is used for analysis. The upper zone represents the transition zone, and the lower zone represents the unweathered bedrock. It is assumed that the bedrock becomes impermeable below a depth of 200 ft (approximate depth of well 17S/2E-6R2). In observation well 17S/2E-6R3, the water level in the top interval is assumed to represent the average head over the entire thickness of the transition zone, and the water level in the bottom observation interval is assumed to represent the average head from the top of the unweathered bedrock to the bottom of well 17S/2E-6R3. The physical dimensions (as defined in the section "Analysis for Two- and Three-Zone Tests") used in the two-zone model are as follows: b_1 = 42 ft, b_2 = 120 ft, and r = 30.8 ft; for the upper interval in 17S/2E-6R3, z_1 = 42 ft and z_2 = 0 ft; for the lower interval, z_1 = 0 ft and z_2 = −79 ft.

In both tests, the packer in the pumped well was installed slightly below the inferred contact between the transition zone and the unweathered bedrock. For the purpose of the analysis, however, pumpage from the top interval (test 1) is assumed to be entirely from the transition zone, and pumpage from the bottom interval (test 2) is assumed to be entirely from the bedrock. Thus, for test 1, d_1 = 0 ft and L_1 = 42 ft. For test 2, d_2 = 0 ft and L_2 = 120 ft.

The aquifer-property values obtained from analysis of test 1 are given in table 4. The measured and simulated drawdowns are shown in figure 30. Sensitivity analysis shows that the horizontal hydraulic conductivity and the specific storage of the transition zone could be determined with an error range of about plus or minus 25 percent. The values for the horizontal hydraulic conductivity and the specific storage of the unweathered bedrock are both upper bounds; the slow response time of the bottom observation interval in well 17S/2E-6R3 causes a lack of

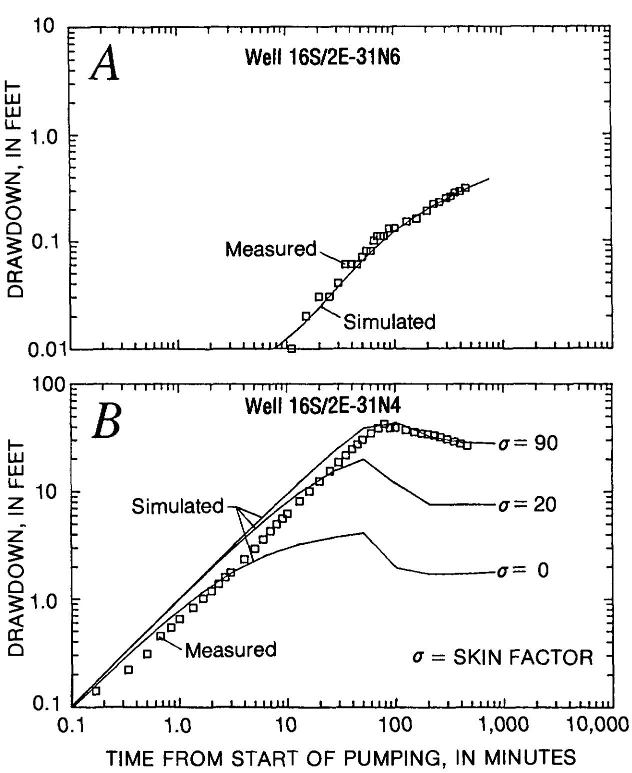


Figure 26. Measured and simulated drawdowns at site 31N4 in top interval of well 16S/2E-31N6 (*A*) and in pumped interval of well 16S/2E-31N4 (*B*).

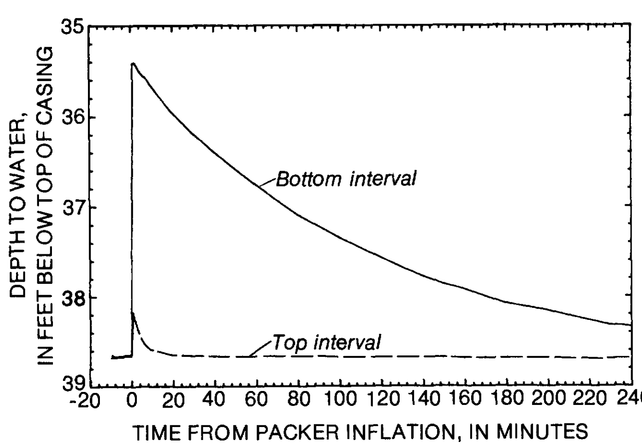


Figure 27. Water-level response to packer inflation in well 17S/2E-6R3 at site 6R2.

sensitivity. The values for the vertical hydraulic conductivities of both zones are lower bounds; increasing these values does not change the quality of the fit. The zero value of specific yield could indicate that the transition zone is locally confined at this site. This conclusion is supported by the presence of barometric fluctuations in long-term water-level records in well 17S/2E-6R2.

Aquifer-property values obtained from analysis of test 2 are given in table 4. As shown in figure 31, the simulated drawdowns fit the measured drawdowns well. Sensitivity analysis shows that the aquifer-property values can be varied slightly in offsetting directions so that the resultant fits are not changed substantially. The horizontal hydraulic conductivity and the specific storage of the transition zone, as well as the horizontal and vertical hydraulic conductivity of the unweathered bedrock, can be determined with an error range of about plus or minus 50 percent. The vertical hydraulic conductivity of the transition zone and the specific storage of the bedrock should be regarded as order-of-magnitude estimates.

A comparison of the results of test 1 and test 2 (table 4) shows that the aquifer-property values for the transition zone obtained from both tests are within a factor of two from one another. The values for the unweathered bedrock indicate a larger variation (about an order of magnitude). Despite these variations, the two tests together provide a fairly consistent picture of the fractured-rock aquifer at the site. The horizontal hydraulic conductivity of the transition zone is about 1 to 2 orders of magnitude greater than that of the unweathered bedrock. The specific storage of the transition zone is at least two orders of magnitude greater than that of the unweathered bedrock. The transition zone may be slightly anisotropic, the horizontal hydraulic conductivity being two to five

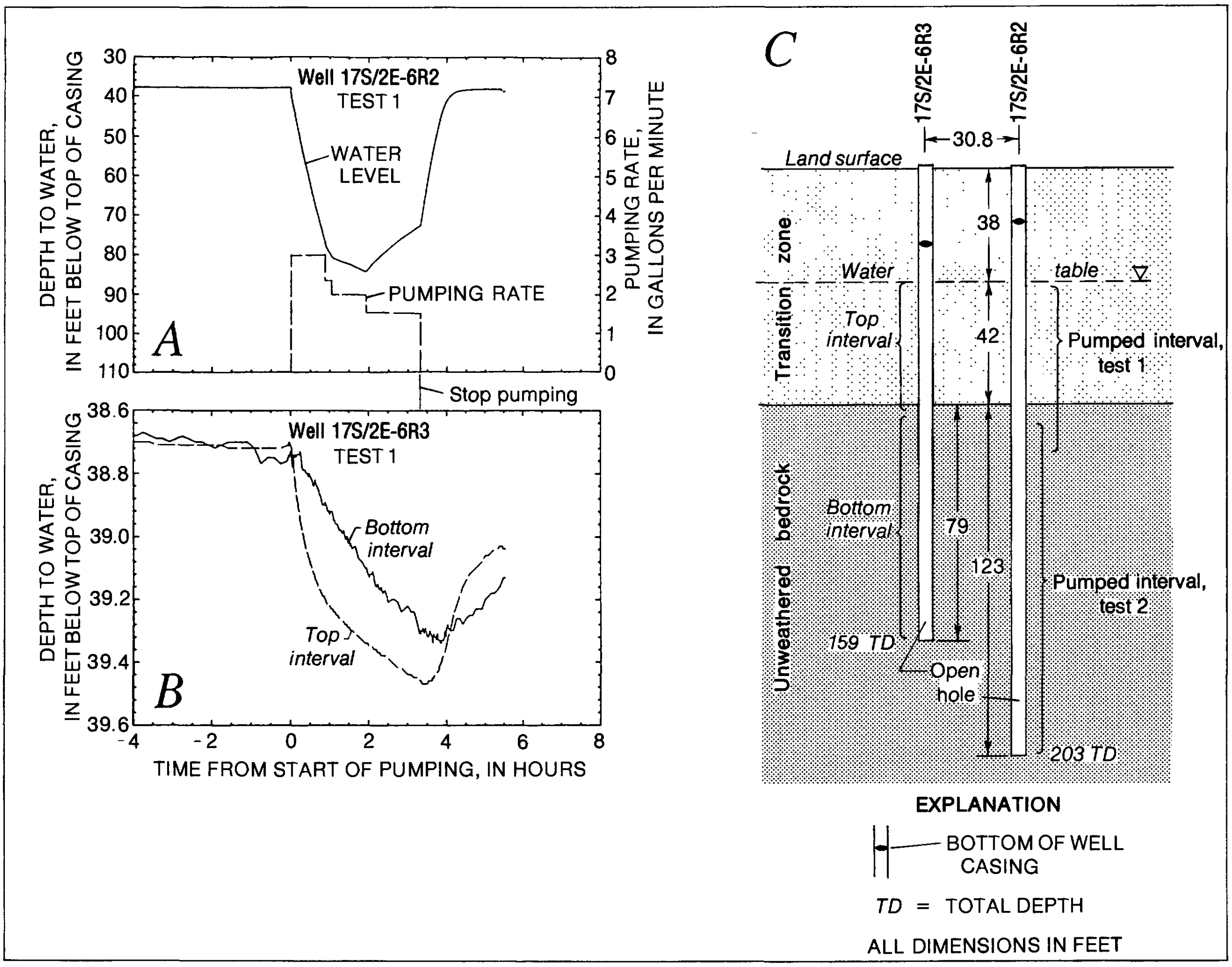


Figure 28. Measured water level and pumping rate at site 6R2 in pumped interval of well 17S/2E-6R2, test 1 (*A*); and measured water levels at site 6R2 in top and bottom intervals of well 17S/2E-6R3, test 1 (*B*). Schematic diagram (*C*) illustrates configuration of wells for test 1 and test 2.

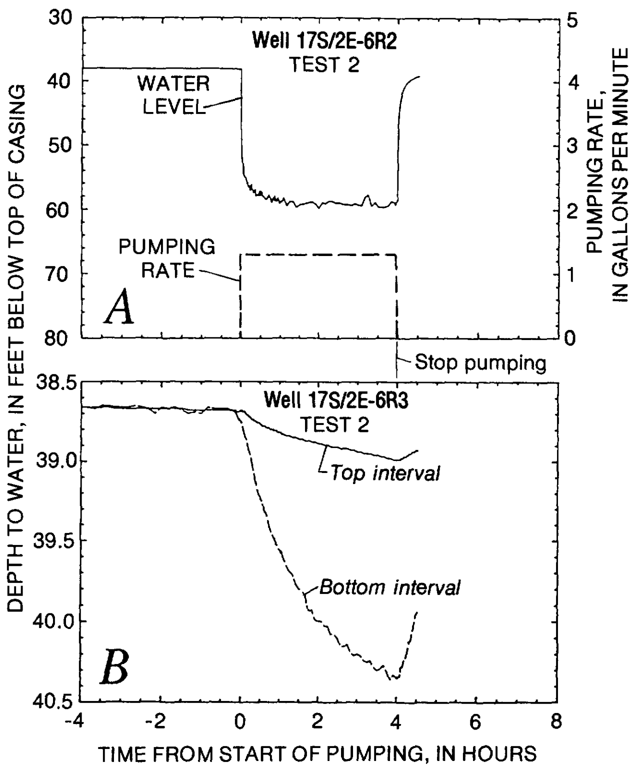


Figure 29. Measured water level and pumping rate at site 6R2 in pumped interval of well 17S/2E-6R2, test 2 (*A*); and measured water levels at site 6R2 in top and bottom intervals of well 17S/2E-6R3, test 2 (*B*).

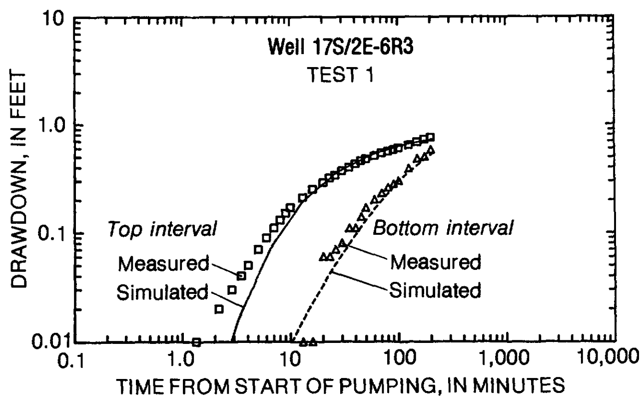


Figure 30. Measured and simulated drawdowns at site 6R2 in top and bottom intervals of observation well 17S/2E-6R3, test 1.

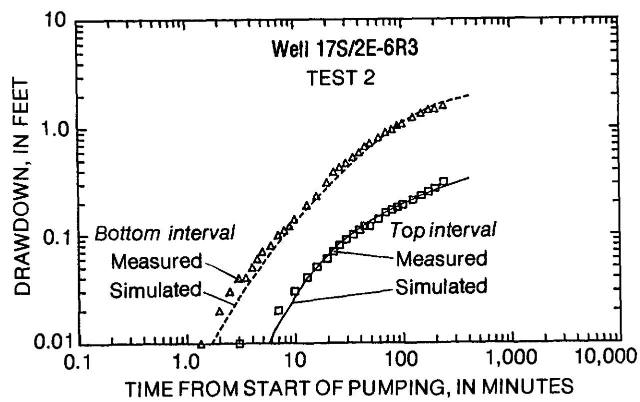


Figure 31. Measured and simulated drawdowns at site 6R2 in top and bottom intervals of observation well 17S/2E-6R3, test 2.

times greater than the vertical hydraulic conductivity. The unweathered bedrock, on the other hand, appears to be anisotropic in the opposite sense of the transition zone. The ratio of horizontal to vertical hydraulic conductivity ranged from 1:67 to 1:3.

Site 6F9

Site 6F9 is in the west-central part of Lee Valley (fig. 1). Drillers' logs indicate that the regolith at this site extends from land surface to a depth of about 40 ft. The transition zone is estimated to extend from 40 to 80 ft. At the time of the aquifer tests (January 1988), depth to water was about 28 ft from land surface. Of the five sites at which hydraulic tests were done in this study, this is the only site in which the water table lies within the regolith. Consequently, the site is more heavily instrumented, with two wells and four piezometers. The configuration of the wells and piezometers at the site is shown in figure 32. Well 17S/2E-6F9 is a privately owned, unused water well. Well 17S/2E-6F14 and the four piezometers, 17S/2E-6F10, -6F11, -6F12, and -6F13, were installed by the USGS for this study. Piezometers 17S/2E-6F10 and 17S/2E-6F13 are completed in the regolith, and piezometers 17S/2E-6F11 and 17S/2E-6F12 are completed in the transition zone. Construction data for wells and piezometers are given in table 3.

Three packers were installed in well 17S/2E-6F14 to create four observation intervals: 57 to 85 ft (top interval), 87 to 109 ft (upper-middle interval), 111 to 141 ft (lower-middle interval), and 143 to 189 ft (bottom interval). The top interval allows monitoring primarily of the transition zone, and the lower three intervals allow monitoring at different depth ranges within the unweathered bedrock.

Characteristic response times of the piezometers and the observation intervals all are small. Thus, effects of delayed response are not considered in the analysis of the tests.

Two aquifer tests were done at this site. The first test involved pumping from the transition zone, and

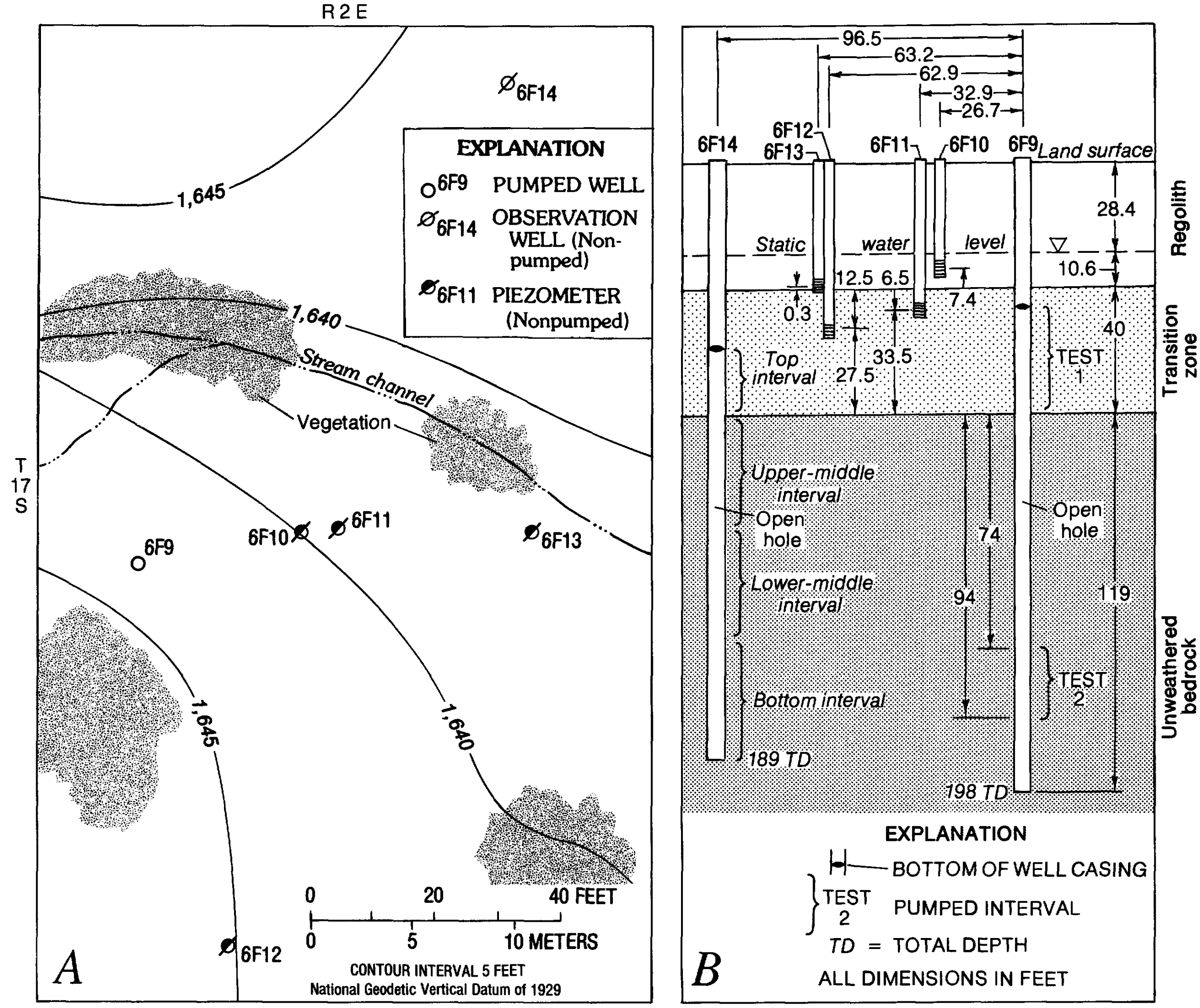


Figure 32. Configuration of wells and piezometers at site 6F9 (T. 17S., R. 2E.) for test 1 and test 2. *A*, Plan view. *B*, Diagrammatic section.

the second from the unweathered bedrock. In test 1, a packer was installed in well 17S/2E-6F9 at a depth of 83 ft. The interval above the packer was pumped at a constant rate of 2.3 gal/min for 20 hours. The relatively long duration was needed to create a measurable drawdown in the regolith while pumping from the transition zone. Measured heads are shown in figure 33. Note that the regolith piezometers (17S/2E-6F10 and 17S/2E-6F13) showed relatively small drawdowns, whereas the transition-zone piezometers (17S/2E-6F11 and 17S/2E-6F12) showed larger drawdowns. No discernible drawdown was measured in the four observation intervals in well 17S/2E-6F14.

In test 2, the packer in well 17S/2E-6F9 was lowered to 150 ft, and water was pumped from the interval below the packer at a constant rate of 4.0 gal/min for 6.5 hours. Measured heads are shown in figure 34. Prior to the start of pumping, a nearby domestic well was turned on for a brief period, creating a disturbance in the pretest water level in well 17S/2E-6F9. For this well, the measured drawdowns were corrected by graphically extrapolating the recovery data from the pretest disturbance. In the piezometers, the effect of the disturbance is almost imperceptible and therefore is neglected. Also, note that the regolith piezometers (17S/2E-6F10 and 17S/2E-6F13, fig. 34*B*) showed no drawdown during the test, and the lower three observation intervals in well 17S/2E-6F14 showed virtually identical heads. Therefore, only the head of the bottom interval is shown along with the head of the top interval (fig. 34*C*).

An additional test was attempted in well 17S/2E-6F9 by pumping from a packed-off interval from 86 to 126 ft. This interval is of interest because caliper and acoustic-televiewer logs seem to indicate the presence of several large fractures. However, the interval could not yield enough water to sustain the lowest pumping rate (about 1.2 gal/min) of the equipment. Therefore, this interval could not be tested.

In using the two-zone model for analysis of test 1, it is assumed that the unweathered bedrock is impermeable. In the test, pumping from the transition zone induces flow from both the regolith and the weathered bedrock to the transition zone. Neglecting the unweathered bedrock has the effect of neglecting the volume of water contributed from the unweathered bedrock. This volume is expected to be much smaller than that from the regolith because water from the regolith can be released by lowering of the water table, whereas water from the unweathered bedrock is released from compressive storage. Thus, the simplification seems reasonable. The physical dimensions (see the section "Analysis for Two- and Three-Zone Tests") used in the model for the four piezometers are b_1 = 10.6 ft and b_2 = 40 ft; r = 26.7 ft and z = 7.4 ft for 17S/2E-6F10; r = 32.9 ft and z = –6.5 ft for 17S/2E-6F11; r = 62.9 ft and z = –12.5 ft for 17S/2E-6F12; and r = 63.2 ft and z = 0.3 ft for 17S/2E-6F13.

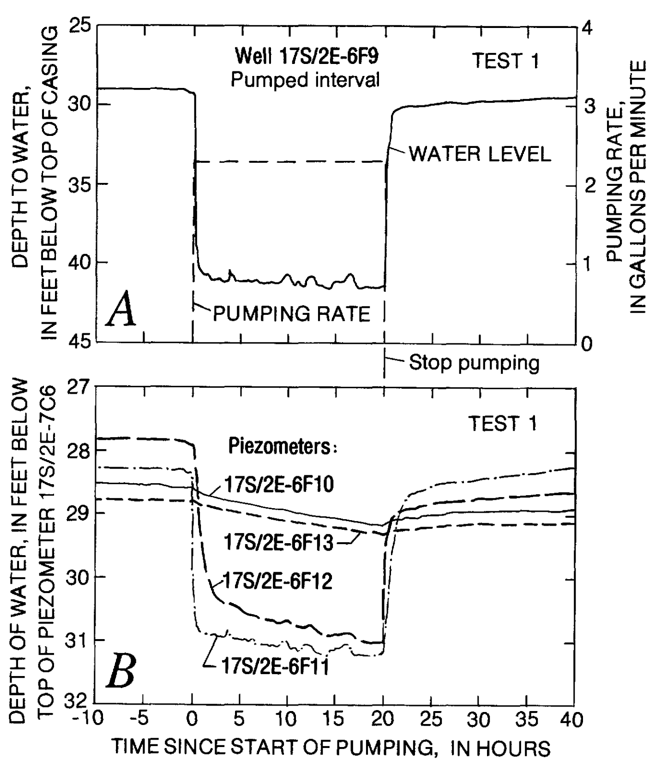


Figure 33. Measured water level at site 6F9 in pumped interval of well 17S/2E-6F9, test 1 (*A*); and in piezometers 17S/2E-6F10, 17S/2E-6F11, 17S/2E-6F12, and 17S/2E-6F13, test 1 (*B*).

Aquifer-property values obtained from test 1 are shown in table 4. The drawdown data are analyzed in pairs—piezometers 17S/2E-6F10 and 17S/2E-6F11 form a regolith/transition zone data pair at a radial distance of about 30 ft, and piezometers 17S/2E-6F12 and 17S/2E-6F13 form another pair at a radial distance of about 60 ft. For site 6F9, two values are given in table 4 for each aquifer property because the fitting was done separately for each data pair. The first and third lines under the heading "6F9, Test 1," give combined results from analysis of data from piezometers 17S/2E-6F10 and 17S/2E-6F11, and the

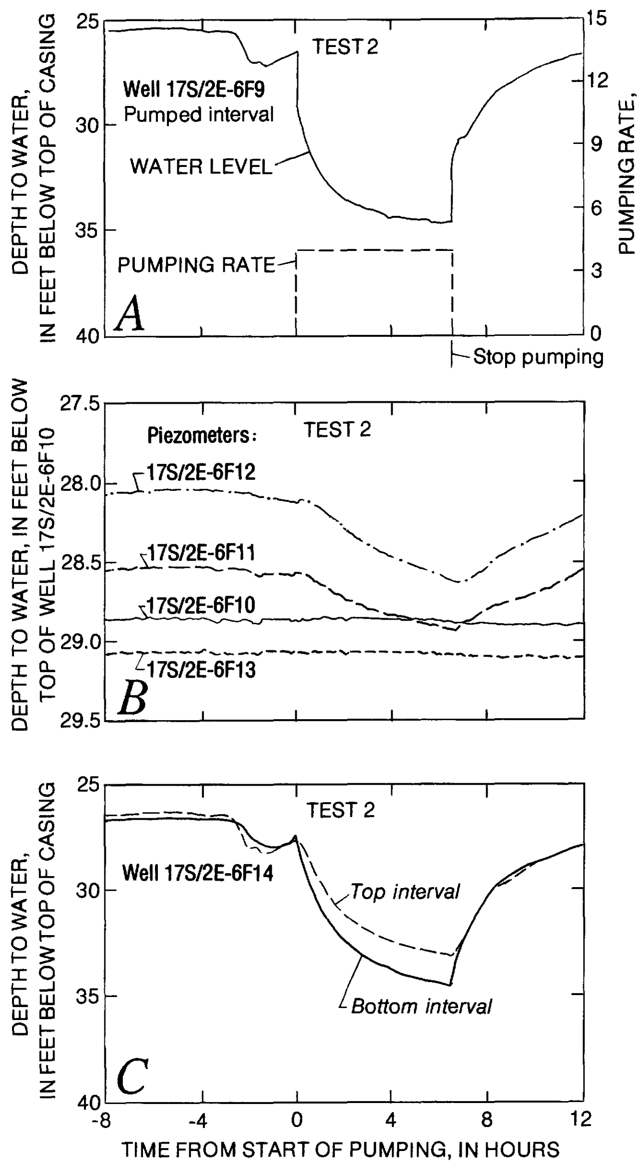


Figure 34. Measured water level at site 6F9 in pumped interval of well 17S/2E-6F9, test 2 (*A*); in piezometers 17S/2E-6F10, 17S/2E-6F11, 17S/2E-6F12, and 17S/2E-6F13, test 2 (*B*); and in top and bottom intervals of observation well 17S/2E-6F14, test 2 (*C*).

second and fourth lines give combined results from analysis of data from piezometers 17S/2E-6F12 and 17S/2E-6F13. The test was not sensitive to specific storage of the regolith, so a value of 1×10^{-5}/ft was assumed for the purpose of test analysis. The simulated and measured drawdowns are shown in figure 35. Except for early-time drawdown in the regolith, the simulated drawdowns match the measured drawdowns well. The similarity between the aquifer-property values determined from the data for the two pairs suggests that the two-zone model adequately represents the aquifer at the test site.

In analyzing test 2, the upper zone of the two-zone model is used to represent the transition zone, and the lower zone represents the unweathered bedrock. The regolith is not accounted for explicitly. Because of the presence of the water table, the regolith has a much larger storage capacity than the transition zone or the unweathered bedrock. Consequently, the drawdown in the regolith should be very small and can be considered negligible. This assumption is supported by the test data, which showed no measurable drawdowns in the two regolith piezometers, 17S/2E-6F10 and 17S/2E-6F13. For test analysis, the two-zone model is modified slightly so that the water-table boundary of the upper zone is replaced by a boundary of zero drawdown. This is achieved by assigning a larger value to S for the upper zone in the model. The physical parameters used in the model (see the section "Analysis for Two- and Three-Zone Tests") are b_1 = 40 ft, b_2 = 117 ft, d_2 = 74 ft, and L_2 = 94 ft; r = 32.9 ft and z = 33.5 ft for piezometer 17S/2E-6F11; r = 62.9 ft and z = 27.5 ft for piezometer 17S/2E-6F12; r = 96.5, z_1 = 25 ft, and z_2 = 0 ft for the top observation interval in well 17S/2E-6F14; and r = 96.5, z_1 = −61 ft, and z_2 = −107 ft for the bottom observation interval in well 17S/2E-6F14.

The aquifer-property values obtained from analysis of drawdowns measured in the two transition-zone piezometers, 17S/2E-6F11 and 17S/2E-6F12, are shown in table 4 (under the heading "6F9, Test 2," see first line of "transition zone" entries and first line of "unweathered bedrock" entries). The two sets of drawdown data were analyzed jointly. The measured and simulated drawdowns are shown in figure 36. A certain amount of subjective judgment was required to select the aquifer-property values because no one set of values produced drawdown curves that match

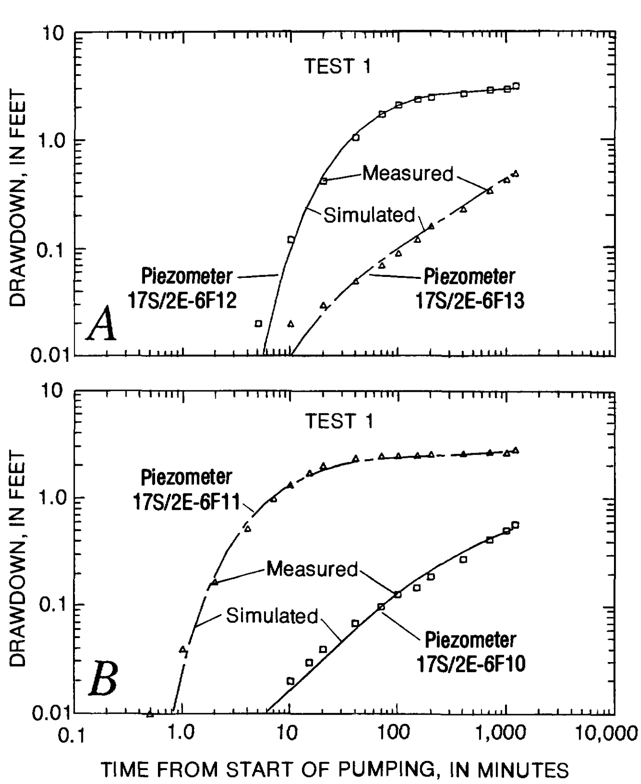


Figure 35. Measured and simulated drawdowns at site 6F9 in piezometers 17S/2E-6F12 and 17S/2E-6F13, test 1 (*A*); and in piezometers 17S/2E-6F10 and 17S/2E-6F11, test 1 (*B*).

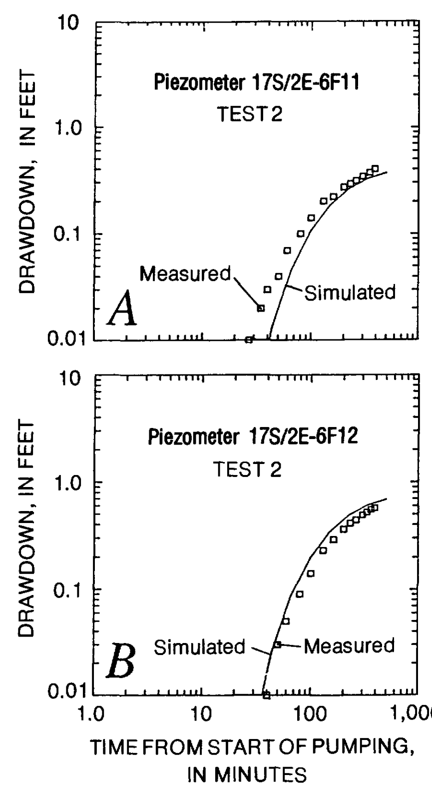


Figure 36. Measured and simulated drawdowns at site 6F9 in piezometers 17S/2E-6F11 (*A*) and 17S/2E-6F12 (*B*), test 2.

both data sets. However, the matches shown in figure 36 (taking into account the simplicity of the model) are considered adequate.

The aquifer-property values obtained from analyzing drawdowns observed in the top and bottom intervals in well 17S/2E-6F14 are given in table 4 (under the heading "6F9, Test 2," see second line of "transition zone" entries and second line of "unweathered bedrock" entries). The measured and simulated drawdowns are shown in figure 37. We were not able to produce drawdown curves that fit all the data, and thus a fit to the late-time data was selected. The lack of fit suggests that the current two-zone model may not be adequate for representing the aquifer at this site. The aquifer may be sufficiently heterogeneous that the homogeneity assumption for each zone may not be appropriate. Therefore, the results for site 6F9, test 2, given in table 4 should be considered to be preliminary.

The results of the tests from two different pumping intervals indicate that the horizontal hydraulic conductivity and the specific storage decrease from the regolith downward through the transition zone to the unweathered bedrock. The transition-zone property values obtained from the two tests are consistent (table 4). The regolith and the transition zone show similar degrees of anisotropy, with the ratio of horizontal to vertical hydraulic conductivity ranging from about 3:1 to 13:1. Conflicting results are obtained for the anisotropy in the unweathered bedrock. For example, one set of results from test 2 (table 4) gives an anisotropy ratio of 6:1, but a second set of results (table 4) gives a ratio of 1:50—in the opposite sense of the first ratio. In view of the poor fit between simulated and measured drawdowns in the analysis, the conflicting results are not surprising. Resolution

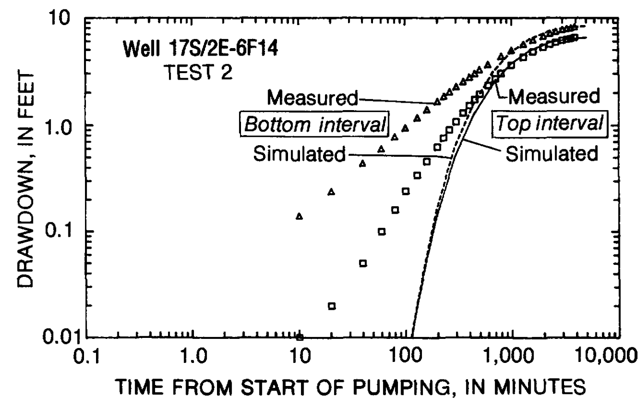


Figure 37. Measured and simulated drawdowns at site 6F9 in top and bottom intervals of observation well 17S/2E-6F14, test 2.

of this conflict would require further study to characterize the site in greater detail so that an improved aquifer model could be formulated.

Summary of Aquifer-Property Values

A summary of the aquifer-property values determined from aquifer tests done at five selected sites is given in table 5. For each pumping interval that was tested, the table gives the horizontal hydraulic conductivity (K_r), the ratio of horizontal to vertical hydraulic conductivity (K_r:K_v), the specific storage (S_s), and (where appropriate) the specific yield (S_y). For tests in which more than one pumping interval was used at a particular site, the values in the table represent the average of the results obtained.

Hydraulic Conductivity

Saturated regolith may occur in disconnected patches throughout Lee Valley. At four of the five aquifer-test sites, the regolith was either absent or unsaturated. However, those four sites (7C1, 5N5, 31N4, 6R2) are near the edge of the valley floor. In the valley center, the saturated regolith may form a more extensive layer. On the basis of one test, at site 6F9, the horizontal hydraulic conductivity of the regolith was determined to be 1×10^{-4} ft/s. The vertical hydraulic conductivity is about one-fifth of the horizontal hydraulic conductivity.

Perhaps the most important feature shown by aquifer-property data in table 5 is the similarity between the horizontal hydraulic conductivities of the transition zone. For the five selected sites, the values ranged from 3×10^{-6} to 4×10^{-5} ft/s. This range is quite small, in view of the high degree of heterogeneity that is characteristic of fractured-rock aquifers. The test results indicate that the horizontal hydraulic conductivity of the transition zone is relatively uniform from one part of Lee Valley to another. Thus, the transition zone is expected to provide the most consistent well yields in Lee Valley.

The aquifer-property values for the transition zone (table 5) also suggest a relation between anisotropy and confinement. Of the four sites at which the transition zone occurs as the topmost saturated layer (that is, the regolith is either absent or unsaturated), two seem to be locally confined and the other two show water-table conditions. Note that the locally confined transition-zone sites have higher horizontal than vertical hydraulic conductivities, and that the

Table 5. Summary of aquifer-property values determined at five test sites

Site No.	Horizontal hydraulic conductivity (foot per second)	Ratio of horizontal and vertical hydraulic conductivity	Specific storage (per foot)	Specific yield (unitless)
		Regolith		
7C1[1]				
5N5[2]				
31N4[2]				
6R2[1]				
6F9	1×10^{-4}	5:1	([3])	1×10^{-2}
		Transition zone		
7C1	2.8×10^{-5}	37:1	5×10^{-6}	([4])
5N5	1.8×10^{-5}	1:1.1	3.0×10^{-5}	2×10^{-2}
31N4	1.3×10^{-5}	1:15	[5]1×10^{-6}	6×10^{-3}
6R2	4×10^{-5}	3:1	4×10^{-5}	([4])
6F9	3×10^{-6}	8:1	2×10^{-6}	([3])
		Unweathered bedrock		
7C1[6]				
5N5[6]				
31N4[6]				
6R2	7×10^{-7}	1:35	1×10^{-7}	([3])
6F9	2×10^{-7}	6:1 to 1:50	2×10^{-7}	([3])

[1]Regolith absent. [2]Regolith unsaturated. [3]Not tested. [4]Locally confined. [5]Unreliable result. [6]Nonproductive.

reverse is true for the two transition-zone sites that show water-table conditions. This apparent relation may be due to a greater degree of vertical fracturing at the sites with water tables. The lack of vertical fractures would cause lower vertical hydraulic conductivity, resulting in confined conditions.

The hydraulic conductivity of the unweathered bedrock (table 5) seems to be highly variable. At three of the five test sites, the unweathered bedrock did not yield enough water to sustain the lowest pumping rate of the equipment; thus, a test to determine hydraulic conductivity could not be done. At the remaining two sites, the horizontal hydraulic conductivities ranged from 2×10^{-7} to 7×10^{-7} ft/s. These values are about 1.5 to 2 orders of magnitude lower than those of the transition zone. However, the study did not include testing of areas near lineaments that have been mapped in Lee Valley. Drillers' logs indicate that the hydraulic conductivities along these lineaments may be significantly higher than in areas away from the lineaments.

The vertical hydraulic conductivity of the unweathered bedrock also shows large variations (table 5). There are some indications that the vertical hydraulic conductivity can be more than an order of magnitude greater than the horizontal (see results for site 6R2). If this is characteristic of the unweathered bedrock, then it implies the presence of open, vertical fractures to depths of 100 to 200 ft. Pumping from the unweathered bedrock would induce significant vertical flow from the overlying transition zone. The implication is that although a well may be pumped from the unweathered bedrock, a significant part of the pumped water may be derived from the transition zone and possibly from the saturated regolith. This conjecture, however, has not been substantiated in this study.

Specific Storage and Specific Yield

The specific storage of the regolith could not be determined accurately because the test was not sensitive to it. On the basis of one test, the specific yield of the regolith was 1×10^{-2}. This value is not unreasonable in view of the relatively high clay content of the regolith.

The specific storage of the transition zone ranged from 1×10^{-6}/ft to 4×10^{-5}/ft. At the two sites where the water table was within the transition zone, the specific yields were 6×10^{-3} and 2×10^{-2}. These values are within the range found for near-surface fractured crystalline rocks.

At the two sites where the unweathered bedrock is productive, the specific storage ranged from 1×10^{-7}/ft to 2×10^{-7}/ft. This is 1 to 2 orders of magnitude less than the specific storage of the transition zone. The results suggest that in regions away from lineaments, the unweathered bedrock, in itself, may not yield a significant amount of water.

GROUND-WATER RECHARGE AND DISCHARGE

Recharge to the fractured-rock aquifer in Lee Valley varies from year to year and is dependent on several factors—some varying with time and some constant. These factors include the quantity and timing of rainfall; the ability of the aquifer to take water into, or release water from, storage (the storage coefficient); and the quantities of ground water pumped (pumpage), discharged by evapotranspiration (a combined product of climatic variables and vegetation), and discharged from the aquifer to streams.

The relation used to calculate ground-water recharge is

Ground-water recharge = change in ground-water storage
+ ground-water pumpage
+ evapotranspiration from the saturated zone
+ stream base flow

Each of these factors is discussed below. Ground-water recharge was calculated for water year 1988 (October 1, 1987, through September 30, 1988).

Change in Storage

The change during water year 1988 in the quantity of water in storage in the aquifer was calculated by first determining the difference in water levels between October 1, 1987, and September 30, 1988. The water-level rises and declines were mapped and grouped using a weighted average to allow area measurements of each change category (fig. 38). Each area of change was multiplied by the amount of change and then summed. This volume of aquifer was multiplied by the low and high values of the range of specific yield (1×10^{-2} and 2×10^{-2}) determined from the aquifer tests discussed in this report. Thus, the calculated volume of change in storage during water year 1988 ranges from –5.5 to –11.0 acre-ft, depending on the value of specific yield used.

The amount of change in storage fluctuates from year to year, depending primarily on rainfall. For comparison with water year 1988, the change in storage during the relatively dry water year 1984 ranges from –45.3 to –90.6 acre-ft. For the moderately wet 1987 water year, the change in storage ranges from –16.5 to –33.0 acre-ft.

Pumpage

Ground-water pumpage for domestic use and irrigation during water year 1988 was estimated for three households (from pumpage records) and for a fourth property (from calibrated electrical usage for a pump). The values ranged from 0.21 to 0.31 acre-ft, and the average use per household was 0.23 acre-ft (205 gal/d). Therefore, the total pumpage during water year 1988 for 80 households is estimated to be 18.4 acre-ft.

The average ground-water use by the monitored households for water year 1988 was less than the average (0.52 acre-ft) for 1983–88. The latter value yields an average annual ground-water pumpage of 42 acre-ft. The difference may be a result of the relatively high rainfall of 20.99 in. in rain year 1987–88 (1983–88 average was 15.87 in.) and (or) a change in the amount of water used for irrigation. In the recharge calculation, 18.4 acre-ft is used as the estimate of minimum annual ground-water pumpage, and 42 acre-ft is used as the estimate of maximum pumpage.

Evapotranspiration from the Saturated Zone

The calculation of recharge requires an estimate of evapotranspiration from the saturated zone. This is a quantity that is difficult to determine. In this study, two methods were used to estimate ground-water evapotranspiration: (1) analysis of daytime decrease in stream base flow; and (2) approximation based on calculated potential evapotranspiration. Together, the two methods provided a range of values that could be used in the recharge calculations. For both methods, the assumption was made that all ground-water evapotranspiration from the saturated zone may be attributed to the holly-leaf oaks (*Quercus agrifolia*) growing along Jamul Creek and in the valley flat (fig. 2).

Base-Flow Analysis

Hourly measurements of base flow in Jamul Creek were made during two 24-hour periods, in winter and in early summer 1984, using a flume near well 17S/2E-7C1 (see the "Stream Base Flow" section). The purpose of the measurements was to determine the volume-per-day decrease in base flow presumed to be caused by evapotranspiration by oak trees near the flume. Assuming that evapotranspiration does not occur (or is minimal) at night, the nighttime base flow can be taken to be the ground-water discharge into the creek. During daytime, however, a part of this ground-water discharge is taken up by transpiration by oak trees along the creek; therefore, base flow in the creek is decreased. The rate of evapotranspiration peaks in the afternoon: data from an early summer 24-hour measurement period show that afternoon base flow decreases by as much as 60 percent in comparison with the nighttime base flow. The area of oak trees causing the decrease in streamflow was assumed to be small because there was virtually no lag between the hottest time of day and the maximum decrease in flow.

Evapotranspiration from the saturated zone for the entire basin was calculated by multiplying the localized volume of ground-water evapotranspiration (that is, the decrease in base flow measured near the flume) by the ratio of total area of trees to the area of trees near the flume. The amount of ground water transpired, calculated using the smaller (and probably more appropriate) unit area of trees near the flume, is 135 acre-ft, in comparison with 26.7 acre-ft calculated using a larger unit area. The latter value is the estimate of minimum ground-water evapotranspiration.

Potential-Evapotranspiration Analysis

Potential evapotranspiration, which is calculated from temperature, solar-radiation, and wind measurements, can be used as an estimation of evapotranspiration (Harold Weaver, U.S. Geological Survey, oral commun., 1989). The necessary data were not available for Lee Valley, but calculated potential-evapotranspiration data have been compiled for Temecula, 50 mi north of the study area, as part of the California Irrigation Management Information System (CIMIS).

CIMIS daily potential-evapotranspiration estimates for Temecula were summed for October 1, 1987, to September 30, 1988. When the total, 5.1 ft, is multiplied by the total area of oak trees in Lee Valley, an approximate estimate of maximum ground-water evapotranspiration of 202 acre-ft is obtained. This value probably is an overestimate, for two reasons. First, because the altitude at Temecula (1,000 ft above sea level) is about 600 ft lower than the altitude of Lee Valley, temperatures at Temecula probably are higher. Second, potential evapotranspiration approximates the total transpiration by oak trees from both the unsaturated and the saturated zones. However, during most of the year (except for December–February, when soil moisture is high), the holly-leaf oaks likely draw most of their water from the saturated zone, and thus this part of the overestimation probably is not great. The December–February period also coincides with the time of year when the oaks are not transpiring as much water.

Stream Base Flow

Stream base flow was measured using a small flume at the discharge end of the basin (near well 17S/2E-7C1). A calibrated bucket and a stopwatch were used to make rate measurements (an average of about 2 days per week) during the part of water year 1988 in which continuous streamflow occurred (October 12, 1987, to June 17, 1988). Data were extrapolated for days on which measurements were not taken. For days on which stormflow occurred, an average 24-hour flow rate was selected (using, in some cases, USGS recording-gage data) that reflected increased base flow but excluded stormflow-runoff peaks.

The total base flow for the sample year was estimated to be 8.0 acre-ft. Base flow decreases during daylight hours because of the effect of ground-water evapotranspiration. To obtain a 24-hour volume of base flow, which then could be summed as part of a yearly volume, each base-flow measurement was corrected for the time of day that the measurement was made.

Calculated Recharge

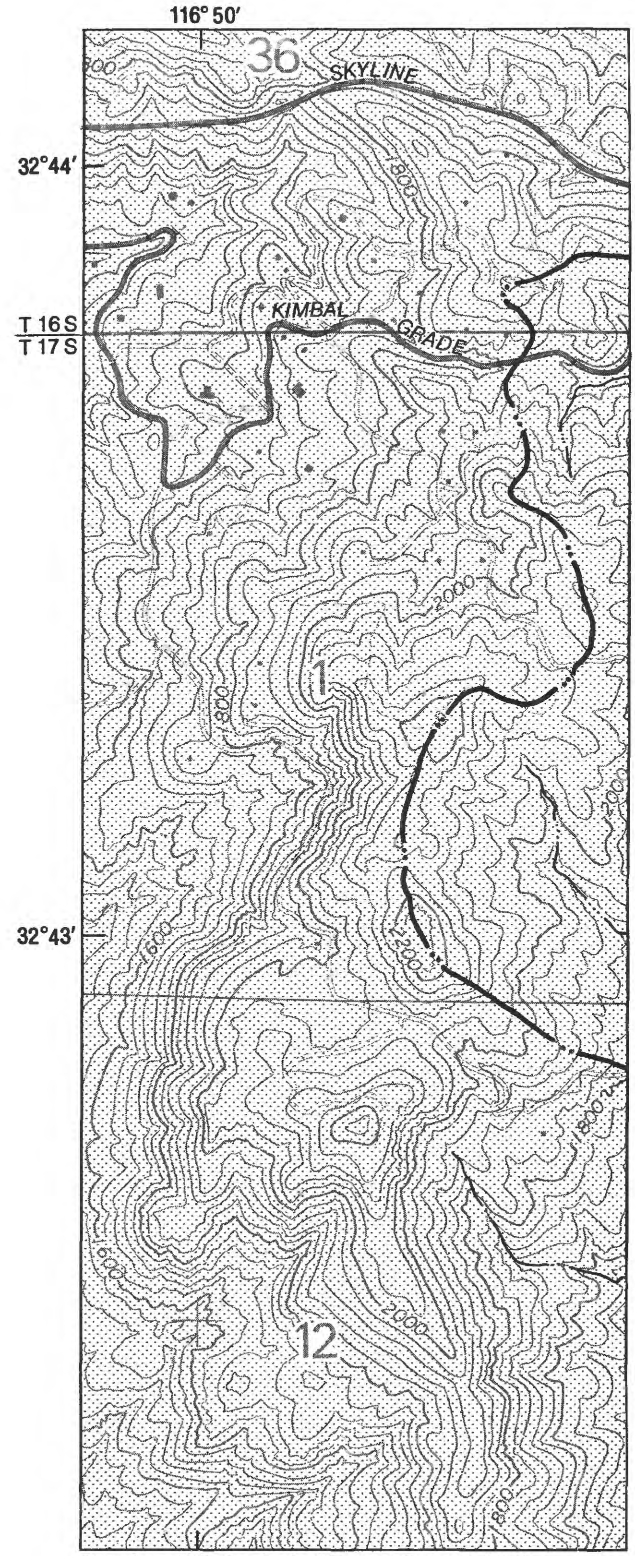
Separate calculations of net recharge to ground water were made using maximum, minimum, and "likely" values of change in storage, ground-water pumpage, evapotranspiration from ground water, and stream base flow. A summary of these values, and resulting calculated maximum,

EXPLANATION

+4 ZONE OF AVERAGED WATER-LEVEL CHANGE, IN FEET—Boundary is approximate. Minus (-) indicates water-level decline; plus (+) indicates water-level rise

SURFACE-WATER BASIN BOUNDARY

•4 WELL—Number indicates water-level change, in feet



Base from U.S. Geological Survey, Dulzura, 1972

Figure 38. Change in water level, October 1987–September 1988.

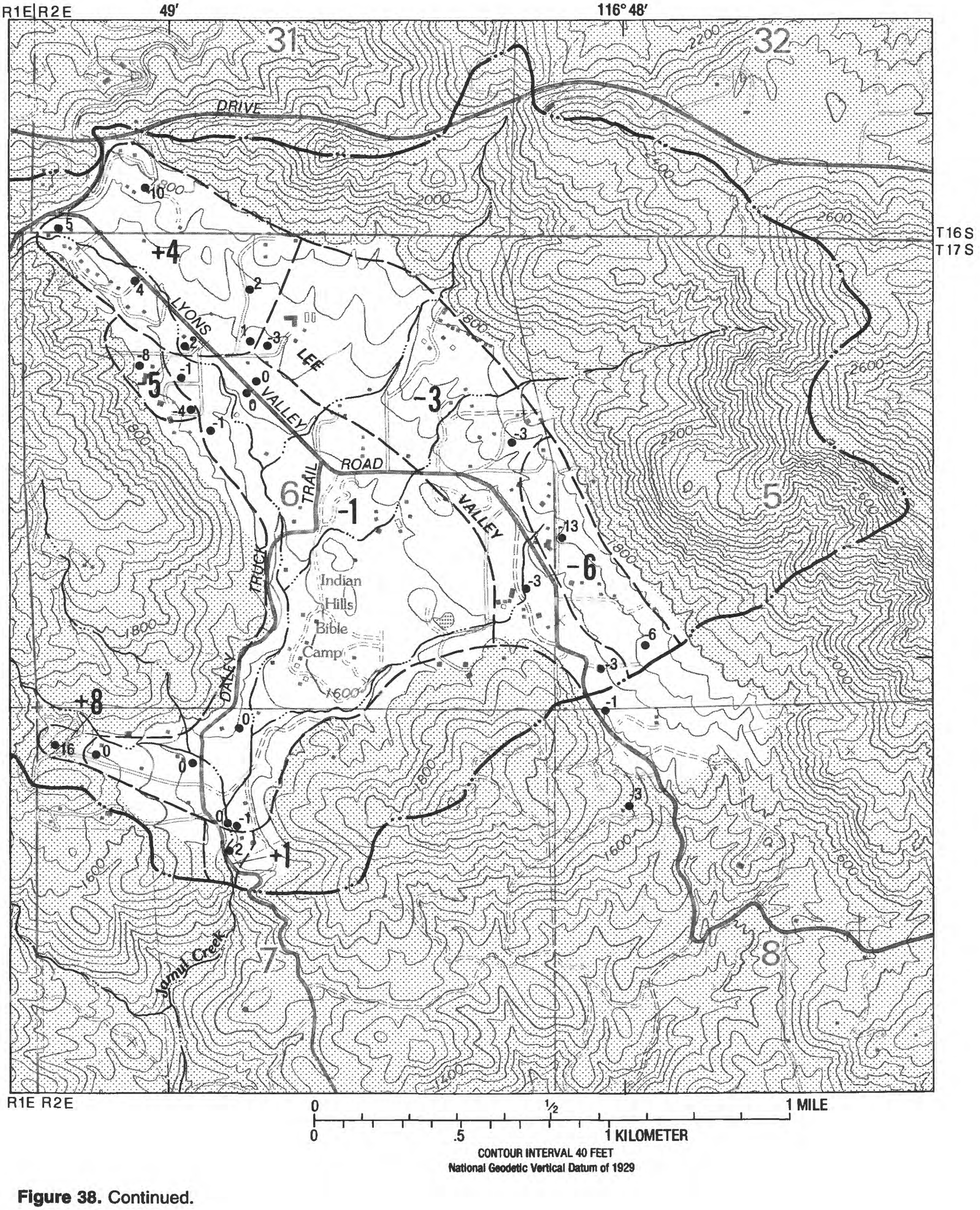


Figure 38. Continued.

Table 6. Summary of calculation of ground-water recharge for water year 1988

[Values in acre-feet. S_y, specific yield]

Component	Maximum value	Minimum value	Likely value	Remarks
Change in storage	-5.5			Maximum value based on $S_y = 1\times10^{-2}$
		-11.0		Minimum value based on $S_y = 2\times10^{-2}$
			-8	An average value
Ground-water pumpage	42			Average of extrapolated measurements for 1983-88
		18.4		Extrapolated from measured values for water year 1988
			25	An average, weighted toward data from the sample period
Ground-water evapotranspiration (GWET)	202			Potential evapotranspiration for the sample period (an overestimation of GWET)
		26.7		Based on assumption of a large area of oak trees causing daytime streamflow decline at the flume
			135	Based on assumption of trees within 100 feet of flume being cause of flow decline
Stream base flow	9.2			Based on continuous-recording gage at the flume
		7.7		Based on data from a downstream flume
			8	Based on correction of individual measurements to a 24-hour average base flow
Net recharge to ground-water system	247.7	41.8	160	

minimum, and likely values of net recharge, are given in table 6.

The likely value of recharge to ground water (rounded to nearest acre-foot) is 160 acre-ft. In computing this value, the assumption was made that no significant ground-water underflow discharge from the basin occurred. This assumption is based, in part, on the following observations: Bedrock crops out at the discharge end of the basin; ground-water discharge to streamflow does not increase noticeably at the second flume, which is 300 ft past the sharp drop of the streambed out of the basin (fig. 1); and springs were not found downhill from the discharge end of the basin. If there was unaccounted underflow during the sample period, then the calculated recharge is underestimated.

When compared with the total rainfall of 2,330 acre-ft for the sample period (water year 1988), the ground-water recharge value of 160 acre-ft indicates that only 7 percent of the rainfall in Lee Valley enters the ground-water flow system. The remaining 93 percent either is evaporated, is stored in the unsaturated zone and later lost by evaporation, or is discharged to streams by means of overland flow.

Although recharge has been analyzed for only 1 year and long-term average recharge is not known, the findings do have significance with regard to management of ground-water resources. Rough calculations—using assumptions of a 1-mi^2 aquifer (regolith and transition zone) 100 ft in thickness with a porosity of 1 percent—yield a value of about 640 acre-ft of ground water in storage in Lee Valley. Therefore, storage may be about 15 times the average quantity of recent (1983–88) annual use (about 42 acre-ft). If below-average recharge were to occur over a period of several years, especially if combined with increased use of ground water, a significant depletion of ground water in storage could result within a decade. The rate of depletion would be lessened somewhat when the water table declines below the depth from which the holly-leaf oaks withdraw ground water by transpiration.

Recharge volumes and water levels in Lee Valley during the 1983–88 period of monitoring fluctuated and were, at least in part, related to yearly rainfall (fig. 10). The water levels were highest in 1983, when water was within 10 ft of land surface in much of the valley; water levels were high, also, in 1985–86. Both periods had greater than average rainfall, as shown in figures 10 and 39. The long-term record (1910–88) of rainfall for Barrett Dam, which is about 9 mi east of Lee Valley and at the same altitude as the valley floor, and the 1985–88 rainfall record for Lee Valley are shown in figure 39. Correlation for the overlapping years is good. The variation of yearly rainfall at Barrett Dam, therefore, is an indication of the variability of rainfall, and thus of recharge, that occurs in Lee Valley. Ground water in the fractured-rock aquifer of the Peninsular Ranges acts as a buffer during drought periods, and recharge generally allows water levels to recover during years of greater-than-average rainfall. Therefore, fluctuation in recharge, and patterns of drought, may have a greater effect on the fractured-rock aquifer in Lee Valley than does ground-water pumpage at current levels of withdrawal.

SUMMARY AND CONCLUSIONS

Test drilling and geophysical logging of test wells indicate that the fractured-rock aquifer underlying Lee Valley can be divided into three zones: (1) regolith, consisting primarily of weathered granodiorite and fine-grained gabbro, generally 20 to 60 ft thick; (2) a transition zone of highly fractured, partly weathered rock, which generally extends to depths of less than 100 ft; and (3) unweathered bedrock that is fractured to various degrees. The water table generally is 10–35 ft below land surface in most of the flat central part of the valley floor, and as much as 70 ft below land surface at the gently sloping edges of the valley floor. The thickness of each of the three zones is variable, and saturated regolith may be limited to isolated patches separated by regions where the water table lies below the regolith.

Fractures mapped in boreholes have diverse orientations. The orientations for the two highest concentrations of fractures are strike N. 35° W., dip 50° NE.; and strike N. 18° E., dip 50° W. However, wide dispersion (rather than concentration) of orientations probably is more characteristic of the fractures in Lee Valley. Lineaments mapped from aerial photographs generally trend northwest and northeast. The northwest-trending lineaments are subparallel to the trend of Lee Valley; both may be related to a major structural trend in the region. Comparison between fractures and lineaments is difficult because of the difference in scale at which they are observed: fractures are mapped in boreholes, and lineaments are mapped on aerial photographs. Although

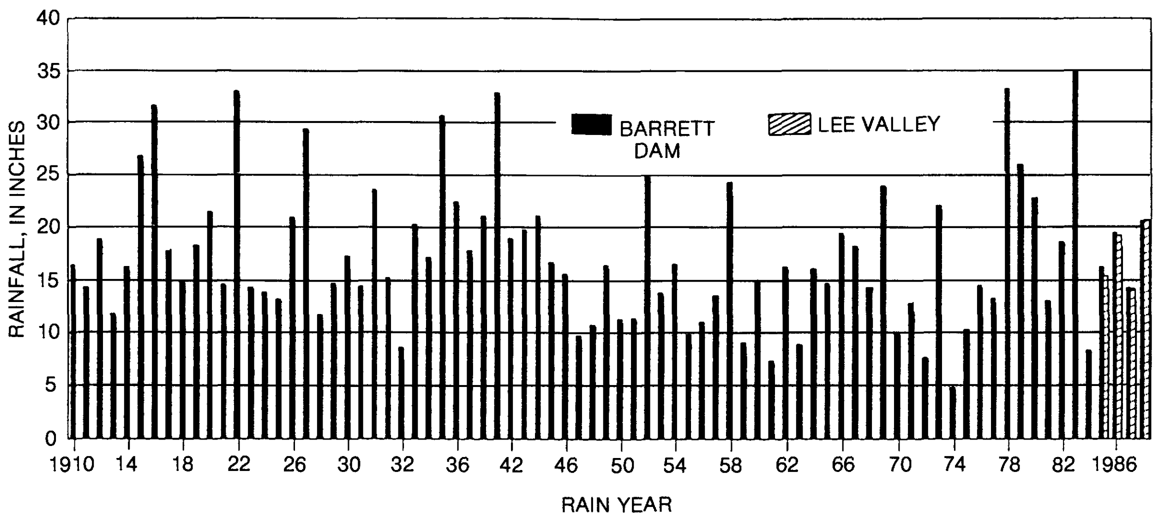


Figure 39. Annual rainfall at Barrett Dam, 1910–88, and at Lee Valley, 1985–88. (Barrett Dam is 1,623 ft above sea level and is 9 mi east of Lee Valley. Rain year is July 1 through June 30.)

the strikes of the fracture concentrations are similar to the trends of the lineaments (northeast and northwest), the relation between fractures and lineaments remains unclear.

The distribution and range of reported water yields of wells drilled in Lee Valley are characteristic of igneous-rock terranes. Most yields of wells are low (less than 15 gal/min), but a few high-yielding wells (greater than 100 gal/min) do exist. Areal distribution of high and low well yields does not indicate an apparent pattern. Two wells close to each other can have a large difference in yield. These differences result from the heterogeneous characteristics of fractured-rock aquifers. In areas where lineaments have been mapped in Lee Valley, all the high-yield wells are near (within 300 ft of) lineaments; however, not all wells that are drilled near lineaments are of high yield. Although lineaments may function as a helpful guide for locating well sites, drilling a well close to (or on) a lineament does not guarantee high yield.

Water levels in wells show seasonal and long-term responses to recharge. In general, the water level rises during the November-to-April rainy season, and declines during May to October. However, the amount of fluctuation is variable from year to year, depending on the rainfall. Water levels in shallow wells constructed in the regolith tend to respond more rapidly to recharge than do water levels in wells open to the transition zone and unweathered bedrock. The water-level fluctuations indicate that all three zones of the fractured-rock aquifer form a dynamic, interactive flow system. A long-term (multiyear) downward trend in water level indicates that the aquifer is extensively recharged only during periods of exceptionally high rainfall, whereupon the water level may rise by tens of feet in some parts of the valley. The water-level rise is followed by a gradual, long-term decline superimposed on the seasonal fluctuation. This trend can continue for a number of years, until the aquifer is again extensively recharged by exceptionally high precipitation.

Chemical analyses of water samples indicate that the age of water from different zones is not markedly different. Water from deep zones in the unweathered bedrock generally had higher ionic concentrations than water from the transition zone and regolith. Analysis of tritium concentrations in the sampled water indicates that it was recharged less than 25 years ago.

Hydraulic conductivities for the three aquifer zones were determined from aquifer tests done at five sites. For the regolith, the hydraulic conductivity, as determined from a test at the only test site at which saturated regolith was present, is 1×10^{-4} ft/s. The vertical hydraulic conductivity of the regolith is about one-fifth of the horizontal conductivity.

The values of horizontal hydraulic conductivity for the transition zone are similar at all five aquifer-test sites, ranging from 3×10^{-6} to 4×10^{-5} ft/s. The transition zone is expected to provide the most consistent yields of water to wells in Lee Valley. At two locally confined transition-zone sites, the horizontal hydraulic conductivity is higher than the vertical hydraulic conductivity; however, the reverse is true of the two transition-zone sites that show water-table (unconfined) conditions. This difference in conductivity ratios may be the result of a greater degree of vertical fracturing at sites where a water table is present.

Results of the aquifer test indicate that hydraulic conductivity of the unweathered bedrock is highly variable. At three of five test sites, the unweathered bedrock was nonproductive. At the remaining two test sites, the average values of horizontal hydraulic conductivity ranged from 2×10^{-7} to 7×10^{-7} ft/s, which is about 1 to 2 orders of magnitude lower than those of the transition zone. However, this study did not include testing of areas near lineaments that have been mapped in Lee Valley. Drillers' logs indicate that the values of hydraulic conductivity along these lineaments may be significantly higher than in areas away from the lineaments.

Storage characteristics also were determined from the aquifer tests. The specific yield of the regolith, on the basis of one test, was 1×10^{-2}. For the transition zone, specific storage ranged from 1×10^{-6}/ft to 4×10^{-5}/ft, and specific yield, where a water table is present, ranged from 6×10^{-3} to 2×10^{-2}. Specific storage of the unweathered bedrock was 2×10^{-7}/ft. This is 1 to 2 orders of magnitude less than the value of specific storage of the transition zone. The results suggest that in areas away from lineaments, the unweathered bedrock, in itself, may not yield a significant amount of water to wells.

Recharge to the fractured-rock aquifer in Lee Valley is variable from year to year. The largest component of the hydrologic budget is evapotranspiration—a component that is difficult to estimate. Recharge in water year 1988 was about 160 acre-ft, and pumpage was estimated to be 18.4 acre-ft. The average pumpage for 1983–88 was 42 acre-ft/yr. Water levels have recovered each year to varying degrees, depending primarily on the amount and timing of rainfall during the year. Water stored in the aquifer acts as a buffer during drought periods. The calculated change in volume of water in storage during water year 1988 ranges from −5.5 to −11.0 acre-ft, depending on the value of specific yield used in making the estimate. For comparison, the change in storage for the relatively dry period 1983–84 ranges from −45.3 to −90.6 acre-ft, and the change ranges from −16.5 to −33.0 acre-ft for the moderately wet water year 1987. If below-average recharge were to occur over a period of several years, especially in combination with increased use of ground water, a substantial depletion of water in storage could result within about a decade.

Additional work would be useful in attaining a better understanding of the geohydrology of Lee Valley, and of fractured-rock hydrology in general. Hydraulic tests in Lee Valley during this study were done in wells that were abandoned owing to their low yields, and thus the results of these tests may be biased toward lower estimates of the hydraulic properties of the aquifer. There is some evidence that wells drilled within 300 ft of a lineament are likely to yield larger quantities of water than wells drilled farther from lineaments. Therefore, hydraulic testing of wells near lineaments would provide hydraulic information about the more permeable parts of a fractured-rock aquifer. In addition, the monitoring of hydraulic heads separately in the different zones would allow investigation of the long-term interaction between the deeper unweathered bedrock and the shallower regolith and transition zones. Other information useful to a more complete understanding of the geohydrologic system of Lee Valley could be provided by geophysical studies to determine the areal extent and thickness of the regolith and transition zones. Finally, estimates of net ground-water recharge could be improved through use of energy-budget methods or porometry methods to estimate evapotranspiration.

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APPENDIX 1

APPENDIX 1: Analytical solution for drawdown at a well in which water is pumped from two zones in a water-table aquifer

Parameters for a two-zone model are illustrated in figure 15 (reproduced on the facing page). We first consider the case in which water is pumped from the upper zone. The equation of ground-water flow is

$$K_{r1}\left[\frac{1}{r}\frac{\partial}{\partial r}\left(r\frac{\partial s_1}{\partial r}\right)\right] + K_{z1}\frac{\partial^2 s_1}{\partial z^2} = S_{s1}\frac{\partial s_1}{\partial t}, \tag{A-1}$$

where s_1 is the drawdown in the upper zone. Similarly, for the lower zone, the equation of ground-water flow is

$$K_{r2}\left[\frac{1}{r}\frac{\partial}{\partial r}\left(r\frac{\partial s_2}{\partial r}\right)\right] + K_{z2}\frac{\partial^2 s_2}{\partial z^2} = S_{s2}\frac{\partial s_2}{\partial t}, \tag{A-2}$$

where s_2 is the drawdown in the lower zone. The initial condition is zero drawdown in the aquifer; that is,

$$s_1(r,z,0) = s_2(r,z,0) = 0. \tag{A-3}$$

The boundary condition at the water table is

$$-K_{z1}\frac{\partial s_1(r,b_1,t)}{\partial z} = S_{y1}\frac{\partial s_1(r,b_1,t)}{\partial t}, \tag{A-4}$$

where the assumption of small drawdown has been invoked. At the boundary between the two zones, continuity of drawdown and flux requires

$$s_1(r,0,t) = s_2(r,0,t), \tag{A-5}$$

and

$$K_{z1}\frac{\partial s_1(r,0,t)}{\partial z} = K_{z2}\frac{\partial s_2(r,0,t)}{\partial z}. \tag{A-6}$$

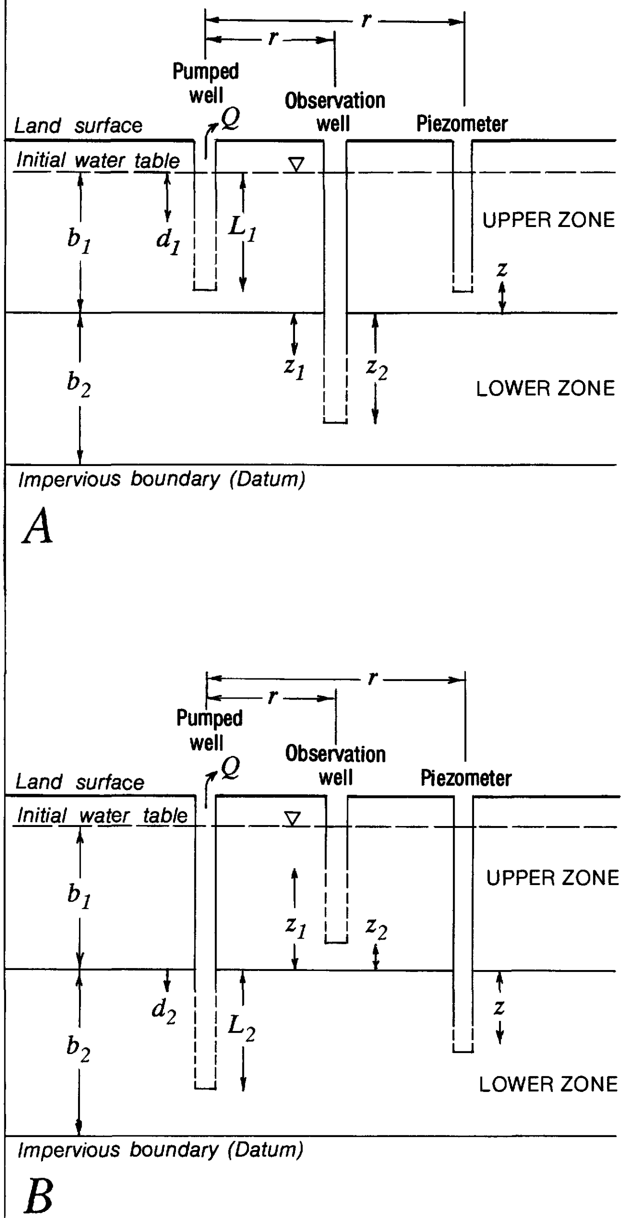
At the impervious base of the lower zone,

$$\frac{\partial S_2(r,-b_2,t)}{\partial z} = 0. \tag{A-7}$$

Along the axis of the pumped well, symmetry requires

$$\frac{\partial s_1(0,z,t)}{\partial r} = 0 \qquad b_1<z<b_1-d_1 \text{ and } b_1-L_1<z<0, \tag{A-8}$$

$$\frac{\partial s_2(0,z,t)}{\partial r} = 0 \qquad -b_2<z<0. \tag{A-9}$$



EXPLANATION

MODEL PARAMETERS

Q = Discharge

b_1 = Initial thickness of the upper zone (distance from initial position of water table to bottom of the upper zone)

b_2 = Thickness of the lower zone

d_1 = Distance from initial position of water table to top of intake interval of a well in which water is pumped from the upper zone

d_2 = Distance from top of lower zone to top of intake interval of a well in which water is pumped from the lower zone

L_1 = Distance from the initial position of water table to bottom of intake interval of a well in which water is pumped from the upper zone

L_2 = Distance from top of lower zone to bottom of intake interval of a well in which water is pumped from the lower zone

r = Distance from the pumped well to the piezometer or observation well

z = Distance from top of lower zone to center of open interval of the piezometer

z_1 = Distance from top of lower zone to top of open interval of the observation well

z_2 = Distance from top of lower zone to bottom of open interval of the observation well

Positive values of z, z_1, and z_2 indicate that the piezometer or observation well is completed in the upper zone, and negative values of z, z_1, and z_2 indicate that the piezometer or observation well is completed in the lower zone

Parameters for two-zone model. ***A*****, Water pumped from upper zone.** ***B*****, Water pumped from lower zone.**

Finally, along the intake interval of the pumped well,

$$\lim_{r\to 0} r\frac{\partial s_1}{\partial r} = \frac{Q}{2\pi(L_1-d_1)K_{r1}} \quad b_1-d_1<z<b_1-L_1. \tag{A-10}$$

The solution of the above problem can be obtained through the method of Laplace and Hankel transforms. After the Hankel inversion, the result is as follows. In the upper zone, above the intake interval of the pumped well ($b_1<z<b_1-d_1$):

$$\overline{s_1} = \frac{Q}{2\pi K_{z1}(L_1-d_1)}\int_{y=0}^{y=\infty}\frac{A_1}{\omega_1^2 pF}\, y\, J_0(yr)dy, \tag{A-11}$$

where $\overline{s_1}$ is the Laplace transform of s_1, p is the Laplace transform parameter, $J_0\,(yr)$ is the zeroth-order Bessel function of the first kind, and where

$$A_1 = [\omega_2K_{z2}\sinh(\omega_2b_2)\{\cosh[\omega_1(b_1-d_1)] - \cosh[\omega_1(b_1-L_1)]\} + \omega_1K_{z1}\cosh(\omega_2b_2)\{\sinh[\omega_1(b_1-d_1)]$$
$$- \sinh[\omega_1(b_1-d_1)]\}] \times \{\cosh[\omega_1(b_2-z)] + \frac{pS_{y1}}{\omega_1K_{z1}}\sinh[\omega_1(b_1-z)]\}, \tag{A-12}$$

$$F = \omega_2K_{z2}\cosh(\omega_1b_1)\sinh(\omega_2b_2) + \omega_1K_{z1}\sinh(\omega_1b_1)\cosh(\omega_2b_2)$$
$$+ \frac{pS_{y1}}{\omega_1K_{z1}}[\omega_2K_{z2}\sinh(\omega_1b_1)\sinh(\omega_2b_2) + \omega_1K_{z1}\cosh(\omega_1b_1)\cosh(\omega_2b_2)], \tag{A-13}$$

$$\omega_1 = [(y^2K_{r1} + pS_{s1})/K_{z1}]^{1/2}, \tag{A-14}$$

and

$$\omega_2 = [(y^2K_{r2} + pS_{s2})/K_{z2}]^{1/2}. \tag{A-15}$$

At the intake interval of the pumped well ($b_1-d_1<z<b_1-L_1$), the solution in the Laplace domain is

$$\overline{s_1} = \frac{Q}{2\pi K_{z1}(L_1-d_1)}\int_{y=0}^{y=\infty}\left(1-\frac{A_2}{F}\right)\frac{y\,J_0(yr)}{\omega_1^2 p}\,dy, \tag{A-16}$$

where

$$A_2 = U_1\left[\sinh(\omega_1d_1) + \frac{pS_{y1}}{\omega_1K_{z1}}\cosh(\omega_1d_1)\right] + U_2\{\cosh[\omega_1(b_1-z)] + \frac{pS_{y1}}{\omega_1K_{z1}}\sinh[\omega_1(b_1-z)]\}, \tag{A-17}$$

$$U_1 = \omega_2K_{z2}\sinh(\omega_1z)\sinh(\omega_2b_2) + \omega_1K_{z1}\cosh(\omega_1z)\cosh(\omega_2b_2), \tag{A-18}$$

and

$$U_2 = \omega_2K_{z2}\cosh[\omega_1(b_1-L_1)]\sinh(\omega_2b_2) + \omega_1K_{z1}\sinh[\omega_1(b_1-L_1)]\cosh(\omega_2b_2). \tag{A-19}$$

Below the intake interval in the upper zone ($0<z<b_1-L_1$), we have

$$\overline{s_1} = \frac{Q}{2\pi K_{z1}(L_1-d_1)} \int_{y=0}^{y=\infty} \frac{A_3}{\omega_1^2 pF}\, y\, J_0(yr)dy, \tag{A-20}$$

where

$$\begin{aligned} A_3 = {} & \{\sinh(\omega_1 L_1) - \sinh(\omega_1 d_1) + \frac{pS_{s1}}{\omega_1 K_{z1}}[\cosh(\omega_1 L_1) - \cosh(\omega_1 d_1)]\} \\ & \times [\omega_2 K_{z2}\sinh(\omega_1 z)\sinh(\omega_2 b_2) + \omega_1 K_{z1}\cosh(\omega_1 z)\cosh(\omega_2 b_2)]. \end{aligned} \tag{A-21}$$

For the lower zone, the result is

$$\overline{s_2} = \frac{Q}{2\pi K_{z1}(L_1-d_1)} \int_{y=0}^{y=\infty} \frac{A_4}{\omega_1^2\, pF}\, y\, J_0(yr)dy, \tag{A-22}$$

where $\overline{s_2}$ is the Laplace transform of s_2 , and

$$A_4 = \{\sinh(\omega_1 L_1) - \sinh(\omega_1 d_1) + \frac{pS_{s1}}{\omega_1 K_{z1}}[\cosh(\omega_1 L_1) - \cosh(\omega_1 d_1)]\} \times \cosh[\omega_2(b_2+z)]. \tag{A-23}$$

The Laplace inversion is accomplished numerically using the method of Stehfest (1970).

For the case in which the well is pumped from the lower zone, the solution can be obtained by a similar procedure. The drawdown in the Laplace domain is given as follows. In the upper zone,

$$\overline{s_1} = \frac{Q}{2\pi K_{z2}(L_2-d_2)} \int_{y=0}^{y=\infty} \frac{B_1}{\omega_2^2\, pF}\, y\, J_0(yr)dy, \tag{A-24}$$

where

$$B_1 = \{\sinh[\omega_2(b_2-d_2)] - \sinh[\omega_2(b_2-L_2)]\}\{\cosh[\omega_1(b_1-z)] + \frac{pS_{s1}}{\omega_1 K_{z1}}\sinh[\omega_1(b_1-z)]\}. \tag{A-25}$$

Above the intake interval in the lower zone ($-d_1<z<0$), we have

$$\overline{s_2} = \frac{Q}{2\pi K_{z2}(L_2-d_2)} \int_{y=0}^{y=\infty} \frac{B_2}{\omega_2^2\, pF}\, y\, J_0(yr)dy, \tag{A-26}$$

where

$$B_2 = \{\omega_2 K_{z2}\cosh(\omega_1 b_1)\cosh(\omega_2 z) - \omega_1 K_{z1}\sinh(\omega_1 b_1)\sinh(\omega_2 z) + \frac{pS_{s1}}{\omega_1 K_{z1}}[\omega_2 K_{z2}\sinh(\omega_1 b_1)\cosh(\omega_2 z) - \omega_1 K_{z1}\cosh(\omega_1 b_1)\sinh(\omega_2 z)]\} \times \{\sinh[\omega_2(b_2-d_2)] - \sinh[\omega_2(b_2-L_2]\}. \tag{A-27}$$

At the intake level of the pumped well ($-d_2<z<-L_2$), we have

$$\overline{s_2} = \frac{Q}{2\pi K_{z2}(L_2-d_2)}\int_{y=0}^{y=\infty}\left(1 - \frac{B_3}{F}\right)\frac{y\,J_0(yr)}{\omega_2^2\,p}\,dy, \tag{A-28}$$

where

$$B_3 = \left(v_1 + \frac{pS_{y1}}{\omega_1 K_{z1}}V_2\right)\cosh(\omega_1 b_1) + \left(V_2 + \frac{pS_{y1}}{\omega_1 K_{z1}}V_1\right)\sinh(\omega_1 b_1), \tag{A-29}$$

$$V_1 = \omega_2 K_{z2}\{\sinh(\omega_2 d_2)\cosh[\omega_2(b_2+z)] + \cosh(\omega_2 z)\sinh[\omega_2(b_2-L_2)]\}, \tag{A-30}$$

and

$$V_2 = \omega_1 K_{z1}\{\cosh(\omega_2 d_2)\cosh[\omega_2(b_2+z)] - \sinh(\omega_2 z)\sinh(\omega_2(b_2-L_2)]\}. \tag{A-31}$$

Finally, below the intake interval ($-L_2<z<-b_2$), we have

$$\overline{s_2} = \frac{Q}{2\pi K_{z2}(L_2-d_2)}\int_{y=0}^{y=\infty}\frac{B_4}{\omega_2^2\,pF}\,y\,J_0(yr)\,dy, \tag{A-32}$$

where

$$B_4 = \{\omega_2 K_{z2}[\cosh(\omega_1 b_1) + \frac{pS_{y1}}{\omega_1 K_{z1}}\sinh(\omega_1 b_1)][\sinh(\omega_2 L_2) - \sinh(\omega_2 d_2)] + \omega_1 K_{z1}[\sinh(\omega_1 b_1) + \frac{pS_{y1}}{\omega_1 K_{z1}}\cosh(\omega_1 b_1)] \times [\cosh(\omega_2 L_2) - \cosh(\omega_2 d_2)]\}\cosh[\omega_2(b_2+z)]. \tag{A-33}$$

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