FLOOD-HAZARD STUDY--100-YEAR FLOOD STAGE FOR APPLE VALLEY DRY LAKE SAN BERNARDINO COUNTY, CALIFORNIA

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations 11-75

Prepared in cooperation with the San Bernardino County Flood Control District

BIBLIOGRAPHIC DATA SHEET	1. Report No.	2.	3. Recipient's Accession No.
4. Title and Subtitle FLOOD-HAZARD STUDY--100-YEAR FLOOD STAGE FOR APPLE VALLEY DRY LAKE, SAN BERNARDINO COUNTY, CALIFORNIA			5. Report Date June 1975 6.
7. Author(s) Mark W. Busby			8. Performing Organization Rept. No. USGS/WRI-11-75
9. Performing Organization Name and Address U.S. Geological Survey, Water Resources Division 345 Middlefield Road Menlo Park, Calif. 94025			10. Project/Task/Work Unit No. 11. Contract/Grant No.
12. Sponsoring Organization Name and Address U.S. Geological Survey, Water Resources Division 345 Middlefield Road Menlo Park, Calif. 94025			13. Type of Report & Period Covered Final 14.

15. Supplementary Notes

Prepared in cooperation with the San Bernardino County Flood Control District

16. Abstracts

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17. Key Words and Document Analysis. 17a. Descriptors

*Flood frequency, *Flood routing, *Flood stages, *Synthetic hydrology, California, Desert playas, Water loss

17b. Identifiers/Open-Ended Terms

17c. COSATI Field/Group

18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
No restriction on distribution	UNCLASSIFIED	45
	20. Security Class (This Page) UNCLASSIFIED	22. Price

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June 1975

UNITED STATES DEPARTMENT OF THE INTERIOR

Stanley K. Hathaway, Secretary

GEOLOGICAL SURVEY

V. E. McKelvey, Director

For additional information write to:

District Chief
Water Resources Division
U.S. Geological Survey
345 Middlefield Road
Menlo Park, Calif. 94025

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CONVERSION FACTORS

Factors for converting English units to the metric units are shown to four significant figures. However, in the text the metric equivalents are shown only to the number of significant figures consistent with the values for the English units.

English	*Multiply by*	*Metric*
acres	4.047×10^{-1}	ha (hectares)
acre-ft (acre-feet)	1.233×10^{-3}	hm^3 (cubic hectometres)
ft (feet)	3.048×10^{-1}	m (metres)
ft/mi (feet per mile)	1.894×10^{-1}	m/km (metres per kilometre)
ft^3/s (cubic feet per second)	2.832×10^{-2}	m^3/s (cubic metres per second)
in (inches)	2.540×10	mm (millimetres)
mi (miles)	1.609	km (kilometres)
mi^2 (square miles)	2.589	km^2 (square kilometres)

FLOOD-HAZARD STUDY--100-YEAR FLOOD STAGE FOR APPLE VALLEY DRY LAKE

SAN BERNARDINO COUNTY, CALIFORNIA

By Mark W. Busby

ABSTRACT

A study of the flood hydrology of Apple Valley, Calif., was undertaken to develop the 100-year flood stage for Apple Valley dry lake. Synthetic hydrologic techniques were used because no adequate hydrologic or meteorologic data were available for the basin. The 100-year flood stage was estimated to be at an elevation of 2,909.0 feet (886.7 metres) above mean sea level.

INTRODUCTION

Accelerated land developments during the past 10 years in the deserts of southern California may cause problems related to urban zoning. Much of the development consists of second or vacation-type homes, but also includes many expensive permanent residences. Figure 1 shows a part of Apple Valley, most of which has been developed in the last 10 years. The figure also shows a mobile-home park under development. Desert playas or dry lakes often seem to be desirable development areas because of their flat topography and consequent abundance of good building sites. Although normally dry, playas commonly contain water after large storms, and homes built in and near the bottom of the playas can be subject to flooding.

Because of this flooding potential, San Bernardino County is establishing flood-zoned areas on many playas. The boundary of the flood zone is defined by the 100-year water level--the water level that is exceeded, on the average, once in a 100-year period.

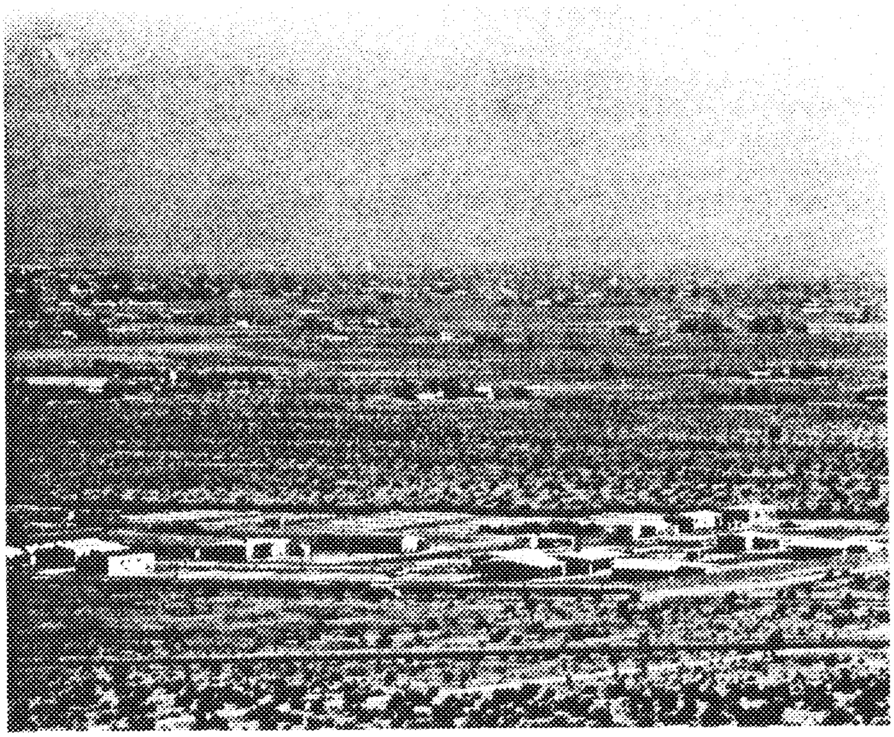

FIGURE 1.--Part of Apple Valley showing development. Note new mobile-home park being developed in foreground.

This report describes a pilot study into the development of the 100-year flood level for Apple Valley dry lake, Calif. (fig. 2). As in most hydrologic studies in desert areas, there are no prior studies to give guidelines for the development of techniques. This problem is further compounded by the lack of adequate hydrologic or meteorologic data within the study area. Thus this study, of necessity, involves the techniques of synthetic hydrology.

The objective of this study was to develop an elevation-frequency curve for Apple Valley dry lake.

This report was prepared by the U.S. Geological Survey in cooperation with the San Bernardino County Flood Control District.

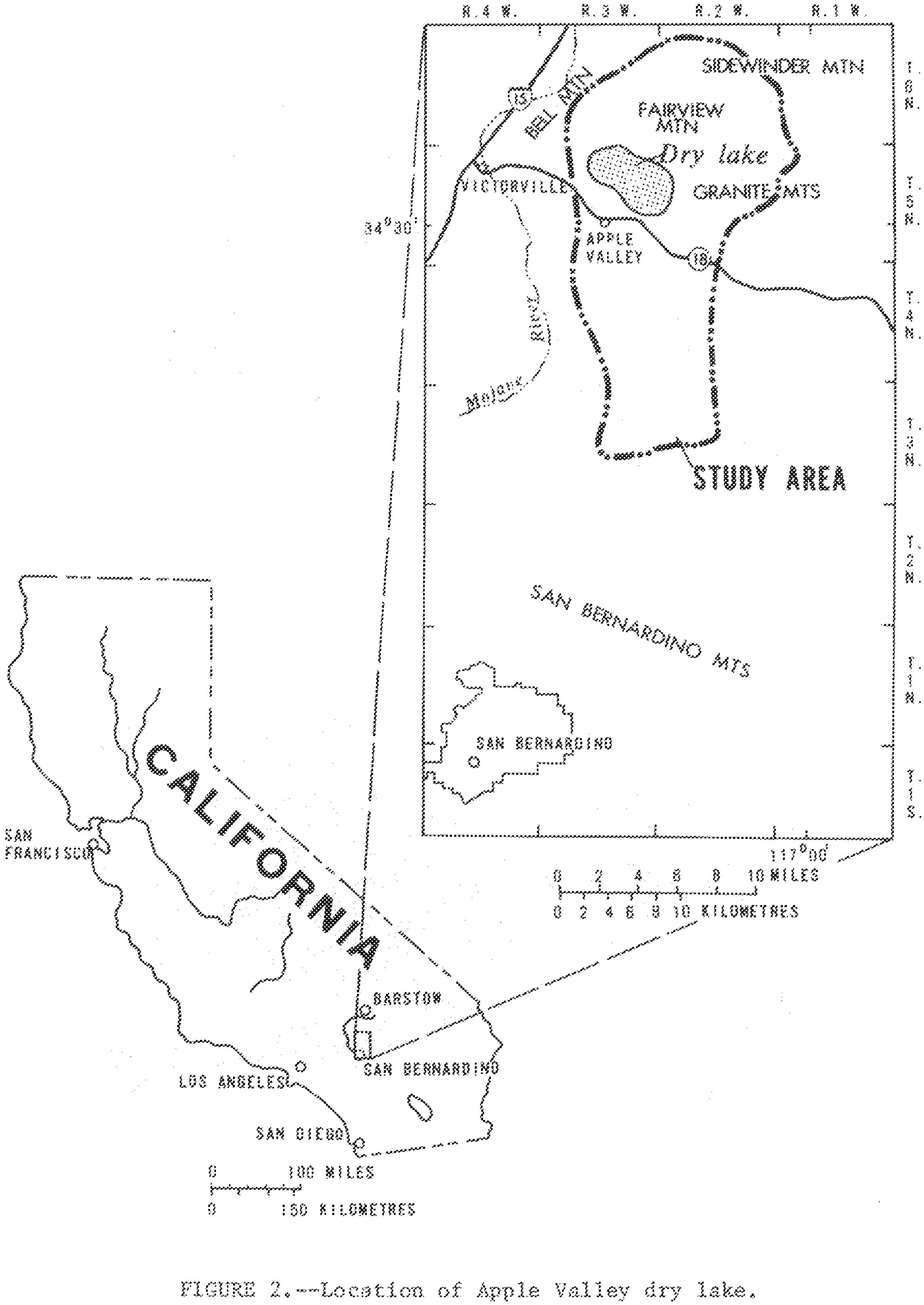

FIGURE 2.--Location of Apple Valley dry lake.

DESCRIPTION OF AREA

Apple Valley dry lake is in Apple Valley in the high desert part of southwestern San Bernardino County, about 6 mi (10 km) east of Victorville and 30 mi (48 km) north of San Bernardino. The dry lake occupies the lowest part of a closed desert basin that is about 9 mi (14.5 km) wide and 18 mi (29.0 km) long, with the basin floor at an elevation of 2,900-3,000 ft (884-914 m) above mean sea level rising to an elevation of 4,900 ft (1,490 m) to the east, 3,800 ft (1,160 m) to the west and north, and 6,000 ft (1,830 m) to the south.

The mountains surrounding Apple Valley are generally barren, rugged, steep walled, and isolated and are composed mostly of schist and gneiss. Quartzite, quartz monzonite, granodiorite, limestone, and sandstone are also found in the bedrock assemblage. A large limestone quarry is on the northeast edge of the valley. The valley floor is unconsolidated sediment, consisting of gravel, sand, silt, and clay. Figures 3 and 4 show some of the mountains around Apple Valley.

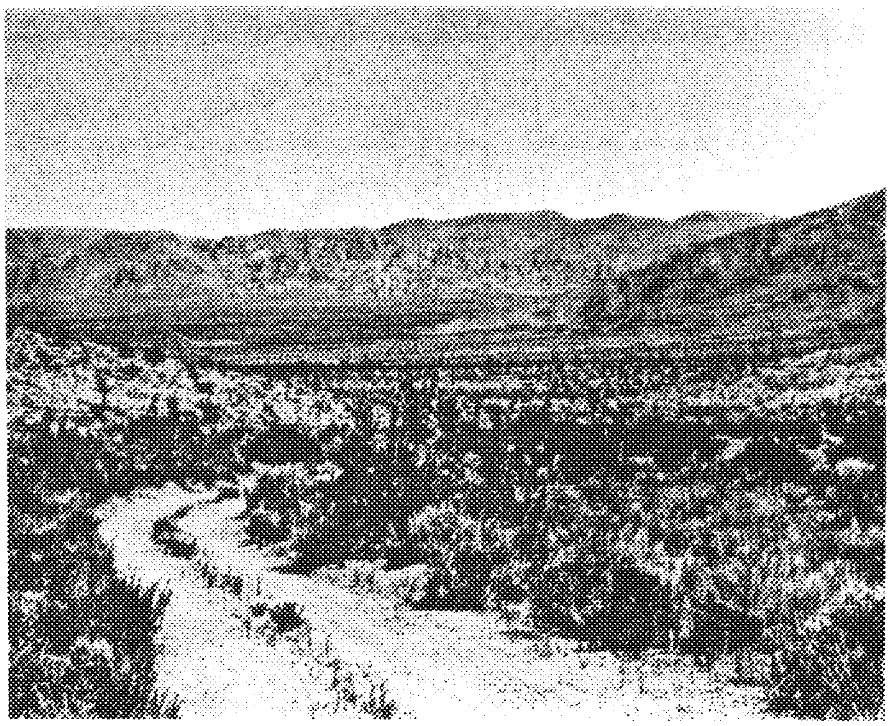

FIGURE 3.--Mountains along the eastern boundary of Apple Valley.

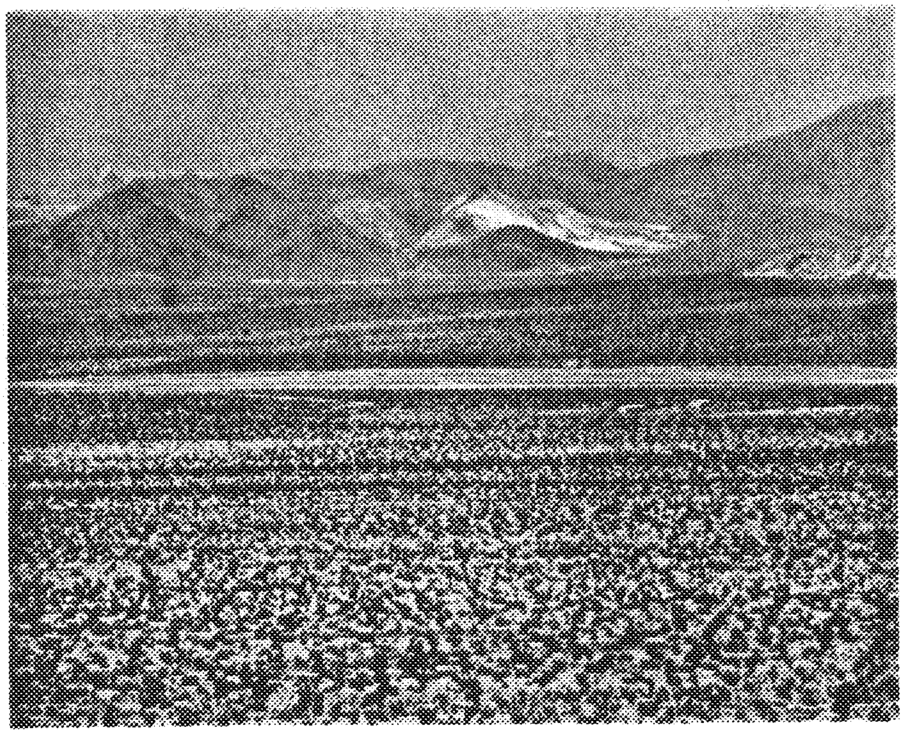

FIGURE 4.--Mountains and valley floor in northern part of Apple Valley. Limestone quarry is in mountains in middle of photograph.

Runoff originates in the mountains surrounding the valley, but little generally reaches the playa. That which does comes only from the northern half of the basin. The streams in the valley are all ephemeral--that is, they carry water only during and immediately after a storm. Most of the channels are well defined for only about 1 mi (1.6 km) after they leave the mountains, whereupon they become braided and ill defined and usually disappear after just a few miles. Only one channel is clearly defined to the playa itself. Figures 5, 6, and 7 show the changes in one channel.

Data from the National Weather Service (U.S. Weather Bureau) show that the mean annual precipitation at Victorville from 1939 to 1968 was 4.97 in (126.2 mm). The mean annual temperature at Victorville for 1940-65 was 59.6°F (15.3°C), and the mean monthly temperature ranged from 42.6°F (5.9°C) in January to 78.7°F (25.9°C) in July. In July and August, temperatures are frequently more than 100°F (37.8°C). Apple Valley dry lake is only 6 mi (10 km) east of Victorville and has a similar climate. Precipitation on the study area is about 6 in (150 mm) per year. In contrast, precipitation in the nearby San Bernardino Mountains (fig. 2) averages about 40 in (1,020 mm) per year.

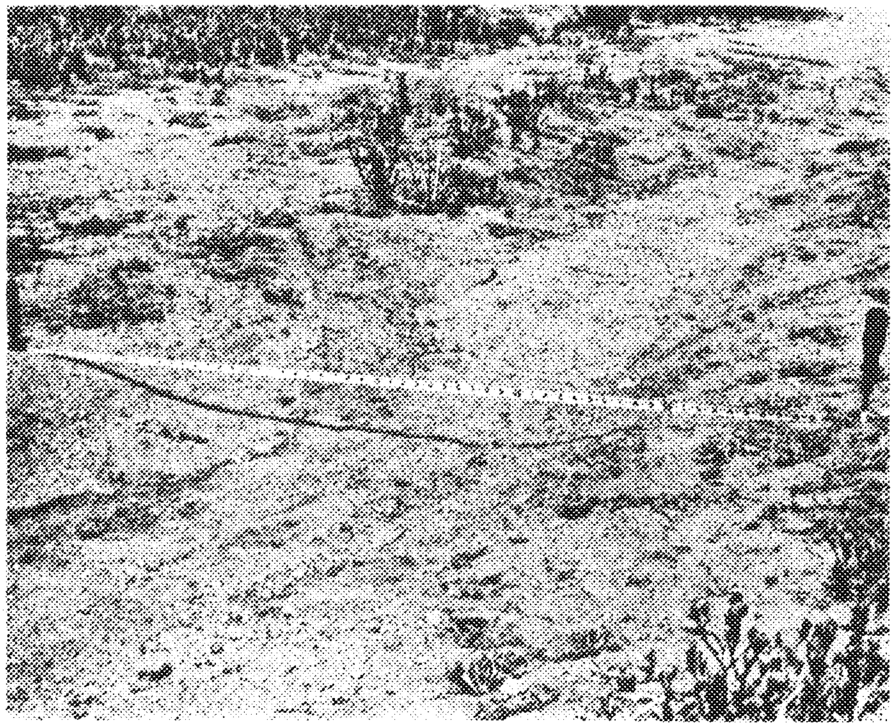

FIGURE 5.--Well-defined channel in northern part of Apple Valley.

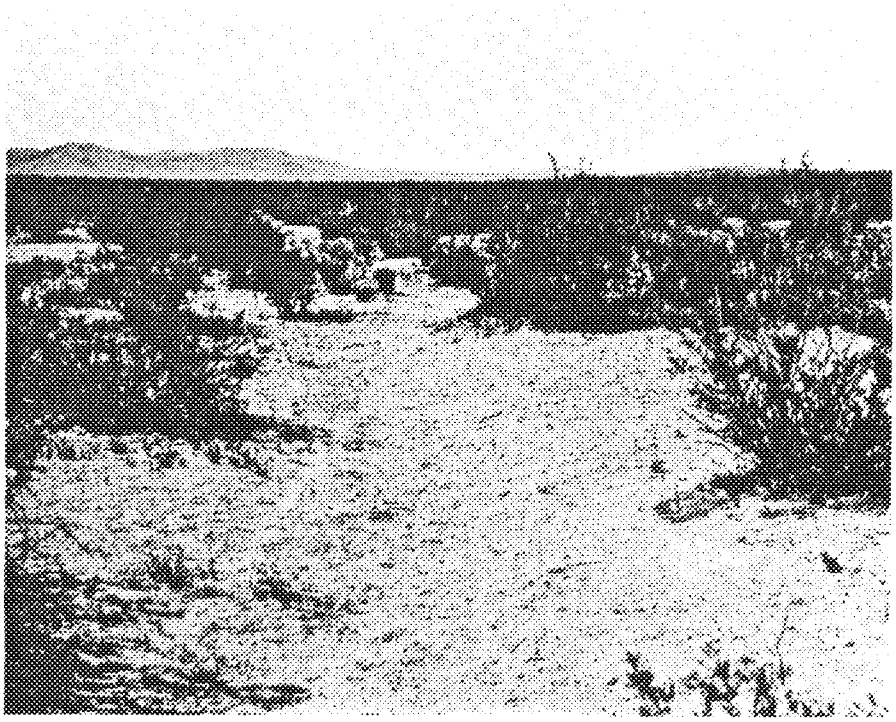

FIGURE 6.--Poorly defined channel about 1 mile (1.6 kilometres) downstream from site in figure 5.

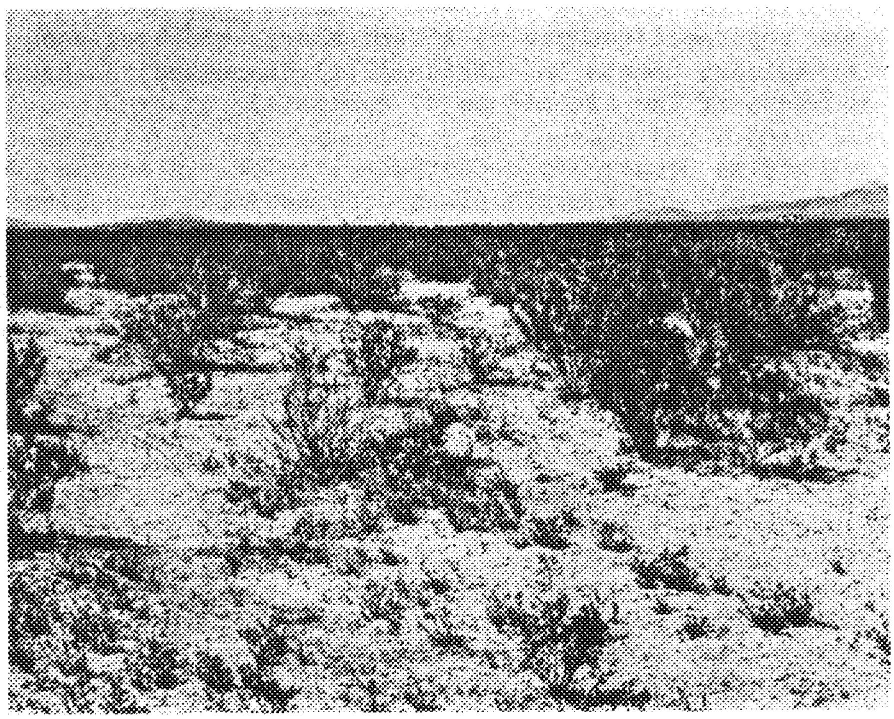

FIGURE 7.--Braided and ill-defined channel about half a mile (0.8 kilometre) downstream from site in figure 6.

METHOD OF ANALYSIS

The lack of hydrologic or meteorologic data for the Apple Valley area requires a synthetic-hydrologic analysis. Available techniques include:

1. Stochastic methods
   a. Regression models
   b. Markovian models
   c. Harmonic analysis
2. Deterministic methods
   a. Stanford Watershed model
   b. U.S. Geological Survey model
   c. Unsteady flow model
3. Empirical methods
   a. Channel routing
   b. Runoff zones
   c. Channel geometry
   d. Rational method
   e. Unit hydrograph

"Channel bar.--A longitudinal, in-channel depositional feature formed along the borders of a stream channel at a stage of the flow regime when the local competence of the stream is incapable of moving the sediment particles on the submerged surface of the bar. Emerged channel bars are generally free of perennial vegetation. A channel bar may extend for a considerable distance along the channel or it may be one of a series of bars that occupy similar relative positions in the channel. These features previously have been termed berms in the literature (Moore, D. O., 1968, p. 34, and Hedman, E. R., 1970, p. E5). It is proposed that the term *channel bar* be used exclusively for this in-channel feature to avoid confusion. Channel bars are used as reference levels in channel-geometry measurements of width and mean depth in estimating flow characteristics.

"Point bar.--A point bar is a depositional feature formed by lateral accretion on the inside, or convex side, of a channel bend. Deposition on the convex edge of the channel and the concomitant erosion of the concave bank both tend to be greatest just downstream from the position of maximum curvature. The processes of erosion and deposition tend to maintain a constant channel width during lateral shifting of the channel (Wolman and Leopold, 1957). The surface of a point bar may be used, together with channel bars or mid-channel bars, to obtain channel-geometry measurements of width and mean depth in estimating flow characteristics."

The concept behind the channel-geometry approach is that the channel dimensions adjust themselves to the streamflow. Expressed simply, large channels carry large discharges, and small channels carry small discharges. Thus, the measurement of the channel geometry should provide empirical evidence of the magnitudes and frequencies of flows carried by that channel. Figures 8, 9, and 10 show the channel-geometry features that were measured for three different streams in Apple Valley.

The methods for selecting the channel and point bars and for measuring the cross sections in the field have been fairly well standardized, but some field training and experience are necessary for consistent results in a given region. Any obstruction in a channel can cause a local variation in the channel geometry, and it is important not to use non-typical bars formed as a result of such obstructions. Only the bars that continue or reappear at a consistent elevation above the streambed thalweg should be used.

D. O. Moore (written commun., 1972) developed a series of curves relating width and mean depth to the 10-year flood for Nevada. Because most of the data used to develop these curves were from California, and the rest were from southern Nevada where the climatic and hydrologic conditions are similar to those in Apple Valley, it was decided to use these curves to determine the 10-year flood for the measured channels in the Apple Valley area.

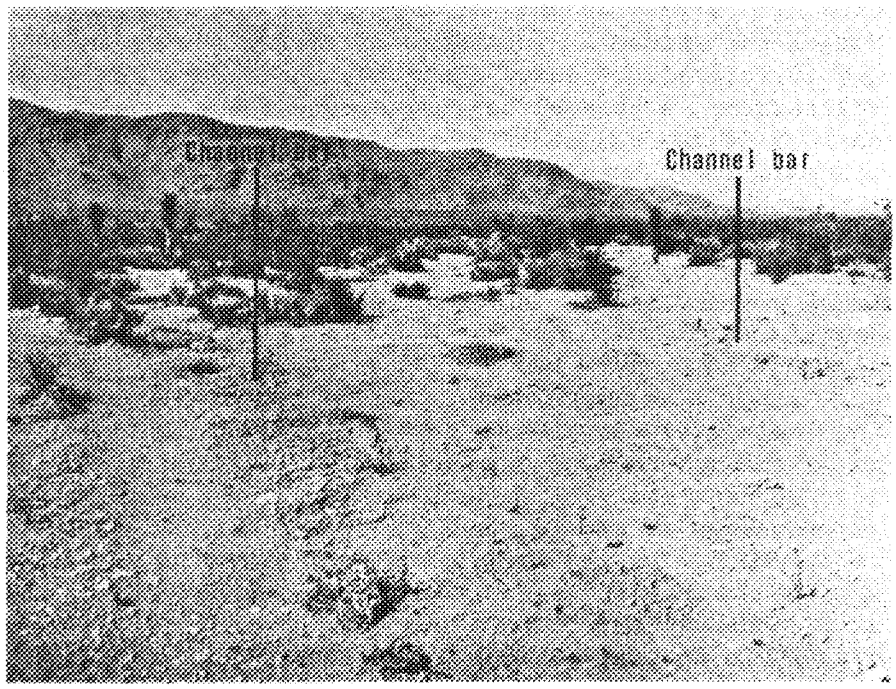

FIGURE 8.--Channel-geometry features, site H, Apple Valley.

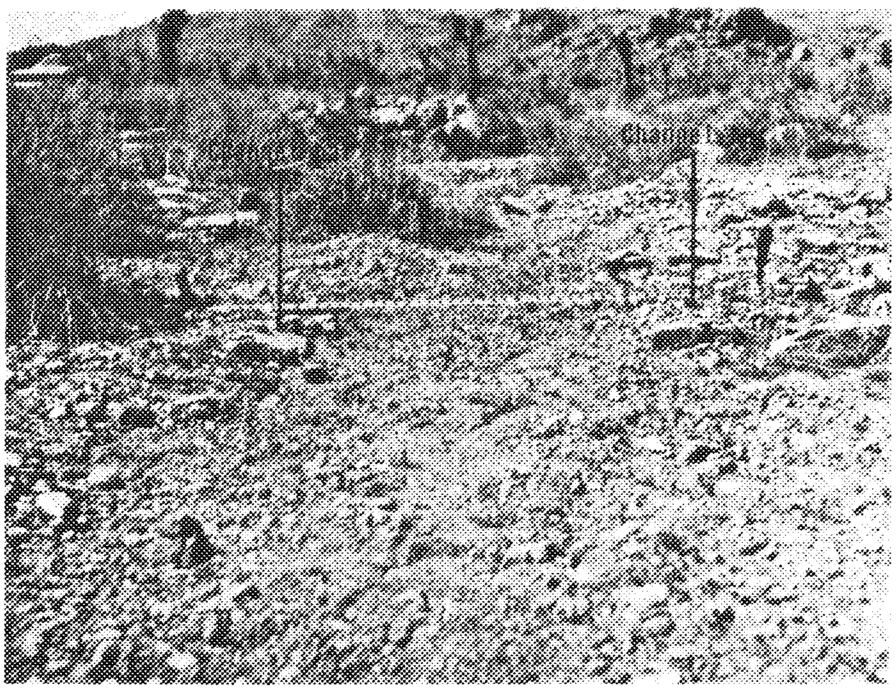

FIGURE 9.--Channel-geometry features, site D, Apple Valley.

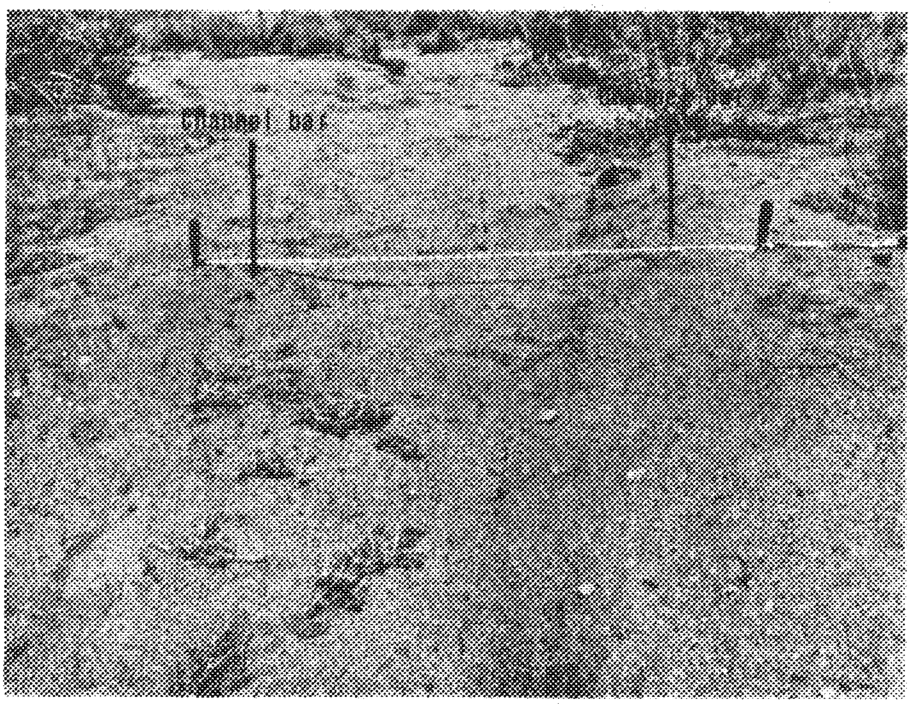

FIGURE 10.--Channel-geometry features, site X, Apple Valley.

Using the techniques described above, the channel geometry was measured at 46 sites, 24 of which were for the routing calibration computation, 17 were for the final discharge computation, and 5 additional sites were for the area 10-year flood relation. Figure 11 shows the location of these 46 sites. Tables 1, 2, and 3 list data on the channel geometry as measured in the field and the computed discharges for these data.

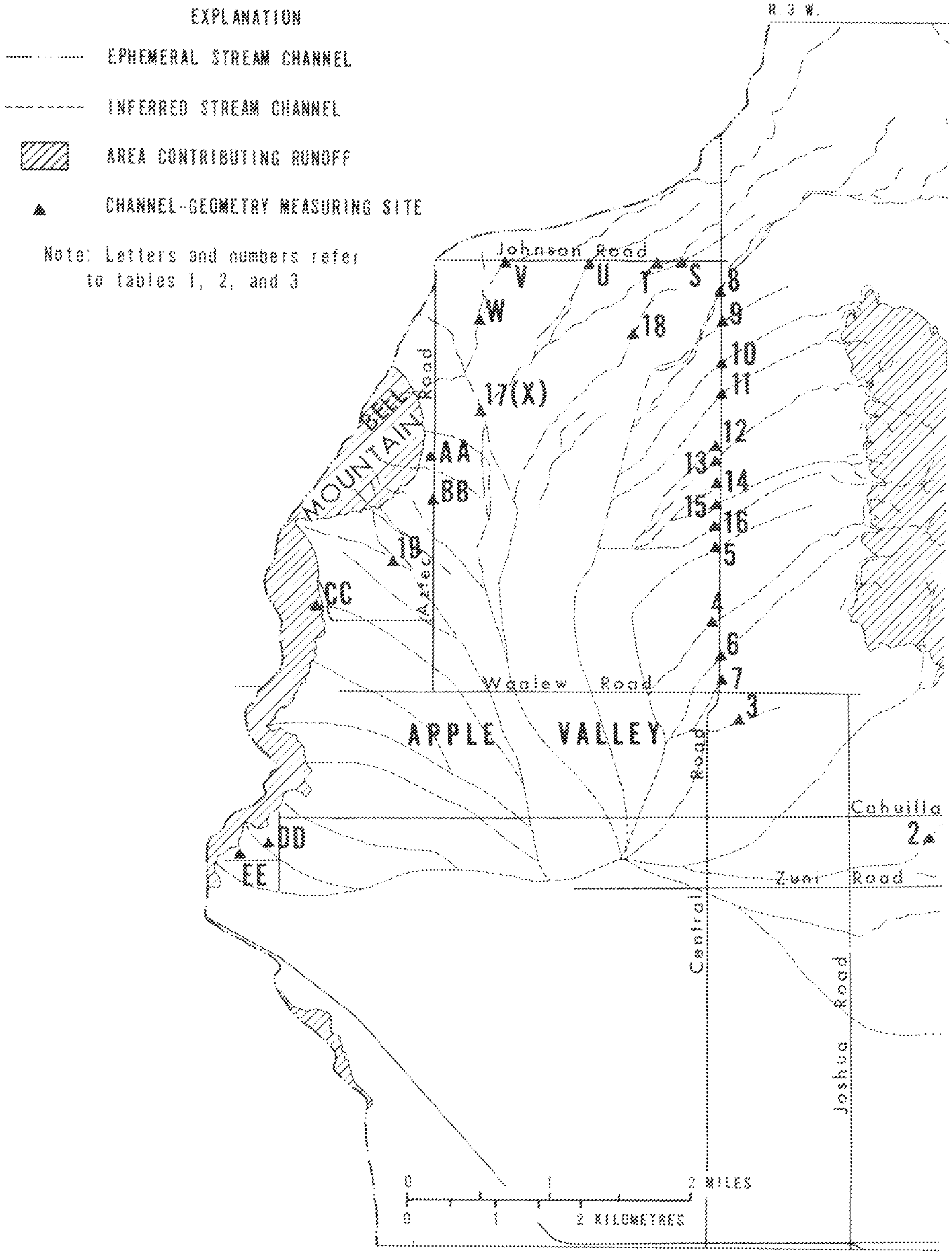

FIGURE 11.—Location of channel-geometry measuring sites.

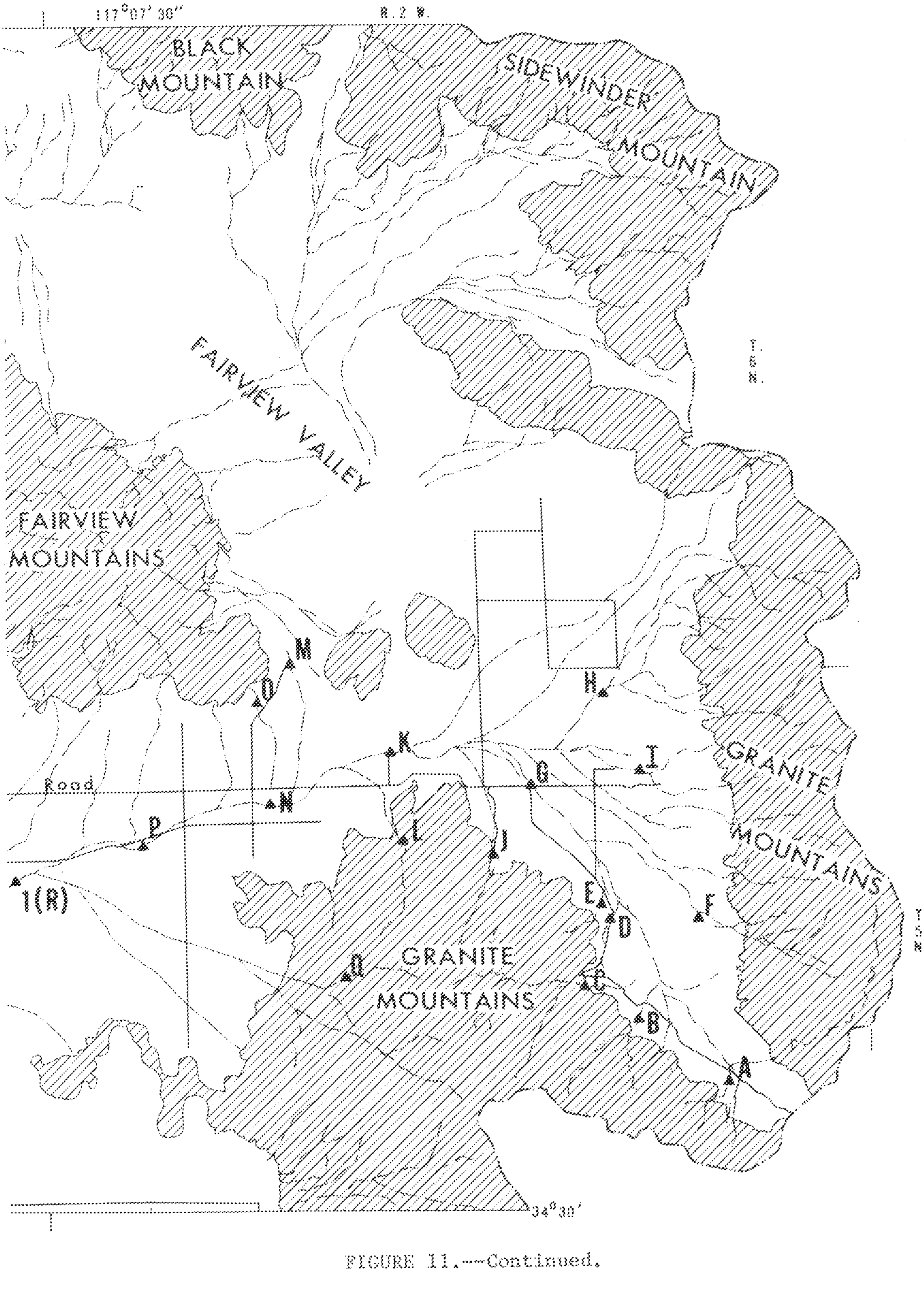

FIGURE 11.--Continued.

TABLE 1.--*Channel geometry and 10-year flood discharge-routing calibration data*

Site	Channel geometry Width (feet)	Channel geometry Depth (feet)	10-year flood discharge (cubic feet per second) Q[1]	Q_{10}[2]
A	4.4	0.11	290	290
B	4.0	.30	460	485
	6.0	.17	510	
C	8.0	.35	1,030	1,000
	9.75	.13	970	
D	4.0	.22	330	330
E	3.6	.13	220	220
F	12.0	.25	1,400	1,400
G	12.0	.22	1,350	1,350
H	16.0	.12	1,770	1,800
	16.5	.13	1,830	
I	7.1	.42	1,100	960
	7.5	.32	910	
	9.0	.14	870	
J	4.0	.14	270	240
	3.2	.18	210	
K	19.5	.20	2,260	1,960
	13.0	.45	1,890	
	13.5	.36	1,790	
L	5.0	.22	440	510
	6.5	.18	580	
M	7.0	.05	580	620
	7.5	.10	660	
N	12.0	.36	1,610	1,610
O	7.5	.13	670	670
	7.0	.21	670	
P	10.5	.34	1,350	1,620
	16.5	.20	1,890	
Q	6.0	.21	550	550
R	20.0	.25	2,400	2,400
S	3.0	.11	150	150
T	7.0	.11	600	600
U	3.9	.23	340	450
	6.0	.22	560	
V	6.0	.20	540	400
	3.5	.19	250	
W	4.0	.18	290	290
X	7.0	.21	670	670

[1]Discharge computed from channel geometry.
[2]Average of one or more computations for a site.

TABLE 2.--*Channel geometry and 10-year flood discharge supplemental data for drainage area relation*

Site	Channel geometry Width (feet)	Channel geometry Depth (feet)	10-year flood discharge (cubic feet per second) Q_{10}[1]
AA	4.0	0.11	250
BB	4.2	.17	300
CC	2.0	.10	90
DD	2.0	.10	90
EE	1.5	.10	70

[1]Discharge computed from channel geometry.

TABLE 3.--*Channel geometry and 10-year flood discharge-computation data*

Site	Channel geometry Width (feet)	Channel geometry Depth (feet)	10-year flood discharge (cubic feet per second) Q[1]		Q_{10}[2]
1(R)	20.0	0.25		2,400	2,400
2	10.0	.21		1,060	1,060
3	16.5	.17		1,850	1,850
4	11.0	.16		1,150	1,150
5a	4.0	.08		240	330
b	2.0	.10		90	
6a	1.5	.10		70	280
b	1.5	.10		70	
c	1.5	.10		70	
d	1.5	.10		70	
7	3.5	.15		220	220
8a	4.2	.20		330	1,600
b	8.2	.13		760	
c	5.0	.13	370	510	
	7.5	.14	680		
9	5.0	.10		360	360
10	9.0	.10		850	850
11	7.5	.18	700	525	525
	5.0	.07	350		
12	4.0	.08		240	240
13a	3.6	.14		220	310
b	2.0	.09		90	

See footnotes at end of table.

TABLE 3.--*Channel geometry and 10-year flood discharge-computation data*--Continued

Site	Channel geometry Width (feet)	Channel geometry Depth (feet)	10-year flood discharge (cubic feet per second) Q[1]	10-year flood discharge (cubic feet per second) Q_{10}[2]
14a	1.2	0.07	50	
b	3.3	.10	80	380
c	4.0	.11	250	
15a	2.5	.15	140	440
b	4.4	.13	300	
16	4.4	.07	280	280
17(X)	7.0	.21	670	670
18	7.0	.13	610	610
19a	4.5	.08	300	
b	4.5	.08	300	
c	4.5	.09	310	1,450
d	3.5	.13	210	
e	4.5	.14	330	

[1]Discharge computed from channel geometry.
[2]Average of one or more computations for a site. Braces indicate values of total column added to determine the 10-year flood discharge.

NOTE: The lower case letters a to e indicate separate channels measured at a site.

Flow from Unmeasured Sites

At many sites it was not possible to measure the channel geometry because: (1) Channel bars or berms did not form, (2) they were destroyed by the wind or by man, or (3) the channels were not readily accessible. For those channels some other means of determining the 10-year flood was needed. Figure 12 is a plot of the 10-year flood against the contributing drainage area for all the measured sites where a contributing drainage area could be reasonably determined. The contributing drainage area was defined as that area upstream from the site above the point where the alluvial fan starts to flatten. On a topographic map it is the point where the contour lines become much farther apart than they are in the mountain areas. The orientation of the basin affected the resultant discharge, so two different but parallel curves were drawn. The basins facing the south and west would logically have more runoff because they face oncoming storms, and the basins facing north and east would have less runoff because they are in a rain shadow.

Figure 12 was used to determine the 10-year flood discharge for all basins except those for which channel geometry was measured downstream from the point of contributing drainage area.

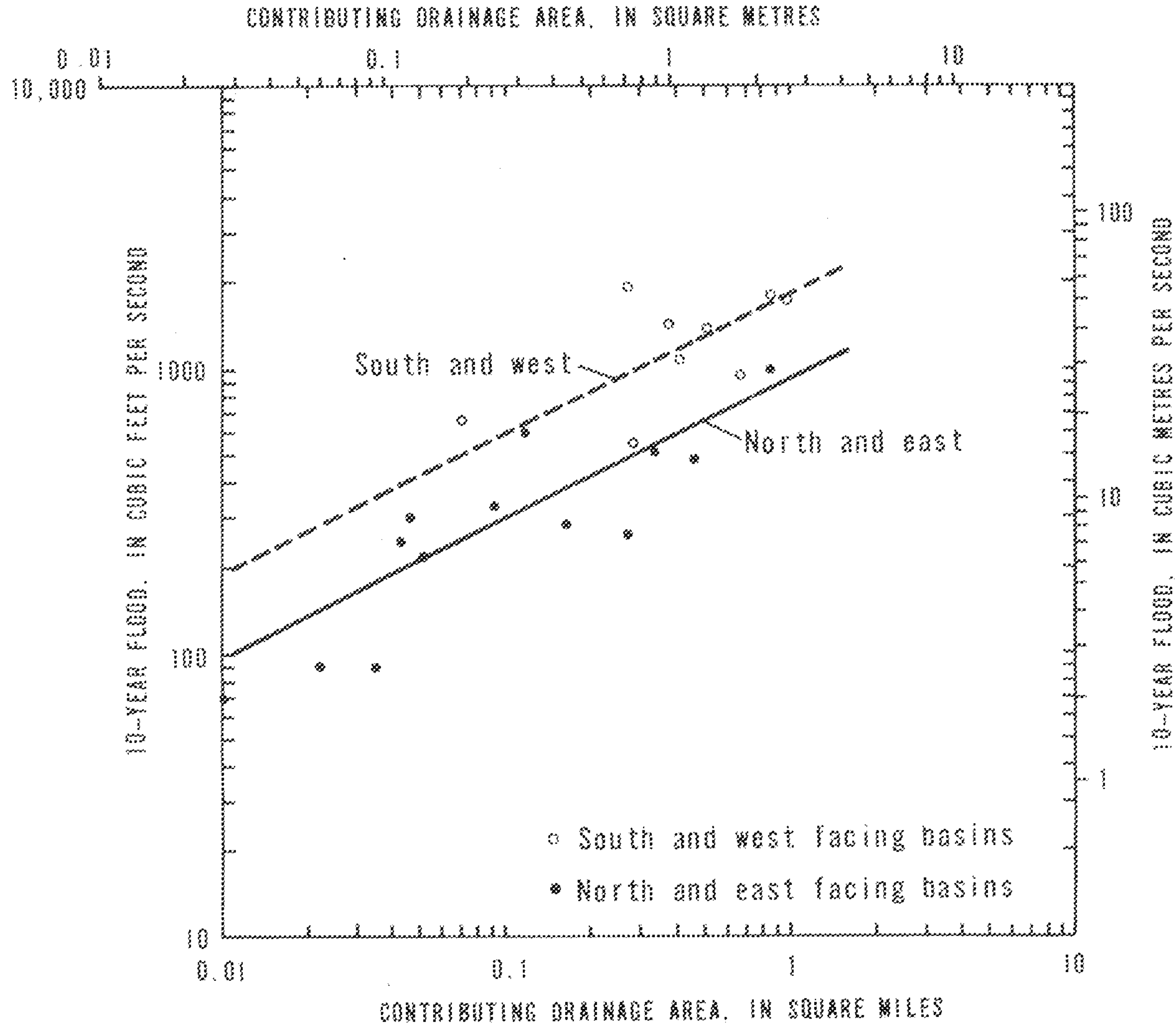

FIGURE 12.--Relation of 10-year flood to contributing drainage area for Apple Valley.

Channel Losses

In arid regions most of the runoff is generated in the mountains and steeper alluvial slopes; little is derived from the valley floors. The runoff generated in the mountains usually decreases as the water infiltrates into the alluvial fans, and if the volume is small, the flow may disappear entirely after traveling a short distance from the mountains. Because of the braided nature of stream channels in the desert, it was not practical to measure the channel geometry downstream from the mountain front. Thus, some means of accounting for losses from infiltration had to be developed for the channels where the channel geometry was measured upstream from the playa. This section describes the technique used to account for these losses.

Unsteady flow techniques could not be used to route the flows downstream from the mountains because the large quantity of continuous flow data necessary to establish the boundary conditions was not available to solve the unsteady flow equations for Apple Valley. Because only channel-geometry data were available, there was no justification to synthetically develop this mass of data.

An alternative in determining channel losses was to develop an empirical method using measurable channel factors. Three of the most important factors used in the Apple Valley study include: (1) Discharge, (2) texture of bed material, and (3) channel slope. Of primary importance was stream discharge, because water losses are directly related to the rate and duration of flow. The losses are also related to the size gradation (sorting) of the bed material. However, a good single index of bed material is difficult to determine. Under the assumption that only normal fluvial processes (excluding mudflows and debris flows common in some arid regions) are operative and that consequently the sorting is related to the distance downstream, the channel distance from the basin divide downstream to the point of loss determination was used. The length of time it takes a known volume of flow to pass a point is also related to the losses. This time cannot generally be measured directly, so some index must be used. Steeper sloped basins usually have shorter and sharper peaked hydrographs with a shorter length of flow time. Channel slope near the point of loss would be an index of the general basin slope and thus be used as an index of the flow time. Therefore, channel slope measured from topographic maps was used as the third factor.

A logical form for the channel-loss relation is that the losses are some percentage of the upstream discharges, expressed as percentage loss per mile. For computational purposes, it is easier to use a retention per mile rather than loss per mile, where retention percentage is merely 100 percent minus the loss percentage. The equation for computing a discharge at the downstream end of a losing reach would be:

$$Q_d = Q_u \times C^{Di} \qquad (1)$$

where Q_d and Q_u are the downstream and upstream discharges, C is the retention percentage coefficient, and Di is the distance in miles between discharge points.

To determine the coefficient C in equation 1 for the Apple Valley area, the channel geometry was measured at a series of points down two channels within the valley. This gave discharges for a series of upstream-downstream points to allow solving for the coefficient. This coefficient was then related to the three channel factors described above to allow transfer to any reach within Apple Valley.

The relation of the retention coefficient to the three channel factors was developed using an optimization computer program developed by D. R. Dawdy (written commun., 1972) that included equation 1. In finding the best solution, the first test was the sum of squares of the differences between the discharges measured by channel geometry at nine points and the discharges computed by equation 1. The several models that gave the smaller sum of squares of differences were then analyzed further. Because the critical routing in the final run is for the long distances into the playa, the sum of squares of differences for the four longer distances was then examined. A comparison of the results of the final run was then made to evaluate reasonableness of the final results.

Many different models were tried in solving for the coefficient C. These included:

1. $C = a + bQ$
2. $C = aQ^b$
3. $\log C = a + bQ$
4. $C = a + b_1Q + b_2D$
5. $C = aQ^{b_1}D^{b_2}$
6. $C = aQ^bS^{1/2}$
7. $C = aQ^{b_1}S^{b_2}$
8. $C = aQ^{b_1}D^{b_2}S^{b_3}$
9. $C = a + b_1Q + b_2Q^2 + b_3D + b_4D^2 + b_5S + b_6S^2$

where a, b, b_1-b_6 are coefficients to be evaluated

Q is discharge in cubic feet per second at upstream end of routing reach
D is distance in miles from divide to upstream end
S is general land slope in feet per mile at upstream end.

Models 1 through 7 did not have the flexibility to fit the data very well. Models 8 and 9 both had sufficient flexibility to fit all points about equally well, but model 9 was a better fit to the longer reaches, which is the most critical point for the generation of the final results. Only these two models had a root mean square error of less than 700 ft^3/s (19.8 m^3/s), so they were clearly the two best choices. Model 8 had a root mean square of 266 ft^3/s (7.53 m^3/s) and model 9 of 297 ft^3/s (8.41 m^3/s). However model 9 was superior for the last two tests. For the four longer distances, model 8 had a root mean square of 355 ft^3/s (10.1 m^3/s), indicating its best fit was for the shorter distances, but model 9 had a root mean square of 265 ft^3/s (7.51 m^3/s), or a better fit for the longer distances than for the shorter distances. Also when the final run was made for these two models, model 8 produced results almost an order of magnitude less than what was anticipated.

The effect of the three factors of discharge, distance, and slope are about as would be expected. The retention of flow increases with increasing discharge, with the exception of the small discharges; the retention is about constant for the shorter distances and then decreases with longer distances; and the retention increases with increasing slope, except for the very flat slopes.

Model 9 had the drawback of producing opposite signs for the two discharge coefficients and the two slope coefficients. This means that the routing coefficient C would have the same value for a discharge of 100 ft^3/s (2.83 m^3/s) and 1,970 ft^3/s (55.8 m^3/s), holding the distance at 2.80 mi (4.51 km) and the slope at 140 ft/mi (26.5 m/km); or a slope of 20 ft/mi (3.79 m/km) and 432 ft/mi (81.9 m/km), holding the discharge at 700 ft^3/s (19.8 m^3/s) and the distance at 2.80 mi (4.51 km), as examples. For the discharge, however, this is not a problem when examined in terms of equation 1 because the change in discharge overrides the change in the coefficient C giving a continually increasing resultant downstream discharge as the upstream discharge increases. The slope problem can only be explained by the fact that apparently the steep reaches at the mountain fronts and the flat reaches near the playa somehow react similarly.

The final equation for the solution of the routing coefficient is:

$$C = 1.32 - 0.853\,Q + 0.412\,Q^2 - 0.026\,D - 0.0036\,(D - 4)^2\,\alpha - 0.714\,S + 0.158\,S^2 \qquad (2)$$

where $\alpha = \begin{cases} 0, & D \leq 4 \\ 1, & D > 4 \end{cases}$

and the other terms are as defined above, except for the scaling multipliers of 0.001 for Q and 0.01 for S.

The analysis indicates the expected error (not standard error) in the use of equations 1 and 2 should be 300 ft^3/s (8.50 m^3/s). However as only nine points were available for testing, the real error is probably larger by some unknown amount.

Appendix A lists the data used in the calibration of the routing model and describes the flow network used in routing to the various calibration points.

Flood Ratios

The previous sections described a means of determining the 10-year flood for any stream channel. However, predictions of floods greater than the 10-year flood are needed to determine the proper zoning boundaries.

The 10-year channel-geometry flood relations cannot be utilized in predicting larger floods. Because it was beyond the scope of this project to develop the necessary channel-geometry relations for larger floods, other methods were used to determine the 100-year flood stage.

For all the gaging stations in the desert regions of southern California, the ratios of the 10-year flood to various other flood levels were examined. Figure 13 shows the flood ratios for the 16 desert stations where the flood frequency curves have been defined. As would be expected, there is some scatter of the points. However the curve joining the means at the various recurrence intervals is fairly well defined. The ratios used to define the curve were:

$$\frac{P_2}{P_{10}} = 0.12,\ \frac{P_5}{P_{10}} = 0.47,\ \frac{P_{25}}{P_{10}} = 2.37,\ \frac{P_{50}}{P_{10}} = 4.39$$

Ratios for any other frequency may be determined from the curve. For example:

$$\frac{P_{100}}{P_{10}} = 7.40$$

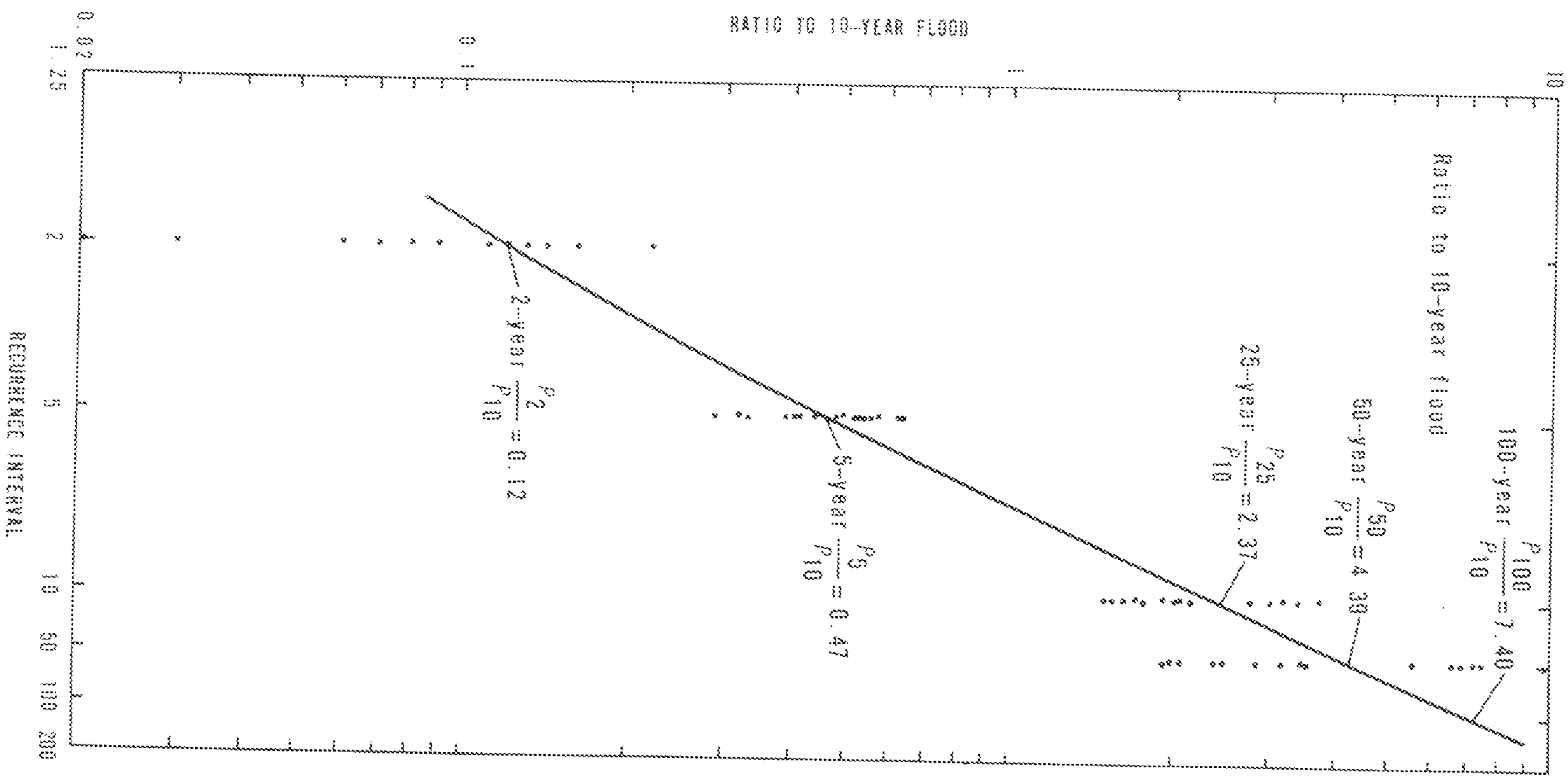

FIGURE 13.--Ratios to 10-year flood for desert basins, California.

The entire flow network from the various discharge points into the playa was next schematically determined, and all the discharges were routed down to the playa using equations 1 and 2. This procedure gave the 10-year flood discharge into the playa. Appendix B lists the data used in the final run and the flow network used in the routing of the 10-year floods to the playa. The ratios of the 10-year flood to the various other floods were then used to compute the required flood peaks into the playa.

One further step was necessary to find the flood volumes into the playa. The records for a number of desert basins were analyzed to compare the peak discharges with the associated storm-runoff volume. Figure 14 is a plot of these data and gives the following relation between peak discharge and volume:

$$V = 0.0339\ P^{1.150} \tag{3}$$

where V = runoff volume, in acre-feet
P = flood peak, in cubic feet per second.

Equation 3 was then used to compute the flood volumes into the playa.

RESULTS

Table 4 presents the flood frequency and flood-volume frequency for Apple Valley dry lake. The flood-volume frequency from table 4 was used with the elevation-volume from table 5 to develop the elevation-frequency curve of figure 15. Using this figure and table, the 100-year flood stage was determined to be at elevation 2,909.0 ft (886.7 m) with a corresponding surface area of 1,810 acres (733 ha). Table 6 is a summary of the results of this study.

TABLE 4.--*Flood frequency and flood-volume frequency for Apple Valley dry lake*

Recurrence interval (years)	Flood peak (ft^3/s)	Flood volume (acre-ft)
2	570	50
5	2,240	242
10	4,770	579
25	11,300	1,560
50	20,900	3,170
100	35,300	5,750

TABLE 5.--*Elevation, area, and volume table for Apple Valley dry lake*

Elevation (ft above m.s.l.)	Surface area (acres)	Volume (acre-ft)
2,902.15	0	0
2,902.5	5.1	1.2
2,903	69	23
2,903.5	133	75
2,904	316	202
2,904.5	481	411
2,905	654	713
2,905.5	798	1,070
2,906	948	1,520
2,906.5	1,080	2,040
2,907	1,290	2,640
2,907.5	1,400	3,300
2,908	1,520	4,060
2,908.5	1,660	4,870
2,909	1,810	5,750
2,909.5	1,930	6,700
2,910	2,060	7,700
2,910.5	2,150	8,770
2,911	2,230	9,860
2,911.5	2,390	11,000
2,912	2,520	12,300
2,913	2,780	15,000
2,914	3,050	17,900
2,915	3,310	21,200

TABLE 6.--*Elevation frequency for Apple Valley dry lake*

Recurrence interval (years)	Elevation (ft above m.s.l.)
2	2,903.2
5	2,904.1
10	2,904.7
25	2,906.0
50	2,907.4
100	2,909.0

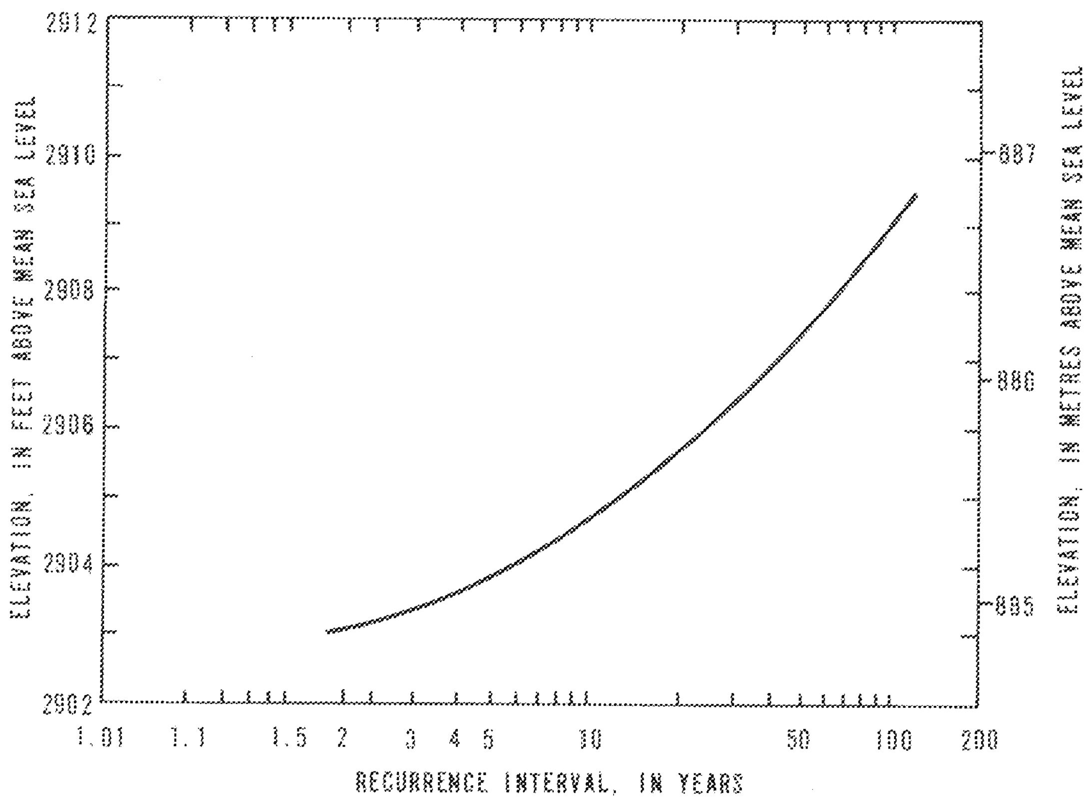

FIGURE 15.--Elevation-frequency curve for Apple Valley dry lake.

As with any analytical technique, there is the possibility of the answer being in error. The magnitude of the error is not directly measurable from a technique as involved as the one used. However an estimate of 1 ft (0.3 m) would not be unreasonable. Thus the true 100-year flood stage should probably be within the range of 2,908.0 ft (886.4 m) to 2,910.0 ft (887.0 m).

An examination of aerial photographs taken in the summer of 1969 shows a distinct textural and vegetal change at about the 2,910-ft (887-m) contour, giving a general confirmation of the probable high water in the past. Figure 16 is a photograph of this obvious change.

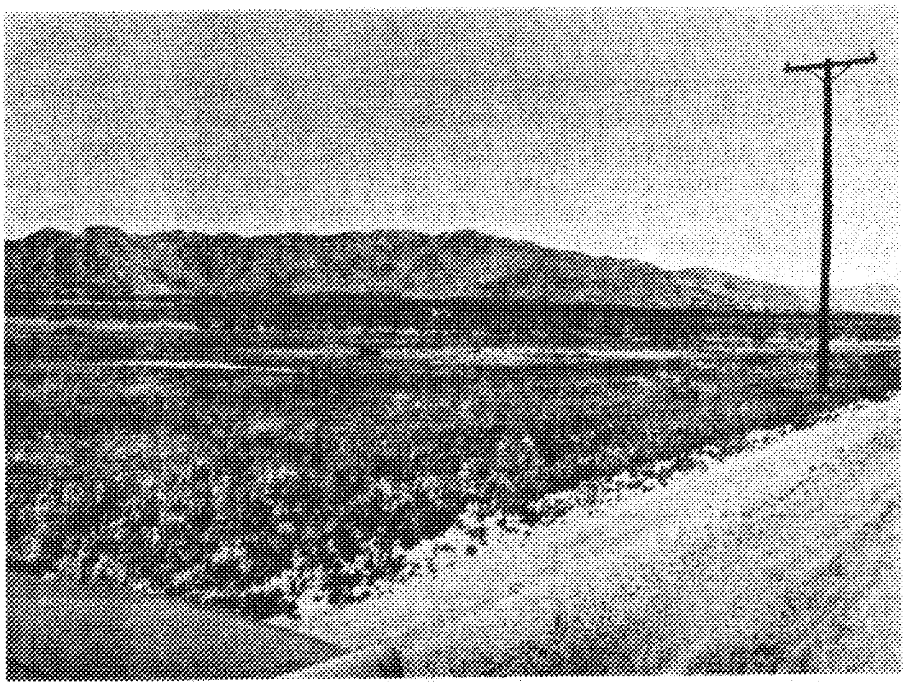

FIGURE 16.--Textural and vegetal change from grass to brush at about the 2,910-foot (887-metre) elevation, Apple Valley.

Future channel-improvement work will alter the 100-year flood stage. Canalization would allow the water to reach the playa faster with an appropriate decrease in channel losses and a consequent increase in the 100-year flood stage. This is particularly true for channels to the south of the playa where, at present, no flow reaches the playa, but where, with improved channels, additions to the inflow are probable.

The results presented in table 6 and figure 15 are considered to be the best available at the time of preparation of this report, but when more data or new techniques are available, better results should be possible.

The results of this study should not be extrapolated to other desert basins.

COMPARISON WITH OTHER METHODS

The infiltration approach called the ϕ index (Linsley and others, 1958) gives a rough check on the results presented in the previous section. The ϕ index is based on the assumption that the rate of basin recharge is constant throughout a storm and thus the volume of rain greater than the ϕ index equals the volume of runoff. The 24-hour rainfall-frequency data were taken from the maps of the National Weather Service (1972) as follows:

$$P_2 = 1.2 \text{ in}$$
$$P_5 = 1.8 \text{ in}$$
$$P_{10} = 2.0 \text{ in}$$
$$P_{25} = 2.75 \text{ in}$$
$$P_{50} = 3.0 \text{ in}$$
$$P_{100} = 3.5 \text{ in}$$

These 24-hour rainfalls were proportioned according to the storm distribution recommended by the U.S. Soil Conservation Service (1957). This gave the following:

Time step	Inches of precipitation in each step					
	P_2	P_5	P_{10}	P_{25}	P_{50}	P_{100}
1	0.096	0.144	0.160	0.220	0.240	0.280
2	.180	.270	.300	.412	.450	.525
3	.564	.846	.940	1.292	1.410	1.645
4	.156	.234	.260	.358	.390	.455
5	.108	.162	.180	.248	.270	.315
6	.096	.144	.160	.220	.240	.280
Total	1.2	1.8	2.0	2.75	3.0	3.5

With the assumption that no runoff occurs to the playa for the 2-year storm, that is, total infiltration, the base ϕ index would be 0.564. Because the infiltration would be less and less for larger and larger storms (wetter antecedent conditions, longer storms), the ϕ index was reduced as follows:

$$\phi_2 = 0.564$$
$$\phi_5 = .56$$
$$\phi_{10} = .49$$
$$\phi_{25} = .40$$
$$\phi_{50} = .34$$
$$\phi_{100} = .25$$

The volume of rainfall left after subtracting the appropriate ϕ index and adjusting from inches to volume from 60 mi^2 (155 km^2) contributing area is as follows:

Time step	Runoff, in inches, in each step					
	V_2	V_5	V_{10}	V_{25}	V_{50}	V_{100}
1	-	-	-	-	-	-
2	-	-	-	0.012	0.110	0.245
3	-	0.28	0.45	.892	1.070	1.365
4	-	-	-	-	.050	.175
5	-	-	-	-	-	.035
6	-	-	-	-	-	-
Total inches	0	0.28	0.45	0.904	1.230	1.820
Total volume (acre-feet)	0	900	1,440	2,890	3,940	5,820

This gives a stage-frequency table as follows:

Recurrence interval (years)	Stage (ft above m.s.l.)
2	2,902.2
5	2,905.3
10	2,905.9
25	2,907.2
50	2,907.9
100	2,909.1

These stages differ from the results of table 6 by only 1.2 ft (0.37 m) for the lower stages and only 0.1 ft (0.030 m) for the 100-year stage. This gives an independent confirmation of the 100-year stage as determined in this study.

Obviously a different rainfall distribution would produce a different runoff, but the distribution used by the Soil Conservation Service has been used successfully by others and has no known bias.

DISCUSSION

Several assumptions and presuppositions are involved in the synthetic techniques of analysis used in this study. A basic presupposition is that channel-geometry techniques will provide a reasonable flood-discharge value. The studies previously mentioned and other studies currently under investigation have shown that channel geometry will indeed provide reasonable results and is an acceptable technique to use in this study. Concomitant with this acceptance is the assumption that flood peaks determined by using channel geometry can be used to develop a usable drainage area-peak discharge relation. This relation was used for all the areas where channel geometry was not or could not be measured.

Four other questions need to be considered as part of the analysis. These questions are:

1. How are the flood stages in the playa produced--by the general winter storms or the short-duration summer storms?

2. What is the areal extent of the storms, or how much of the basin is covered by a given storm?

3. How are the peak discharges related to flood volumes?

4. How is the peak or volume at a site related to the flood volume reaching the playa?

These four questions will be discussed in the following paragraphs.

The question of summer or winter storms is appropriate in that the two seasons produce differently shaped storm hydrographs. For the same-size peak discharge, a winter storm will quite often be of longer duration than a summer storm and therefore will have a larger flood volume. Channel geometry would give the flood peak, independent of whether the storm was summer or winter, because the data used to develop the channel-geometry relations included both summer and winter peaks (D. O. Moore, oral commun., 1973). This, unfortunately, would provide no information on the flood volumes involved.

A study of the few hydrographs available for the desert basins with small drainage areas indicates that for a given size peak a winter storm has about 3 to 5 times the volume of runoff that a summer storm has. However, this is counteracted by the fact that the peaks from summer storms are generally much greater than from winter storms, from 10 to 100 times greater. Table 7 shows the date and size of the peak discharge recorded at the two desert stations nearest to Apple Valley. Unfortunately these are peak stage only stations, so that no volumes are available.

The volumes used to develop the curve of figure 15 were all from summer storms, with either data for winter storms not available or no winter storms occurring during the short record available.

Of the 24 peaks recorded for the two stations, only six were for the winter storm period. Of the six winter peaks, five were less than 3 ft^3/s (0.085 m^3/s) and the highest was only 14 ft^3/s (0.40 m^3/s). Runoff volumes from these storms were small and were probably exceeded by at least 10 of the summer storms. Therefore, summer storms were considered the dominant system.

TABLE 7.--*Annual maximum peak discharge*

Station	Water year[1]	Date	Peak discharge (ft^3/s)
102618 Beacon Creek at	1959	Sept. 13, 1959	28
Helendale, Calif.	1960	Dec. 24, 1959	.5
	1961	Nov. 6, 1960	.1
Drainage area = 0.72 mi^2	1962	Dec. 2, 1961	.1
100-yr flood = 2,200 ft^3/s	1963	Sept. 19, 1963	5.3
	1964	Oct. 18, 1963	.3
	1965	Aug. 17, 1965	43
	1966	--	0
	1967	July 13, 1967	16
	1968	Aug. 7, 1968	[2]360
	1969	June 16, 1969	36
102626 Boom Creek near	1959	Sept. 13, 1959	36
Barstow, Calif.	1960	Sept. 1, 1960	[3]125
	1961	Aug. 22, 1961	15
Drainage area = 0.24 mi^2	1962	Dec. 2, 1961	.1
100-yr flood = 180 ft^3/s	1963	Sept. 19, 1963	1.8
	1964	Oct. 18, 1963	9.1
	1965	July 17, 1965	107
	1966	Nov. 23, 1966	14
	1967	July 15, 1967	17
	1968	June 7, 1968	33
	1969	Sept. 6, 1969	43
	1970	Aug. 26, 1970	33
	1971	--	0
	1972	Aug. 7, 1972	35
	1973	Feb. 14, 1973	2.7

[1]The water year is that period from October 1 of one year through September 30 of the following year and is designated by the calendar year in which it ends.

[2]About 20-yr flood.

[3]About 15-yr flood.

All the largest peaks at the 34 small-area stations in the deserts of southern California were during the July-September period for the records gathered since 1959.

Based on the above analysis, it is reasonable to assume that the summer storms cause the maximum flood peaks in Apple Valley. This should be even more likely for the larger storms that would cause a 100-year flood.

The second question of spatial coverage of thunderstorms is much more difficult to answer. Frontal winter storms undoubtedly could cover the entire 60 mi^2 (155 km^2) of contributing area for Apple Valley. Summer thunderstorms are known for their extremely local nature. It is common for a heavy and intense storm to occur over one basin, and yet less than 1 mi (1.6 km) away an adjacent basin would receive no rain.

The rainfall records for the deserts of California are much too scattered to provide any answers to this question. Radar images of the thunderstorm cells are about the only real data available on the spatial coverage of storms. Copies of the radar images were available for many of the summer thunderstorms during the summer of 1965. These images showed that cells large enough to cover the 60 mi^2 (155 km^2) contributing area of Apple Valley have occurred. The radar, unfortunately, does not show where, within the cell, rain is falling. Thus, even these data are inconclusive.

The greater the recurrence interval, the greater is the probability of a storm covering a larger part of the basin. For a storm that would produce the 100-year flood, it is probably not unreasonable to assume that virtually all the contributing area would be effective. The 100-year flood probably would be composed of some 10-year floods and some 200-year floods for different basins. The admittedly inaccurate assumption of a 100-year flood on all basins was made here for computational purposes. This assumption could possibly bias the results toward a higher stage.

The third and fourth questions are related and will be discussed together. The conversion from flood peaks to flood volume was made using the experience from the small-area program as a guide. This conversion was done after routing for two reasons: first, the inaccuracies in techniques did not justify determining a flood hydrograph at each site, and, second, because only peaks were available to calibrate the flood routing, the peaks rather than the volumes would have to be routed. Any flow-routing technique would require data to calibrate the coefficients, and a calibration using measured data was considered more reliable than modified data.

This study has shown that much more research is needed before these four questions can be answered.

In summary, the techniques used in the determination of the 100-year-flood stage in the playa are far from the final answer, but are at least reasonable. Other methods could have been used, but within the time and budget constraints of the project few alternative methods could be investigated more than superficially.

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APPENDIX A

DATA FOR CALIBRATION OF ROUTING EQUATIONS

Flow network: Upstream point[1]		Flow network: Point routed from	Routing distance (Di) (miles)	Discharge at upstream end[2] (Q) (ft^3/s)	Distance from divide (D) (miles)	General slope (S) (ft/mi)
			Fairview Valley Section			
1		--	0.25	1,470	1.8	235
2		--	.15	460	.75	148
3	A	--	.1	370	.65	174
4		1,2,3	.7	--	2.05	118
5		--	.25	420	.35	143
6		4,5	.1	--	2.75	115
7		--	.5	1,070	.9	273
8	B	--	.25	660	1.15	200
9	C	--	.25	910	1.3	174
10		6,7,8,9	.7	--	2.85	105
11	D	--	0	340	.8	200
12		10,11	.1	--	3.55	148
13	E	--	0	230	.65	190
14		12,13	.4	--	3.65	138
15		--	.25	480	.8	143
16	F	--	1.2	1,510	1.5	200
	G	14,15,16	.5	--	3.7	154
17		14,15,16	.9	--	4.1	128
18		--	.3	1,380	1.5	300
19		18	.1	--	1.8	160
20		--	.2	840	1.0	400
21		--	.35	800	.85	267
22		20,21	.15	--	1.35	138
23		--	.8	1,380	1.4	213
24		--	.5	860	.75	222
25		23,24	.2	--	2.15	167
26		19,22	.1	--	1.75	167
	H	25,26	.1	--	1.35	167
27		25,26	.6	--	1.85	167
28		--	.3	1,100	1.35	364

See footnotes at end of table.

Flow network			Routing distance	Discharge at upstream end[2]	Distance from divide	General slope
Upstream point[1]		Point routed from	(Di) (miles)	(Q) (ft^3/s)	(D) (miles)	(S) (ft/mi)
Fairview Valley Section--Continued						
29		-	.3	920	.7	286
	I	28,29	.1	-	1.05	138
30		28,29	.7	-	1.65	140
31		-	1.3	920	.95	224
32		27,30,31	.25	-	2.95	80
33		-	1.65	1,340	1.3	207
34		32,33	.2	-	3.2	80
35		17,34	.2	-	5.0	61
36		-	2.2	1,380	1.35	165
37		35,36	.1	-	5.2	113
38	J	-	.55	570	1.2	113
39		37,38	.2	-	5.3	59
40		-	.1	530	.9	115
41		39,40	.2	-	5.5	59
42		-	2.65	820	.55	180
43		41,42	.1	-	5.65	59
44		-	.65	500	.2	94
	K	43,44	.1	-	5.75	50
45		43,44	.25	-	5.7	44
46		-	.5	800	.35	113
47		45,46	.15	-	6.05	44
48	L	-	.4	650	1.2	103
49		47,48	.5	-	6.2	67
50	M	-	1.05	780	.8	236
	N	49,50	.1	-	6.35	62
51		49,50	.4	-	6.65	62
52		-	.6	1,030	.55	281
53		51,52	.5	-	7.15	67
54		-	.95	600	.55	114
55	P	53,54	.3	-	7.2	47
56		-	.95	970	1.3	224
57		55,56	.3	-	7.9	50
58		-	1.05	770	.75	98
59		57,58	.4	-	8.2	47
60		-	.3	800	1.8	267

See footnotes at end of table.

Flow network			Routing distance (Di) (miles)	Discharge at upstream end[2] (Q) (ft^3/s)	Distance from divide (D) (miles)	General slope (S) (ft/mi)
Upstream point[1]		Point routed from				
		Fairview Valley Section--Continued				
61	O	--	.5	650	1.0	314
62		60,61	1.1	--	2.1	117
63		--	1.3	600	.75	78
64		62,63	.3	--	3.2	51
65		59,64	.2	--	8.6	41
66		--	1.1	860	1.05	81
67		65,66	0	--	8.8	41
	R	67	--	--	--	--
		Northern Part of Apple Valley				
68	S	--	0.3	150	0.6	73
69	T	--	.15	600	2.3	65
70		68,69	.9	--	2.45	42
71	U	--	.75	450	.9	40
72		70,71	.55	--	3.40	75
73	V	--	.25	400	5.90	67
74		--	.2	50	.2	40
75		73,74	.2	--	6.15	50
76	W	75	.25	--	6.25	50
77		76	.35	--	6.35	50
78		--	.3	50	.2	40
79		77,78	.05	--	6.70	42
80		--	.3	200	.6	400
81		79,80	.3	--	6.75	42
82		72,81	.05	--	7.05	40
83	X	82	--	--	--	--

[1]Upstream point is the upstream end of routing reach, routed as defined in point routed from. Letters refer to site where channel geometry was measured (table 1 and fig. 11); numbers refer to points used in computation.

[2]Discharge is from data on channel geometry or figure 12.

APPENDIX B

DATA FOR FINAL RUN WITH ROUTING EQUATIONS

Flow network		Routing distance	Discharge at upstream end[2]	Distance from divide	General slope
Upstream point[1]	Point routed from	(Di) (miles)	(Q) (ft^3/s)	(D) (miles)	(S) (ft/mi)
101 R	-	1.55	2,400	8.8	41
102	-	2.2	880	.3	48
103	101,102	.9	-	10.3	6
104	-	.1	1,260	1.4	200
105	-	.1	1,240	1.1	200
106	104,105	2.15	-	1.55	190
107 2	-	1.8	1,060	1.4	100
108	106,107	.4	-	3.7	5
109 3	-	1.35	1,850	2.95	95
110 4	-	.65	1,150	2.65	174
111 5	-	1.3	330	1.05	175
112	110,111	.35	-	3.3	50
113 6	-	.7	280	1.4	175
114 7	-	.55	220	1.0	175
115	112,113,114	.9	-	3.65	40
116 8	-	.35	1,600	4.3	83
117 9	-	.2	360	1.5	95
118	116,117	1.15	-	4.65	75
119 10	-	1.05	850	1.6	130
120	118,119	.2	-	5.8	78
121 11	-	1.1	525	1.35	143
122	120,121	.35	-	5.95	75
123 12	-	.8	240	1.2	147
124 13	-	.75	310	2.0	145
125	123,124	.15	-	2.7	190
126 14	-	.4	380	1.6	195
127 15	-	.35	440	1.55	195
128	126,127	.35	-	2.05	117
129 16	-	.6	280	1.85	150
130	125,128,129	.25	-	2.85	110

See footnotes at end of table.

Flow network			Routing distance (Dl) (miles)	Discharge at upstream end[2] (Q) (ft^3/s)	Distance from divide (D) (miles)	General slope (S) (ft/mi)
Upstream point[1]		Point routed from				
131		122,130	2.25	–	6.3	5
132	X	–	.3	670	7.05	40
133		–	.4	310	.6	400
134		132,133	.3	–	7.35	40
135		–	.45	280	.6	375
136		134,135	0.65	–	7.65	40
137		–	.8	370	.7	350
138		136,137	.55	–	8.3	40
139	17	–	2.65	610	1.1	70
140		138,139	1.6	–	8.85	40
141	18	–	1.9	1,450	1.2	135
142		–	1.85	400	.35	210
143		141,142	.35	–	3.1	5
144		–	1.25	360	.4	300
145		–	1.2	340	.4	400
146		144,145	.7	–	1.65	15
147		143,146	.25	–	3.4	5
148		–	1.75	240	.25	410
149		147,148	.2	–	3.65	5
150		–	.95	290	.35	300
151		–	.7	280	.4	300
152		150,151	.75	–	1.25	25
153		–	1.2	320	.4	275
154		152,153	.75	–	2.05	10
155		149,154	.5	–	3.85	5
156		103,108,109, 115,131,140, 155	–	–	–	–

[1]Upstream point is the upstream end of routing reach, routed as defined in point routed from. Letters refer to site where channel geometry was measured (tables 1 and 3 and fig. 11), numbers refer to points used in computation.

[2]Discharge is from data on channel geometry or figure 12.

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