GILA RIVER BASIN

NEW RIVER AND PHOENIX CITY STREAMS

ARIZONA

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DESIGN MEMORANDUM NO. 2

HYDROLOGY

PART 1

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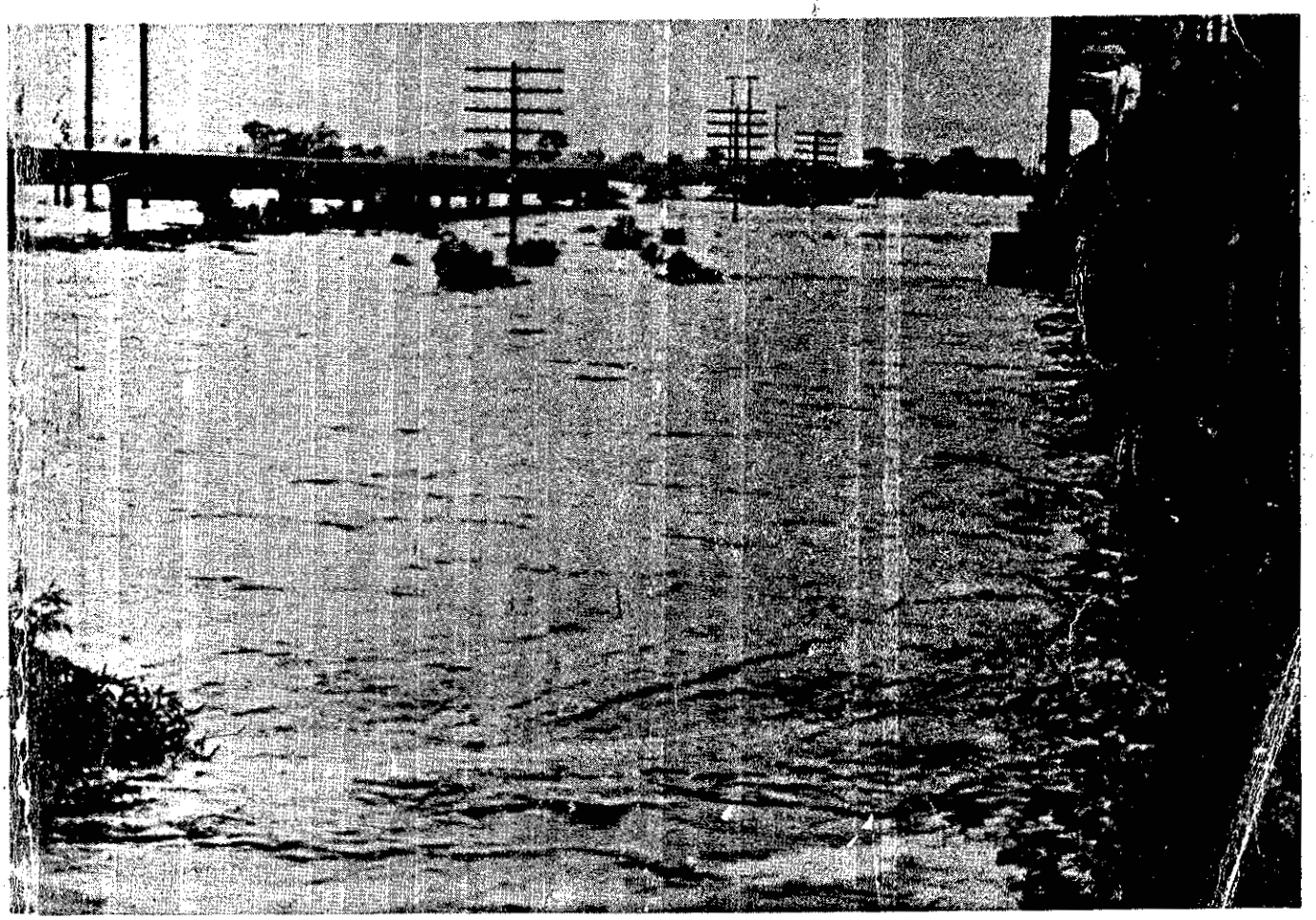

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DM-11

OCTOBER 1974

GILA RIVER BASIN
NEW RIVER
AND PHOENIX CITY STREAMS
ARIZONA

DESIGN MEMORANDUM NO. 2

HYDROLOGY

Part I

U.S. Army Engineer District, Los Angeles
Corps of Engineers

1974

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P. O. BOX 2711
LOS ANGELES, CALIFORNIA 90053

SPLED-WH 28 November 1973

SUBJECT: New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology Part I

Division Engineer, South Pacific
ATTN: SPDED-H

1. Inclosed are 15 copies of New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology Part I.

2. Approval of this report for flood control planning and design studies in Phoenix, Arizona and vicinity is requested.

FOR THE DISTRICT ENGINEER:

1 Incl (15 cys)
as stated

for GARTH A. FUQUAY
Chief, Engineering Division

SPDED-H (28 Nov 73) 1st Ind
SUBJECT: New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology Part I

DA, South Pacific Division, Corps of Engineers, 630 Sansome Street, Room 1216, San Francisco, California 94111 18 January 1974

TO: HQDA (DAEN-CWE) WASH DC 20314

Approval of the Hydrology Design Memorandum is recommended subject to comments in Inclosure 2.

FOR THE DIVISION ENGINEER:

JOHN W. GERHART
Chief, Engineering Division

2 Incl
wd 5 cys incl 1
Added 1 incl
2. Comments

DAEN-CWE-B (SPLED-WH, 28 Nov 73) 2nd Ind
SUBJECT: New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology Part I

DA, Office of the Chief of Engineers, Washington, D.C. 20314 14 March 1974

TO: Division Engineer, South Pacific, ATTN: SPDED-H

1. Approved, subject to the comments of the Division Engineer and to the comments in the following paragraphs.

2. The subject design memorandum should have included bound-in copies of the transmittal correspondence in accordance with paragraph 20b of ER 1110-2-1150.

3. Paragraph 4-05b. Determination of flood wave travel times should be checked by using observed flood hydrographs and correlated with unit-hydrograph time of concentration values.

4. Paragraph 6-03b. The assumption that the standard project flood would have a 200 to 500 year frequency should be varified. Comparing the frequency curves of several stream gages having long records with computed standard project flood peak flows at the same gage locations would be one procedure to check this assumption.

FOR THE CHIEF OF ENGINEERS:

1 Incl
wd incl 1

C. E. Slayton
for HOMER B. WILLIS
Chief, Engineering Division
Directorate of Civil Works

SPDED-H (28 Nov 73) 3d Ind
SUBJECT: New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology Part I

DA, South Pacific Division, Corps of Engineers, 630 Sansome Street Room 1216, San Francisco, California 94111 26 March 1974

TO: District Engineer, Los Angeles, ATTN: SPLED-WH

J. W. G.

1 Incl
2. nc

SPLED-HE (28 Nov 73) 4th Ind
SUBJECT: New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology, Part I

DA, Los Angeles District, Corps of Engineers 8 November 1974

TO: Division Engineer, South Pacific
ATTN: SPDED-H

1. Inclosed are ten (10) copies of New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology, Part I.

2. SPL concurs with all SPD comments presented in the first indorsement dated 18 January 1974 with the exception of comment No. 4. All other comments have been incorporated in the inclosed report.

3. In comment No. 4, the value of 0.3 acre-foot per square mile per year for the rate of sediment production at the proposed damsites in the Phoenix, Arizona area has not been revised.

a. In telephone conversations with Mr. Les Kister (USGS, Tucson, Arizona) and Mr. Tom Maddock, Jr. (USGS, Reston, Virginia), SPL was unable to obtain confirmation of a generalized estimate by the USGS of 0.5 acre-foot per square mile per year for the rate of sediment production in the semi-desert areas in Arizona and New Mexico. The range of 0.2 to 0.5 acre-foot per square mile per year was concluded as the best estimate of the proper range for the sediment production rate in the Phoenix, Arizona area.

b. Precipitation at Phoenix for the 47-year period (1923 through 1969) averages 7.11 inches. Precipitation at Phoenix for the 97-year period (1876 through 1969) averages 7.36 inches (94 years was used to compute the average since 3 years of record are incomplete). The percentage difference is 3.4%. The 47-year period and the long-term period are very close in terms of average precipitation.

c. The value of 0.3 acre-foot per square mile per year was determined from actual records at Cave Creek Dam over a 47-year period (1923 through 1969), and hence was taken as the best estimate of the sediment production rate in the study area.

4. SPL concurs with all OCE comments presented in the second indorsement dated 14 March 1974 with the exception of part of comment No. 3. All other comments have been incorporated in the inclosed report.

SPLED-HE 8 November 1974

SUBJECT: New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology, Part I

5. With regard to comment No. 3, observed flood hydrograph travel times for the flood of 1970 are compared in Table 5 with travel times used for the present without project conditions.

6. Approval of the inclosed report for flood control planning and design studies in Phoenix, Arizona and vicinity is requested.

FOR THE DISTRICT ENGINEER:

for Walter Raketich
GARTH A. FUQUAY
Chief, Engineering Division

2 Incl
Added 1 incl (10 cys)
3. Report

SPDED-H (28 Nov 73) 5th Ind

DA, South Pacific Division, Corps of Engineers, 630 Sansome Street Room 1216, San Francisco, California 94111 5 Feb 1975

TO: HQDA (DAEN-CWE-Y) WASH DC 20314

1. The subject report has been revised to satisfy comments in the 1st and 2nd indorsements.

2. Subsequent to receipt in the Division office paragraph 6-02 "Graphical Frequency Analysis" has been revised and frequency curves added to support the conclusion of .2 to 5. per cent exceedence frequency for the Standard Project Flood.

FOR THE DIVISION ENGINEER:

for A. L. O'Neill
A.E. WANKET
Chief, Engineering Division

1 Incl
wd 2 cys

DAEN-CWE-B (SPLED-WH, 28 Nov 73) 6th Ind
SUBJECT: New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology Part I

DA, Office of the Chief of Engineers, Washington, D.C. 20314 6 March 1975

TO: Division Engineer, South Pacific, ATTN: SPDED-H

Information furnished in the 4th and 5th indorsements is satisfactory.

FOR THE CHIEF OF ENGINEERS:

for R. J. Thorpe
HOMER B. WILLIS
Chief, Engineering Division
Directorate of Civil Works

1 Incl
wd Incl 1

ED-H 7th

13 Mar 75

Dist Engr, Los Angeles
ATTN: SPLED ~~WH~~ HL

for A. E. W.

1 Incl nc

DESIGN MEMORANDUMS PREVIOUSLY ISSUED

No.	Title	Date	Date Approved by OCE
1	Feature Design for Dreamy Draw Dam	January 1972	May 1972

CURRENTLY SCHEDULED FOR FUTURE ISSUANCE

No.	Title	Scheduled Date
3	Phase I General Design Memorandum, Project Formulation	January 1975
4	Feature Design Memorandum For Cave Buttes Dam	January 1975
5	Master Plan For Recreation, Cave Buttes Dam	January 1975

REVISIONS

Date	New Pages or Drawings	Date Approved by OCE

SPDED-H 18 January 1974

COMMENTS

GILA RIVER BASIN
NEW RIVER & PHOENIX CITY STREAMS

D. M. NO. 2 - HYDROLOGY

1. Page 6 - Para 2-05 c(1)

Delete 7th sentence. The heat to melt snow from rainfall is more than offset by cloudiness. Heat from radiation and the warm moist tropical air associated with the storm provides the increase in melt.

2. Page 14 - Para 4-03 c

It is not clear how curves are chosen. Relationship of Plates 17 and 18 are not clear; explain.

3. Page 20 - Para 4-05

Present the results of analysis of 1970 flows to support choice of Muskingum values.

4. Page 28 - Para 7-01

The value of .3 acre-feet per square mile per year appears low. The USGS has made a generalized estimate of .5 acre-feet per square mile per year for the semi-desert areas in Arizona and New Mexico. Precipitation during the 47-year period is lower than the long term normal.

5. Page 39 - Table 6

Reference to Plate 31 for location of sub-areas.

6. Page 40 - Table 7

Reference to map for identification of reaches.

7. Pages 44 & 45 - Tables 11 & 12

Show comparison of computed to actual flows in table or graph.

8. Page 50 - Table 15

Reference to Plate 54

9. Plates 5 & 6

Add scale. This map should be the same scale as watershed map.

10. Plates 7 & 8

Add scale.

11. Plates 9 through 13

The storm isohyetal maps should be to the same scale so a visual comparison of storms can be made.

12. Plate 14

The intensity duration curve for area number 7, Queen Creek, should be added since it is used as the SPF.

13. Plate 15

The scale should be the same scale as the storm maps.

14. Plates 17 & 18

Reference the parameter curves on Plate 18 to Plate 17.

15. Plate 19

The scale should be the same as the storm maps.

HYDROLOGY DESIGN MEMORANDUM
NEW RIVER AND PHOENIX CITY STREAMS

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HYDROLOGY DESIGN MEMORANDUM
NEW RIVER AND PHOENIX CITY STREAMS

I - INTRODUCTION

1-01. PURPOSE AND SCOPE. This report is in support of phase 2 GDM studies on Phoenix, Arizona and vicinity. A thorough investigation of the runoff process and compilation of hydrologic data was made in preparing this study. There are four objectives of this report: (a) to present the basic meteorologic and hydrologic characteristics of the study region; (b) to outline the methods and techniques used to model the runoff process; (c) to present discharge frequency values for the present and future no-project (natural) conditions; and (d) to provide standard project flood, 100-year, and 50-year design discharges for the authorized flood control plan. Throughout this report the phrase "present conditions" refers to the project year 1 (1977), and likewise "future conditions" refers to project year 100 (2077).

1-02. PREVIOUS REPORTS. Flood control improvements in Phoenix have been made by numerous Federal, County, State, and City agencies as well as local organizations in the region. Appendix 1 lists references with material of hydrologic content, and Appendix 2 lists reports concerning authorized or completed projects in the study area.

1-03. AUTHORIZED PLAN OF IMPROVEMENT. The authorized plan for flood control (See pl. 1) comprises: (a) four dams, one each on Cave Creek, Skunk Creek, New River, and Dreamy Draw; (b) the Union Hills and Arizona Canal diversion channels; and (c) the Cave Creek, Dreamy Draw, Skunk Creek, New River, and Agua Fria River channel improvements. The plan provides for controlling floodflows in each respective drainage area; for diverting residual flows in Cave Creek and several small washes to Skunk Creek, and for improving Skunk Creek, New River, and the Agua Fria River to carry these flows to an adequate point of disposal in the Gila River. Pertinent information on each unit of the improvements under the authorized plan is given in the following subparagraphs:

a. Cave Buttes Detention Basin. The Cave Buttes detention basin (with two dikes) would be constructed about 2 miles downstream (south) from the existing Cave Creek Dam and would control runoff from 195 square miles of drainage area. The main embankment would be compacted earthfill structures with a maximum height of about 120 feet above streambed. The outlet works would consist of a 12-foot-diameter ungated concrete conduit located under the main embankment and capable of discharging 5,400 cubic feet per second with the water surface at the spillway crest.

b. Cave Creek Channel. The Cave Creek channel would be a trapezoidal, concrete-lined channel about 3.6 miles long. It would extend from the outlet of the proposed Cave Buttes Dam to the proposed Union Hills diversion channel.

c. Union Hills Diversion Channel. The Union Hills diversion channel would be an entrenched trapezoidal concrete-lined channel about 9-3/4 miles long extending from the divide between Cave Creek and Indian Bend Wash drainage areas, near 40th Street, to Skunk Creek.

d. Dreamy Draw Detention Basin. Dreamy Draw detention basin, completed in July 1973, is the first increment of the proposed flood control construction program for Phoenix. "Design Memorandum No. 1, Feature Design for Dreamy Draw Dam, New River and Phoenix City Streams, Arizona" dated January 1973, presents complete design details of this project. Briefly, Dreamy Draw Dam is a compacted-earthfill dam in Dreamy Draw just south of Northern Avenue (formerly known as Shea Boulevard) and about one mile east of 16th Street. The main embankment with crest elevation at elevation 1420 feet is 480 feet long and about 50 feet above streambed. A detached unlined spillway is excavated in rock at the left abutment of the main embankment. The dam provides a capacity of 317 acre-feet, of which 36 acre-feet is allocated for sedimentation and 281 acre-feet would be allocated for flood control. It is designed to reduce standard project flood with a peak inflow of 3,600 cubic feet per second to an outflow of 196 cubic feet per second.

e. Dreamy Draw Channel. The Dreamy Draw channel would be an entrenched rectangular-concrete channel extending from the Dreamy Draw detention basin outlet to the Arizona Canal diversion channel – a length of about 3-1/2 miles.

f. Arizona Canal Diversion Channel. The Arizona Canal diversion channel would be just upstream from the Arizona Canal and would be nearly parallel to that canal. The upstream end consists of a rectangular-concrete channel, 2 miles in length, extending from a point just west of 12th Street to Central Avenue. The downstream end, 10 miles in length, would be a trapezoidal earth section extending from Central Avenue to Skunk Creek. A reinforced concrete transition channel and a side-channel spillway structure would be constructed at the Skunk Creek channel to assure proper confluence of the two flows.

g. Adobe Detention Basin. The Adobe detention basin would be constructed on an unnamed tributary of Skunk Creek, about seven miles north of Bell Road and about 1 mile west of the Black Canyon Highway. The embankment would be a compacted-earthfill structure with a maximum height of about 76 feet above streambed. The outlet would be an eight foot diameter ungated reinforced-concrete conduit located near the right abutment. The capacity of the outlet would be 2,000 cubic feet per second with the water surface at the spillway crest.

h. A diversion channel about 2-1/2 miles long would be excavated to divert a standard project flood on Skunk Creek to the Adobe detention basin area. The channel would be located with its downstream end about 5,000 feet north of the left abutment of the main embankment.

i. Skunk Creek Channel. The Skunk Creek concrete-lined trapezoidal channel would be constructed along Skunk Creek from a point just upstream from the outlet of the Union Hills diversion channel downstream to the confluence with the New River, a distance of about 6-1/2 miles. The inlet for the channel would consist of a concrete-lined transition. Two wing levees, each about 1,500 feet long, with stone facing and extending to high ground on the left and right banks, would train the flow to the transition. Downstream from the Arizona Canal confluence, the channel would end in a stone-lined transition that would act as an energy dissipator to reduce the velocity of flows entering the New River channel.

j. New River Detention Basin. The New River detention basin would be constructed on the New River about 8 miles upstream from the confluence with Skunk Creek. The main embankment would be a compacted-earthfill structure with a maximum height of about 80 feet above streambed. The outlet would be a 6-foot-diameter ungated reinforced-concrete conduit capable of discharging 1,000 cubic feet per second with the water surface at the spillway crest. An earthfill dike, about 7,000 feet northwest of the right abutment of the main embankment, would be required along the west edge of the detention basin area to confine the design flood.

k. New River Channel Improvement. The improved channel for the New River would extend from the mouth of Skunk Creek downstream to the confluence with the Agua Fria River, a distance of about 8 miles. The trapezoidal channel, which would be excavated in earth, would have revetted side slopes.

l. Agua Fria Channel Improvement. An excavated earth channel would be constructed in the Agua Fria River for a distance of 7-1/2 miles from the mouth of the New River to a point about 2 miles downstream (south) from the bridge for U.S. Highway No. 80. The trapezoidal earth-bottom channel would have rock-revetted side slopes.

II – GENERAL DESCRIPTION OF DRAINAGE AREA

2-01. PHYSIOGRAPHY AND TOPOGRAPHY. Phoenix, Arizona is located within the Gila River basin, which is the largest drainage area tributary to the lower Colorado River. The area pertinent to flood problems in Phoenix and vicinity is in Maricopa and Yavapai Counties in the central part of Arizona (See pls. 2 and 3), and comprises approximately 2,610 square miles. The area is roughly oval in shape with a maximum length and width of approximately 90 and 45 miles respectively. Approximately 70 percent of the basin is mountainous and the remaining 30 percent is valley. The mountain areas above 3,000 feet in elevation are characterized by rugged terrain and steep gradients. The lower areas consist of fairly flat valley land with regular alluvial slopes. Plate 4 presents streambed profiles for streams in the study area. Natural vegetation as well as land used for agriculture are rapidly being replaced by residential, commercial, and industrial development in the valley areas. Urbanization can significantly increase the runoff potential of a watershed. Maps showing percentage impervious cover for the present (1977) and future (2077) conditions are presented on plates 5 and 6 respectively. A description of the physical characteristics of the major watercourses considered in this report follows:

a. Agua Fria River. The Agua Fria River originates about 7,000 feet above sea level in the mountains of central Arizona (See pls. 2 and 3) and flows southward for about 130 miles before emptying into the Gila River 15 miles west of downtown Phoenix at elevation 910 feet. The course of the stream is nearly equidistant between two parallel mountain ranges, the Black Hills - New River Mountains and the Bradshaw Mountains, that form the eastern and western boundaries respectively, of the drainage area. Lake Pleasant Dam, located in the Hieroglyphic Mountains controls 1,457 square miles of drainage. The gradient of the Agua River ranges from about 300 feet per mile in the headwaters to about 70 feet per mile at the canyon mouth and to about 10 feet per mile at the Gila River. Although the Agua Fria River has the largest drainage size in the study area, flood control improvements are being considered only on that part of the Agua Fria River below the confluence with New River, a distance of about 10 miles.

b. New River. New River, the major tributary of the Agua Fria River, has its headwaters in the New River Mountains, roughly 40 miles north of Phoenix. (See pls. 2 and 3.) New River flows generally southward for about 40 miles to its confluence with the Agua Fria River, about 15 miles west of Phoenix. The drainage area of New River at its mouth is 340 square miles, of which approximately one third is mountainous. Elevations in the basin range from a little over 5,000 feet in the New River Mountains to about 1,040 feet at the confluence with the Agua Fria River. Stream gradients decrease from 370 feet per mile in the mountains to 10 feet per mile in the valley.

c. Skunk Creek. Skunk Creek, the major tributary of New River, rises in the New River Mountains about 35 miles north of Phoenix and flows generally southwestward for about 30 miles to its confluence with the New River about 15 miles northwest of Phoenix. (See pls. 2 and 3.) Only about 20 percent of the 110 square miles at the mouth is mountainous.

d. Cave Creek. Cave Creek has its source in the New River Mountains to the north of Phoenix where elevations rise to as high as 5,000 feet. (See pls. 2 and 3.) The stream then descends to the alluvial fan near the town of Cave Creek and flows south for 13 miles before encountering Cave Creek Dam, which controls 175 square miles of drainage area. Cave Creek then flows through an alluvial fan which is undergoing urbanization between Cave Creek Dam and the Arizona Canal. Floodflows exceeding the freeboard capacity of Arizona Canal flow directly through metropolitan Phoenix to the Salt River. The drainage area of Cave Creek at the Salt River is 311 square miles.

e. Dreamy Draw. Dreamy Draw, a tributary of Cave Creek rises in the Phoenix Mountains and flows generally southeastward for about 5 miles to its confluence with Cave Creek in Phoenix. Dreamy Draw Dam controls 1.26 square miles of the 2.0 square miles of drainage area above the Arizona Canal.

2-02. RUNOFF CHARACTERISTICS. None of the watercourses in the Phoenix area flow perennially. Generally runoff occurs only during and immediately following the heavier precipitation because climatic and drainage area characteristics are not conducive to continuous runoff. Significant runoff occurs in the summer months (June through September) as a result of summer local storms and to a lesser degree general storms. Even the major rivers, which flowed year round in their pristine state, have been dried up by dams and diversions. Stream channels are distinct and well defined in the mountain ranges, but upon reaching the valley transition they become braided and poorly defined channels which carry the bulk of large floodflows in their wide flat overbanks.

2-03. VEGETATION. In general, the vegetation is sparse. Cacti grow throughout the area along with other desert shrubs on the fairly level areas at the lower elevations. A few stunted trees including Juniper, Paloverde, Mesquite, Iron Wood and Scrub Oak exist among the shrubs. The vegetation tends to be thicker along and adjacent to the stream courses. Perennial grasses form a very small portion of the vegetation, but a good cover of annual grasses occur after the winter rains.

2-04. GEOLOGY. The rock types encountered in the mountainous areas within the study area are very similar. The basement complex is composed predominantly of Precambrian schistose and massive metaigneous rocks with lesser amounts of gneiss and quartzite. These are overlain with and intruded by igneous rocks. The igneous rocks consist of granites, rhyolite, andesite, flows of vesicular basalt, tuff, and tuffaceous agglomerate.

Alluvium fills the valleys and covers the slopes of hills and mountains. Older alluvium consists of medium to well-cemented residual soil and talus debris. It is generally found along the side slopes of the valleys and underlying the Recent alluvium. In the valleys, the Older alluvium is mostly sand and silty sand containing varying amounts of caliche. Recent alluvium is found in the valley areas along the streambed channels and consists of uncemented silts, sands, gravels, cobbles, and boulders. Rocks similar to those of the hills and mountains probably underlie the alluvial deposits. The deep dissection of the valleys in the mountains and the great extent of the alluvial fans suggest that the Phoenix area has had a long stable history. There is evidence of ancient folding and faulting, as seen in the outcrops of the older rocks, but no recent seismic activity has been recorded in the area. In general, sediment production would be classified moderate, with a higher potential at points near the mountains and a correspondingly lesser potential in the valley areas with flatter streambed profiles.

2-05. CLIMATOLOGY.

a. The climate of Phoenix and vicinity ranges from warm and arid over the desert floor to cool and moderately humid in the mountainous portions of the basin. Mean maximum/minimum January temperatures range from approximately 65/35 degrees Fahrenheit in the valleys (64.0/35.3 at Phoenix Weather Bureau Airport) to about 50/25 in the mountains. Mean Maximum/minimum daily temperatures during July vary from 105/75 in the lower portions of the region (104.6/75.0 at Phoenix Weather Bureau Airport) to 90/60 in the higher mountains. The extreme temperature experienced in the region ranges from 120 degrees in portions of the lower desert to near zero in some of the more remote mountain areas. Prevailing winds are generally rather light. However, moderate winds are often observed in conjunction with general winter storms, particularly in the higher elevations, and moderate winds frequently occur in spring when low pressure develops over Nevada. Summer thunderstorms often produce strong gusty winds over local areas.

b. Mean annual precipitation in the basin ranges from around 7 inches in the area south and west of Phoenix to more than 22 inches in the New River Mountains. (See pl. 2.) The average annual precipitation for the entire drainage area is 11.40 inches, of which 4.37 inches (38.3 percent) falls during the summer months of June through September, and the greatest portion of the remainder falls during the period December through March. Much of the winter precipitation falls as snow at elevations above 6,000 feet, and snow can occur at times over the entire basin, although snow below 2,000 feet is very rare. There is considerable year-to-year variability in the individual monthly, seasonal, and annual precipitation amounts which fall in the vicinity of Phoenix, Arizona. Some of the drier months of the year have at times experienced more than ten times the normal precipitation, and each month has passed at least once during the 20th century with no measurable precipitation reported at some stations.

c. There are three basic types of storms which can affect the Phoenix area, although some individual storms may consist of a combination of types.

(1) General winter storms. (a) Storms of this general type normally occur over the southwestern United States as extratropical cyclones, move inland from the Pacific Ocean spreading light to moderate precipitation over large areas, and last from a half day to several days. General winter storms are usually most prevalent and most intense during the months of December through March, although they can occur any time from October through May, and occasionally in combination with other types of storms during the summer months. This type of storm is characterized most typically by cool, stable air masses with widespread overcast stratiform cloudiness and steady, light rain or snow. A few locally heavy showers and occasional isolated thunderstorms may occur, however. Winter precipitation in the southwestern United States normally occurs as the result of the influx of moisture from the tropical Pacific Ocean south of Baja California, low-level convergence and rising motion, warm or cold fronts, orographic uplift and instability. Despite the relatively low intensities of rainfall, the large areal extent and the relatively long durations of this type of storm can produce significant volumes of runoff and even peak flows on the larger rivers of the region. (b) Significant historical storms of the general winter type which have affected central and south-central Arizona, including Phoenix and vicinity, include those of February-March 1884, 21-25 February 1891, 25-28 November 1905, 17-24 December 1914, 15-21 January 1916, 25-30 January 1916, 6-8 February 1937, 27 February - 5 March 1938, 11-17 March 1941, 31 October 1957, 9-11 December 1965, 13-20 December 1967, and 18-20 October 1972 (with the storms of October 1957 and October 1972 exhibiting some summer type characteristics). The storms of February-March 1884, 21-25 February 1891, and 15-21 January 1916 produced the largest known general winter floods in the greater vicinity of Phoenix, although available records for these events, especially for those prior to 1916, are relatively scanty.

(2) General summer storms. (a) These storms usually consist of general rains of a convergence, frontal, and/or orographic nature, with moderate to heavy thunderstorms often superimposed. General summer storms occur primarily during the months of July, August and September, although it is possible for this type of storm to occur any time from May through October (often with some of the characteristics of general winter storms, especially during the latter portions of the greater summer season). Cloudiness in this type of storm is dominated by the convective types: cumulus and cumulonimbus, although considerable stratiform cloudiness is often present as well. Rainfall normally consists of a mixture of general steady rain and numerous convective showers with locally heavy precipitation associated with convective activity. The convective type usually accounts for the bulk of a general summer storm's total rainfall. The major meteorological prerequisite for general summer storms is a strong, deep flow of tropical moisture from the Pacific Ocean (including the Gulf of California) or the Gulf of Mexico, or both. This flow of tropical moisture is often enhanced considerably by the presence of one or more tropical storms or hurricanes off the west and/or east coasts of northern or central Mexico, or by the remnants of such a tropical cyclone which moves across land and directly invades the Phoenix area. The triggering mechanisms for the release of this tropical moisture include solar heating, orographic uplift, the movement into the region of a cold front or upper level closed low from the north or northwest, and other mechanisms causing general low-level convergence and/or instability. (b) Significant general summer storms which have occurred in the study area include those of 1-2 July 1911, 4-5 October 1911, 7-10 September 1916, 20-21 August 1921, 4-5 October 1925, 7-8 August 1930, 31 July - 6 August 1931, 22-24 August 1935, 24-26 July 1936, 4-6 September 1939, 1-3 August 1943, 23-26 September 1943, 17-19 September 1946, 26-29 August 1951, 22-25 July 1955, 22-24 August 1955, 4-6 September 1970 and 6-8 October 1972 (with the storms of July 1911, October 1911, September 1916, October 1925, August 1930, September 1943, September 1946, and September 1970 exhibiting some winter type characteristics).

(3) Summer thunderstorms. (a) Thunderstorms can occur in the vicinity of Phoenix, Arizona at any time of the year, but the most common and the most significant thunderstorms occur during the summer months, usually between late June and late September. These summer thunderstorms result from the invasion of tropical moisture orginating in the Gulf of California, the Pacific Ocean southwest of Baja California, and on occasion the Gulf of Mexico, and are normally triggered by the instability resulting from the intense solar heating of the ground surface which occurs at this time of the year. Conditions favoring these air mass thunderstorms may be further intensified by some of the meteorological factors listed in the previous section (General summer storms). Summer thunderstorms are normally scattered or isolated phenomena, and are more than twice as common over the higher mountain peaks as they are in the desert valleys. The most severe of these thunderstorms, however, appear to have little preference for either high or low-elevation areas. Heavy thunderstorms, sometimes referred to as "cloudbursts," can produce severe flash floods over small drainage basins, resulting in serious local damage and sometimes loss of life. (b) The most severe historical example of a heavy air mass summer thunderstorm occurring within the State of Arizona is the Queen Creek thunderstorm of 19 August 1954. This event is discussed in detail in paragraph 3-03d. There have been numerous other heavy thunderstorms in that part of the southwestern United States during this century, including some very heavy thunderstorm precipitation in the general vicinity of Phoenix which occurred in August 1963, August 1964, July 1967, September 1969, June 1972, and in conjunction with the general summer storms of August 1921, August 1930, August 1931, August 1935, July 1936, September 1939, August 1943, September 1946, August 1951, July 1955, and September 1970. There was also some heavy thunderstorm rainfall in the Kingman-Wikieup area of western Arizona and the Imperial Valley of California in association with the tropical storms of 3-8 September 1939, and an exceptionally heavy thunderstorm in Indio, California on 24 September 1939, which also occurred in conjunction with a tropical storm.

2-06. EXISTING STRUCTURES AFFECTING RUNOFF

a. Existing structures affecting runoff in the area include several dams and irrigation canals. (See pls 1 - 3.) The Cave Creek Flood Control Dam (completed in 1923), which is under the jurisdiction of the Salt River Valley Water Users Association, is about 18 miles north of downtown Phoenix. Cave Creek Dam controls runoff from 175 square miles of drainage area on Cave Creek. Reservoir storage capacity to the top of dam (elevation 1,642 feet) is approximately 12,400 acre feet. The spillway, a natural saddle, is in the hills to the east of the dam and has a crest elevation of 1,638 feet. Spillway capacity at top of dam is approximately 4,600 c.f.s. Lake Pleasant Dam (completed in 1927), which is under the jurisdiction of the Maricopa County Municipal Water Conservation District No. 1, is about 30 miles north-northwest of downtown Phoenix. Lake Pleasant Dam controls runoff from 1,457 square miles of drainage area on the Agua Fria River. The present reservoir capacity is 157,000 acre-feet. Spillway flow is controlled by a series of Tainter gates which can pass an estimated 105,000 c.f.s. McMicken Dam (completed in 1956), which was constructed by the Corps of Engineers about 25 miles northwest of downtown Phoenix, is also under the jurisdiction of Maricopa County Municipal Water Conservation District No. 1. McMicken Dam controls runoff from 238 square miles of drainage area tributary to the Agua Fria River. Reservoir capacity at spillway crest (1,354 feet) was approximately 19,300 acre-feet when constructed. No recent surveys have been made to determine its present storage capacity.

b. There are several irrigation canals with levees in the Phoenix area under the jurisdiction of Salt River Valley Water Users Association, that were constructed in the early 1900's. The Arizona and Grand Canals flow through the City of Phoenix, north of and generally parallel to the Salt River. Their capacities in the vicinity of Cave Creek are approximately 800 and 600 c.f.s. respectively. The irrigation canals intercept low flows using available freeboard capacity to convey the additional water. Larger floodflows cause ponding behind the upstream canal levee with subsequent breaks in the canal. This form of flooding of areas downstream of irrigation canals is a major contributor to flood damages in the Phoenix area. The irrigation canals have little effect on very large discharges emanating from major watercourses.

2-07. PROPOSED IMPROVEMENTS AFFECTING RUNOFF.

a. A channel to control floodflows originating north of Interstate Highway 10 is proposed by the Arizona State Highway Department to run east-west across the Phoenix Valley adjacent to McDowell Road and to carry runoff to the Agua Fria River near McDowell Road. (See pl. 1.)

b. An updated alinement of the Glendale-Maryvale channel proposed in phase C of the comprehensive flood control plan presented in the Survey Report for Flood Control, Phoenix, Arizona and vicinity (including New River) is shown on plate 1. The channel was assumed to be a rectangular-concrete channel with a cross section capable of carrying the standard project flood from the contributing area.

c. The Central Arizona Project (CAP) is an authorized project under the jurisdiction of the U.S. Bureau of Reclamation which will provide Colorado River water to the southeastern portion of Arizona. The CAP consists of a dike several feet high upstream of the canal, with cross drainage structures (culverts and overchutes) designed to pass a 50-year flood peak determined by procedures detailed in the Soil Conservation Service publication "National Engineering Handbook". Plate 7 shows the location and capacity of the proposed levee and cross drainage structures. The canal crosses major watercourses by siphoning under them. Since the amount of storage created behind the canal dike is small, the influence of the small reservoirs was neglected in the computation of standard project floods. However, to the east of the study area in the Paradise Valley reach of CAP from 32nd Street to the McDowell Mountains, the USBR is planning to construct a 28 foot high dike capable of storing all floodwaters emanating from upstream. In summary, the CAP was found to have a negligible influence on flood computations along the major watercourses in the Phoenix area. Flood potential on small streams and swales which are intersected by the CAP would be significantly altered by the USBR project.

2-08. BREAKOUTS. There exist two critical sections in the Phoenix Valley area where floodwaters breakout from their main watercourse and flow overland. One of the sections occurs at the confluence of Skunk Creek and Scatter Wash immediately adjacent to the southerly end of Hedgpeth Hills and has a channel capacity of 60,000 c.f.s. The other section is on Cave Creek approximately 2,000 feet downstream of Cactus Road where channel capacity was determined to be 50,000 c.f.s. Floodwaters exceeding the channel capacity at these points, travel overland as sheet flow toward the Salt River.

III – PRECIPITATION AND RUNOFF

3-01. PRECIPITATION RECORDS. The U.S. Geological Survey (USGS) operates 71 precipitation gages, of which 27 are recording gages, in and near Maricopa County. The U.S. Weather Bureau operates an additional 55 precipitation stations in and near Maricopa County. (See pl. 8 and table 1.) The longest record in the study area is for the Phoenix Post Office which has 97 years of record during the period 1876 to 1972 and a recording gage record beginning in 1901. The 30-year mean annual precipitation as determined by the National Weather Service is shown on plate 2.

3-02. STREAMFLOW RECORDS. Since water year 1961, the USGS has established a fairly extensive stream gage system in the Phoenix area. Table 2 lists the stream gages in the immediate Phoenix vicinity. Plate 8 shows the location of stream gages in the Phoenix area.

3-03. STORMS AND FLOODS OF RECORD. Historical accounts indicate that damaging floods have occurred in the Gila River Basin. Sizeable floods were produced by the general storms of February 1884, February 1891, January 1916, and February-March 1938, but available records and estimates of severity are insufficient for detailed analysis for the study area. General winter storms may cause flooding in the study area, but the most severe floods generally occur during the summer months as a result of local thunderstorms. Severe local storms and floods occurred in the Phoenix area in 1921, 1935, 1936, 1939, 1943, 1951, 1955, 1956, 1957, 1963, 1964, 1967, 1969, 1970 and 1972. Brief descriptions of the storms and floods of January 1916, August 3, 1943, August 26-29, 1951, September 25-26, 1962, December 13-21, 1967, September 4-6, 1970, and June 21-22, 1972, together with the August 19, 1954 storm (southeast of Phoenix) which was used to develop the standard project flood hydrology, are given in the following subparagraphs:

a. Storms and Floods of January 1916. Two general winter storms occurred over the Gila River Basin in January 1916. The first storm period extended from January 14-21, and the second from January 25-30. Both storms originated over the Pacific ocean. The heaviest rainfall in each storm was centered in the area north of Roosevelt Reservoir, with secondary centers in the Pinal and Santa Catalina Mountains. The second storm had another secondary heavy rainfall center in the area tributary to the Agua Fria and the Hassayampa Rivers. The first storm, which was of broader areal extent than the second, produced the larger flood. The isohyetal map on plate 9, shows the areal distribution of precipitation that occurred between January 14-21. Observed total precipitation at Phoenix for the two storms was only 2.07 inches. The ground surface was conditioned for runoff owing to the occurrence of the precipitation on January 10-12, and to the presence of snow cover over much of the mountain area. On the Salt River near Roosevelt, 17 miles upstream from Roosevelt Dam (drainage area, 4,310 sq. mi.), the peak discharge of the first flood was estimated at 100,000 cubic feet per second. The maximum discharge of the second flood on the Agua Fria River at Lake Pleasant Dam, (drainage area, 1,457 sq. mi.) was 105,000 cubic feet per second.

b. Storm and Flood of August 3, 1943. The August 3, 1943 flood was caused by heavy precipitation resulting from thunderstorms over the desert areas north and east of Phoenix. Storm conditions started on August 1. Late on August 2, at Tempe, 2.11 inches were recorded in 30 minutes. Heavy precipitation occurred early on August 3. The total precipitation for August 3, was 2.12 inches at Phoenix, 2.99 inches at Phoenix airport, 3.50 inches at Tempe, and 2.63 inches at Granite Reef Dam. It is likely that more rain than this

fell in the desert areas to the north, but no records are available. Runoff was heavy upstream of Arizona Canal. A series of 22 breaks occurred in the south bank levee of the canal in the vicinity of Indian Bend Wash. A break in the south bank of the Arizona Canal in the Cave Creek area released water that caused nine breaks in the Grand Canal. The total peak inflow into the Arizona Canal was estimated at 30,000 cubic feet per second and Cave Creek upstream of Arizona Canal was estimated at 9,000 cubic feet per second. The maximum peak discharge in Indian Bend Wash at Arizona Canal was estimated at 15,000 cubic feet per second.

c. Storm and Flood of August 26-29, 1951. A tropical hurricane entered the mainland of Mexico from the east in the vicinity of Tampico on August 22. Moist air associated with this storm crossed Mexico to the eastern coast of the Gulf of California. This moist air augmented by moisture outflow from a tropical storm on the west side of Mexico began flowing into southwestern Arizona during the 26th, mostly in the vicinity of Organ Pipe Cactus National Monument. By the morning of the 27th, precipitation had become quite general over southern and central Arizona. Heavy precipitation spread northward and northeastward to the northern border of Arizona by the 29th. Precipitation was moderate to heavy from the 27th through the 29th. The storm was the most severe east and north of Phoenix. The total storm precipitation at Phoenix was 3.85 inches. Heaviest precipitation for the period was 13.35 inches at Crown King and 12.11 inches at Sunflower. About 65 percent of the total rainfall occurred during the maximum 24 hour period. The isohyets of the total storm precipitation are shown on plate 10. An estimate by the U.S. Soil Conservation Service, based on high water marks at numerous breaks of the Beardsley Canal in the Trilby Wash area (about 25 miles northwest of Phoenix) indicated a total peak discharge of 35,000 cubic feet per second if all the numerous flood peaks along Beardsley Canal had occurred at the same time. The total volume of runoff for this flood was estimated at 10,600 acre-feet. The peak discharge at Luke Air Force Base was estimated at 5,000 cubic feet per second by the U.S. Geological Survey. No flood estimates are available for the study area.

d. Storm and Flood of August 19, 1954. Very moist warm tropical air that originated over the Gulf of Mexico and the Gulf of California entered Arizona and New Mexico from the south during the storm period accompanied by widespread thunderstorm activity. The storm and flood of August 19, 1954, was the most severe on record within the Queen Creek drainage area approximately 50 miles east, southeast of Phoenix. Precipitation in the area occurred between 0100 and about 1000 hours on the morning of August 19, in the Superstition Mountains and Pinal Mountain areas. The precipitation intensities were very high during portions of the storm, especially between 0500 and 0900 hours. The Boyce Thompson Southwestern Arboretum, about 4 miles west of Superior reported the highest measured precipitation amount of 5.3 inches (most of it falling within 3 hours) although greater amounts are believed to have fallen in the mountains to the south. Florence Junction, about 15 miles west of Superior, reported 1 and 6 hour amounts of 1.75 and 4.25 inches respectively, while the smelter at Ray about 11 miles southeast of Superior measured 4.05 inches in less than 2 hours. An estimated 140 square miles of area had over 5 inches of precipitation, and approximately 850 square miles had over 1 inch of precipitation. The isohyets of total storm precipitation are shown on plate 11. Peak discharge at the gaging station on Queen Creek at Whitlow Ranch Dam site near Superior, Arizona (drainage area 144 square miles) was estimated at 42,900 cubic feet per second. No estimate of runoff is available for the study area.

e. Storm and Flood of September 25-26, 1962. The unusually heavy precipitation during the storm was associated with a tropical storm originating off the coast of Baja California. The main stream of moist air which was about 70 miles wide passed over Sells, the Tucson mountains-Cortaro area, Oracle, and on into New Mexico. Heaviest rain fell during the night of September 25 and most of September 26. A total of 4 inches of precipitation occurred at Sells in a ten hour period on September 25 and 26. Estimates of depths of 7 inches were made for two locations about 17 and 22 miles west of Tucson. Peak discharges were estimated by the U.S. Geological Survey for the Santa Rosa Wash near Vaiva Vo (1,782 square miles) at 53,100 cubic feet per second, and for the Santa Rosa Wash at State Highway No. 84, between Casa Grande and Stanfield, at 12,800 cubic feet per second. A significant amount of flood wave attenuation and channel percolation must have occurred to reduce the peak discharge so greatly in the 13 mile reach between the two measuring points. No estimate of runoff is available for the study area.

f. Storm and Flood of December 12-21, 1967. This storm period consisted of two general storm systems - one during December 12 through 16, and the other during December 17 through 21. During December 12 and 13, very cold air invaded Arizona from the north while a deepening upper level low pressure center off the southern California coast brought strong southerly winds aloft to Arizona and caused widespread substantial precipitation over much of the state. Snow fall was very heavy in the mountain areas with some stations reporting unprecedented snow depths and the snow level dropped to as low as 1,000 feet on December 13 and14. Precipitation from this first storm system generally diminished from December 15 through December 17, as the storm began moving to the east. A strong flow of warm moist air from the south began invading Arizona ahead of the second storm system and rainfall over the area began to increase with the snow level rising to around 5,000 feet. Around mid-day on December 19, precipitation became quite heavy over the Phoenix area as a cold front moved through the region from the northwest and a considerable amount of melting snow was added to the runoff. Precipitation intensities diminished and the snow level lowered once again late on December 19, after the passage of the cold front. New December precipitation records were set at several Arizona stations during December 1967, including 16.21 inches at Crown King, 7.30 inches at Flagstaff, and 3.92 inches at Phoenix. All of the months' precipitation fell during the 10 day period of December 12-21 in central Arizona. The heaviest daily precipitation occurred on December 19, with Crown King measuring 6.00 inches and Bumble Bee reporting 4.61 inches. With approximately 5 days of antecedent rainfall during the period December 13-18, the ground conditions were ripe to produce sizeable floods in the Phoenix area during the higher intensity rainfall which occurred during December 19. The New River-Skunk Creek system produced a peak of 19,800 c.f.s. near Glendale (323 square miles).

g. Storm and Flood of September 4-6, 1970. The conditions that led to the storm of September 4-6, 1970 began to develop on September 2, when moist tropical air flowed into southern Arizona, as a result of tropical storm Norma located off the tip of Baja California. The build-up of moist air continued over the entire southwest until September 5, when a cold front entered Arizona from the northwest and strong southerly winds developed over the state. The combination of the upward motion of air ahead of the cold front and the orographic lifting of this moisture laden tropical air led to the extremely heavy rains in central Arizona. Numerous precipitation stations recorded 5 to 8 inches of rainfall in 24 hours. The Workman Creek rain gage, about 60 miles east-northeast of Phoenix, measured 11.4 inches of rainfall which exceeded the previous 24 hour record for Arizona by more than 5 inches. Total storm isohyets for September 4-6, 1970, in Arizona, are shown on plate 12. Record floods occurred in many portions of Arizona, southeastern

Utah, and southwestern Colorado during September 4-6. Heavy rainfall in the mountainous areas of central Arizona resulted in sudden large flood flows on Tonto, Sycamore, Oak, and Beaver Creeks and the east Verde and Hassayampa Rivers. The peak flows on the upper end of New River were the highest since records began in 1960. New River near Rock Springs gaging station (drainage area =67.3 square miles) recorded a peak discharge of 21,100 c.f.s. The Hassayampa River at Box Dam site near Wickenburg had a peak of 58,000 c.f.s. which is more than twice the previous known maximum of 27,000 c.f.s. which occurred in 1927 and in 1951.

h. Storm and Flood of June 21-22, 1972. The heavy thunderstorm which hit northeastern Phoenix, Arizona on the morning of 22 June 1972, was part of a series of moderate to heavy early summer thunderstorms which affected the entire southwest during the period 20-23 June 1972. The storm resulted from a deep flow of very moist unstable tropical air that invaded the far southwestern United States from the Gulf of Mexico and the Pacific Ocean west of Baja California. Most of the storm's rainfall in the Phoenix area occurred during the period 0600 to 1000, on 22 June, and many of the stations observed their greatest intensities during a 1.5 to 2 hour period. The maximum unofficial intensity reported was 5.25 inches during an estimated 2 hours in the vicinity of 24th Street and Camelback Road in Phoenix. Bucket survey amounts of 4.87 inches at 24th Street and Indianola Avenue and 4.8 inches at 28th Street and Indian School Road were confirmed by the National Weather Service. The maximum recording gage intensity measured in this storm was 3.85 inches in one hour and 20 minutes at 18th Street and Turney Avenue. The storm in the Phoenix area was highly localized as shown by the isohyetal pattern on plate 13. The 4 inch isohyet covered 10 square miles while the 2 inch isohyet covered 200 square miles. Heavy runoff from the south slopes of the Phoenix mountains occurred as a result of the intense rainfall of June 22. In Paradise Valley and on the southwest slopes of the McDowell mountains large areas were inundated by sheet flow. Flooding occurred along Indian Bend Wash from Paradise Valley through Scottsdale and Tempe to the Salt River. A peak discharge of 20,000 c.f.s. was measured at Indian Bend Road on Indian Bend Wash. Flooding occurred upstream of Arizona Canal as floodwaters ponded behind the canal levees. Much of the damage downstream of Arizona Canal resulted from breaks in the canal as overtopping occurred.

3-04. CRITICAL FLOOD PRODUCING RAINFALL. The relative severity of past general storms and local storms in the Gila River Basin was determined by analyzing precipitation depth-area-duration relationships. The analyses in the study area show that although the total precipitation for general summer storms is greater, the short time precipitation intensity for the local summer thunderstorm is the more critical flood peak producing factor for drainage areas smaller than about 700 to 800 square miles. For drainage areas larger than about 800 square miles, the greater areal extent of the general summer storm becomes the more important flood peak producing factor in the determination of critical rainfall. The relative magnitude of precipitation for recorded local storms is best shown by intensity-duration and depth-area curves shown on plate 14. The drainage area size over which either storm type governs is also a function of the drainage basin characteristics such as elevation, exposure, shape, and runoff properties.

IV - SYNTHESIS OF STANDARD PROJECT FLOOD

4-01. GENERAL. The standard project flood (SPF) represents the flood that would result from the most severe combination of meteorologic and hydrologic conditions considered reasonably characteristic of the region. It normally is larger than any past recorded flood in the area, and can be expected to be exceeded in magnitude only on rare occasions. It thus constitutes a standard for design that will provide a high degree of flood protection. Preparation of standard project flood estimates in this report were made in accordance with EM 1110-2-1411 (Standard Project Flood Determinations).

4-02. STANDARD PROJECT STORM (LOCAL TYPE). The August 19, 1954 thunderstorm that was centered generally in the Queen Creek drainage area was determined to be the storm with the most severe flood peak producing rainfall depth-area-duration relationship and isohyetal pattern that may reasonably be expected to occur over the central portion of Arizona. While the storm lasted a total of about 9 hours, local observations during the storm indicated that nearly all of the precipitation fell during a 7 hour period and that most of the rainfall occurred at many stations within 3 hours or less. Extremely intense rates of precipitation for very short durations (5 minutes to 1 hour), although not measured in the August 19, 1954 Queen Creek storm because of the complete lack of properly functioning recording rain gages in the area at the time of this storm, have been measured on a number of other occasions in the vicinity of central Arizona, and are therefore considered to be reasonably characteristic of the heavier thunderstorms of this part of the state. Thus, a standard project storm of 7 hours duration, having large portions of the total precipitation occurring within 1 to 3 hours, was developed. The methods used to determine the total precipitation amounts, the intensity-duration relationships, and the precipitation-intensity patterns are explained in the following subparagraphs:

a. Total Precipitation. Total precipitation amounts for the standard project local storm were obtained from the isohyets (pl. 11) of the August 19, 1954 Queen Creek thunderstorm, transposed and centered over various drainage basins within the greater Phoenix area. Because the heaviest precipitation of this storm occurred in mountainous areas where it is felt that orographic influences were significant, the total storm depth was altered as it was transposed to the Phoenix area (as well as to other areas) by 10-year 6-hour precipitation values obtained from the National Weather Service charts. (See pl. 15.) This particular parameter was selected as the transposition factor because it is believed to be the most accurate available rainfall statistic representative of rare-event precipitation of 7 hours' duration. Transposition of the August 19, 1954 thunderstorm from the Queen Creek area to the vicinity of Phoenix by this method results in a reduction of the total storm magnitude by nearly 20 percent. Transposition of the storm to the foothill and mountain areas north of Phoenix results in smaller reductions or even slight increases in the total depth of the original storm. Plate 16 presents the depth-area reduction factor as a function of drainage area and 10-year 6-hour precipitation. Thus, the average rainfall depth over a watershed is equal to the product of 10-year 6-hour precipitation, depth-area reduction factor, and 3.178. The factor 3.178 is the ratio of maximum point rainfall to 10-year 6-hour precipitation for the observed Queen Creek storm.

b. Intensity-Duration Relationships. For six intense storms in the vicinity of central Arizona and for the Queen Creek storm of August 19, 1954, intensity-duration values were complied and drawn (with other intense, historical storms of the area serving as a guide) for durations from 5 minutes to 7 hours. The intensity-duration values complied were for the maximum point value of the storm (see pl. 14). Each intensity-duration value for the Queen Creek storm was then transposed from the Queen Creek area to Phoenix (as well as to other areas) on a separate basis, with a tranposition factor progressively closer to 1.0 for

progressively decreasing durations. The resulting standard project thunderstorm total precipitation is therefore significantly greater in the higher mountain areas than in the lower desert valleys, whereas the maximum 5- and 10-minute intensities are only slightly greater in the mountains than in the deserts.

c. Precipitation-Intensity Patterns. The six time distributions of precipitation used for this study were based upon the limited amount of information available from the August 19, 1954, Queen Creek thunderstorm, aided by the precipitation-intensity patterns for 13 heavy thunderstorms, ranging in duration from 3/4 hour to 3-1/2 hours, that have occurred in Arizona. Plate 17 shows these six patterns. The selection of a time pattern (indicated by pattern numbers) from plate 17 is based on the fact that patterns decrease in relative severity with both increasing drainage area and 10-year 6-hour precipitation statistic. Plate 18 shows the pattern numbers of plate 17 as a function of both these factors. With both factors known, plate 18 is used to determine which pattern number from plate 17 is selected. It can be seen from plate 18 that patterns 1 and 2 should apply primarily to small drainage areas in the lower desert valleys, whereas patterns 4 and 5 would generally apply to the larger drainage areas, especially in the higher mountains.

d. Antecedent Rainfall. Ground conditions characteristic of standard project flood conditions are assumed to be established by 0.5 inches of precipitation occurring within a 24 hour period immediately prior to the standard project storm. This assumption has some basis in that a secondary storm cell formed in the same general area on the day following the Queen Creek storm of August 19, 1954. Meteorologically there is no reason that this secondary storm cell could not have occurred prior to the main Queen Creek storm. The only adjustment in computation of SPF is therefore to reduce the severity of the loss rate function to account for the antecedent rainfall as explained in paragraph 4-04c.

4-03. STANDARD PROJECT STORMS (GENERAL TYPE).

a. The general summer storm of 4-6 September 1970, which brought very heavy precipitation to all of central and northeastern Arizona, as well as parts of other states, was determined to be the storm with the most severe flood peak producing rainfall depth-area-duration relationship for large areas that may reasonably be expected to occur over the central portion of Arizona. Nearly all of the precipitation in central Arizona occurred within the 50 hour period from 1800 MST on 3 September to 2000 MST on 5 September, and hence 50 hours was selected as the duration for the derived standard project general storm.

(1) Meteorology. The storm of 4-6 September 1970 was generated when a cold air mass from the Gulf of Alaska moved southeastward and collided with a deep and very moist layer of air streaming northward from Tropical Storm Norma off the west coast of Baja California. Widespread thunderstorm activity began over nearly all of Arizona late on 3 September, when a strong cold front separating the cold and the warm, moist air masses moved southeastward across Arizona and triggered a 4- to 5-hour period of extremely heavy rainfall over the central portions of the state. The rainfall tapered off and ended during the evening of 5 September, as the colder, drier air mass moved across the region.

(2) Total Precipitation. Total precipitation amounts for the standard project design general storm were obtained from the analyzed isohyets (pl. 12) of the4-6 September 1970 general summer storm in Arizona, transposed and centered over various drainage basins within the greater Phoenix area. Because the largest precipitation totals in this September 1970 storm occurred in rugged, mountainous terrain some distance north of Phoenix, where

orographic influences in the generation of precipitation were of considerable significance, the total storm depth and the individual rainfall intensities were proportionally altered as they were transposed to the study areas by the 10-year 24-hour precipitation values obtained from the National Weather Service charts (pl. 19). This particular parameter was selected as the transposition factor because it is believed to be the most accurate available rainfall statistic representative of rare-event precipitation of 24 to 48 hours' duration. The September 1970 storm isohyets were converted to isopercentuals of the 10-year 24-hour precipitation by dividing the total storm precipitation by the 10-year 24-hour precipitation values. These isopercentuals were then transposed to each desired drainage area, centered, and converted back to isohyets, from which the total precipitation for the standard project design storm was obtained for each drainage area.

(3) Precipitation-Intensity Patterns. The time distribution of precipitation used for the standard project general storm was based upon the recorded time distributions of the September 1970 storm at six automatic rainfall stations in the area north and west of Phoenix where the maximum storm isopercentuals of 10-year 24-hour precipitation occurred. In order to realistically reflect the time distribution patterns likely to be representative of all drainage area sizes, ranging from small areas near the isopercentual center of the storm to large areas encompassing considerable variation in the time distribution, a graduated system incorporating means of September 1970 rainfall time distributions of from one to three recording stations was utilized. For design areas of less than 200 square miles, the time distribution of only one station (Youngtown, Arizona-USGS No. 25D) was used to represent the standard project storm rainfall distribution. For areas between 200 and 600 square miles the mean of the time distributions of rainfall for the Youngtown gage and Skunk Creek (USGS No. 33) gage was used. For areas larger than 600 square miles a distribution equal to the mean of distributions from the two previous gages as well as the Wagoner (USGS No. 30) gage was selected for the standard project storm pattern. The location of the rainfall stations referred to can be found in table 1 and on plate 8.

b. The general summer storm of August 26-29, 1951, which brought heavy precipitation to southern Yavapai County and northeastern Maricopa County of Arizona, was determined to be the storm with the most severe flood volume producing depth-area-duration relationship for large areas that may reasonably be expected to occur over the central portion of Arizona.

(1) Storm Transposition. The transposition of the August 1951 storm over the project area was accomplished by: (a) expressing the actural rainfall amounts in the 1951 storm as percentages of the mean summer seasonal rainfall (June through September); (b) constructing an isopercentual map based on those percentages (see pl. 20); and (c) transposing the isopercentual lines to such a position over the basin as would result in the most total rainfall over the drainage area. Although the mean precipitation for the months of June through September (pl. 21) includes precipitation from both local and general summer storms, the use of this mean summer seasonal precipitation as a storm transposition factor was considered reasonable in view of: (a) the fact that the August 1951 storm consisted of numerous local storm cells as well as general precipitation, and (b) the relatively short distance and similarity of terrain over which the storm was transposed. Also, the total storm depth resulting from transposition by mean summer seasonal precipitation differs by only a small percentage from total storm depths produced by using the 2-year 24-hour or 10-year 24-hour precipitation as a transposition factor.

(2) Rainfall-Intensity Pattern. One hour was selected as the smallest time interval for which rainfall intensities would be required in developing the standard project volume flood. The pattern of rainfall intensities was based upon the mean of the hourly percentage distributions of the August 1951 storm at the Phoenix Weather Bureau City and Poland Junction gages (see pl. 22).

c. Antecedent Rainfall. Standard project general storm is a general summer event. Antecedent ground conditions considered reasonable for standard project flood computations were established by assuming previous summer general storm(s) have occurred in the weeks prior to, but not immediately preceeding the general storm standard project precipitation. Hence, infiltration rates are considerably lower than would be expected on a dry watershed; however, depression storage and interception losses must still be satisfied. Paragraph 4-04c presents the quantitative loss rate function used for standard project flood computations.

4-04. DETERMINATION OF RAINFALL-RUNOFF RELATIONSHIPS.

a. Flood Reconstitutions.

(1) Reconstitution of 22 observed flood events in the Phoenix, Arizona region was accomplished to determine applicable relationships between rainfall and runoff for use in computing hypothetical flood hydrographs. Table 3 outlines the stream gage locations and storms used for each reconstitution. The HEC-1 Flood Hydrograph Package computer program was used to perform a regional unit hydrograph and loss rate study in order to derive rainfall-runoff relationships applicable to the Phoenix area.

(2) To model the subbasin flood runoff process with HEC-1 (Hydrologic Engineering Center) requires complete definition of a unit hydrograph and precipitation loss rate criteria for each subbasin within the river basin to be modeled. The studies to determine the criteria may be termed "Regional Unit Hydrograph and Loss Rate Studies" since the objective is to derive criteria that are applicable generally to the entire river basin including those subbasins which are presently not gaged.

(3) HEC-1 has the capability to automatically determine a set of unit hydrograph and loss rate parameters that "best" reconstitutes an observed runoff event for a subbasin given the subbasin basin average rainfall, the drainage area and a few runoff hydrograph parameter values for starting flow and base flow recession computations. The "best" reconstitution is considered to be that which minimizes the weighted squared deviations between the observed hydrograph and a reconstituted hydrograph. The optimization procedure used to derive values for the variables is the unit variate gradient search procedure. A detailed description of the procedure is contained in HEC Technical Paper No. 2, "Optimization Techniques for Hydrologic Engineering", Leo R. Beard, 1966.

(a) The runoff hydrograph variables that are not automatically derived and thus must be determined by evaluating the characteristics of observed flood hydrographs in the region are:

Runoff Hydrograph Variables

QRCSN	–	The discharge at which recession flow begins, may be a function of peak discharge, precipitation intensity, drainage area, or other watershed characteristics.
STRTQ	–	Recession flow for antecedent runoff, the discharge at the beginning of the first period of reconstitution, a selected value.
RTIOR	–	Recession coefficient that is the ratio of flow at time t to that 10 computational periods ($t+10\Delta t$) later during recession, may be a function of soil type, land use, water table level, soil profile and permeability. RTIOR is applied to both STRTQ and the runoff hydrograph when the flow is less than or equal to QRCSN.

(b) The runoff computation variables that can have values derived automatically include Clark unit hydrograph variables:

Clark Unit Hydrograph Variables

TC	–	Clark unit hydrograph time of concentration, in hours (may vary somewhat from storm to storm).
R	–	Clark unit hydrograph storage coefficient, in hours (may vary somewhat from storm to storm).

To reduce the interdependency between TC and R, in their automatic derivation, the variables are regrouped into two new variables as follows:

TC+R	–	Sum of time of concentration and storage coefficient. Variable used in HEC-1 for optimizing unit hydrograph parameters.
R/(TC+R)	–	Ratio of storage coefficient to sum of time of concentration and storage coefficient . Variable used in HEC-1 for optimizing unit hydrograph parameters.

The loss rate variables that can have values derived automatically are listed below. A graphical representation of these parameters is illustrated on plate 23.

(c) Loss Rate Parameters

DLTKR	—	Amount of initial accumulated rain loss during which the loss rate coefficient is increased. This parameter is considered to be a function primarily of antecedent soil moisture deficiency and is usually different for different storms.
STRKR	—	Starting value of loss coefficient on exponential recession curve for rain losses (snow-free-ground). The starting value is considered a function of infiltration capacity and thus depends on such basin characteristics as soil type, land use and vegetal cover.
RTIOL	—	Ratio of rain loss coefficient on exponential loss curve to that corresponding to 10 inches more of accumulated loss. This variable may be considered a function of the ability of the surface of a basin to absorb precipitation and should be reasonably constant for large rather homogeneous areas.
ERAIN	—	Exponent of precipitation for rain loss function $ALOSS = (AK + DLTK)\ PRCP^{ERAIN}$ that reflects the influence of precipitation rate on basin loss characteristics. It reflects the manner in which storms occur within an area and may be considered a characteristic of a particular region. Varies from 0.0 to 1.0. The terms in the equation are defined as:
ALOSS	=	loss rate for particular time interval in inches per hour
AK	=	Loss rate coefficient at beginning of time interval, value on STRKR exponential loss curve (see pl.23).
PRCP	=	rainfall intensity in inches per hour
DLTK	=	incremental increase in loss rate coefficient. DLTK is assumed to be a parabolic function of the accumulated loss for DLTKR amount of accumulated loss. DLTK is a maximum of 0.2 DLTKR initially, reducing to zero when the accumulated loss equals DLTKR.

(4) HEC-1 determines the unit hydrograph and loss rate parameters in a sequential successive approximation manner as described in Technical Paper No. 2. The sequential successive approximation approach for each variable is necessary because virtually all variables to be optimized are interdependent to some degree. This fact coupled with the previous discussion of the reasons that the variables can assume different values for a given subbasin make it mandatory that a specific procedure or sequence of steps be followed in determining appropriate values.

(5) In this study all reconstitutions were set up and run simultaneously in order to regionalize the various loss rate and runoff variables. On each successive computer run of all reconstitutions the following steps or modifications were made: (1) ERAIN fixed to zero, (2) RTIOL fixed to 2.0 and DLTKR fixed to zero for reconstitutions with appreciable antecedent rainfall and (3) STRKR fixed to the value determined in the previous run. Additional runs were made keeping the loss rate parameters fixed as determined above and utilizing the flow adjustment option to obtain the best fit between observed and computed flood hydrographs.

(6) Table 4 presents a summary of "best-fit" unit hydrograph and rainfall loss rate parameters determined for each reconstitution. Interpretation of the HEC-1 reconstitutions enabled selection of appropriate rainfall-runoff relationships for the Phoenix, Arizona region. Flood reconsitutions for New River near Rock Springs, Skunk Creek near Phoenix, and Agua Fria Tributary at Youngtown for the storm September 4-6 1970 are presented graphically on plates 24, 25, and 26 respectively.

b. Unit Hydrographs. A unit hydrograph is a flood hydrograph resulting from one inch of effective rainfall occurring uniformly over a watershed in a specified time duration. The unit hydrograph concept is a useful tool in that it provides a linear description of runoff from a watershed and facilitates the generation of synthetic flood hydrographs for catchments which are ungaged. The unit hydrograph procedure used by the Los Angeles District, has its basis in an S-graph which is the time distribution of runoff as a function of basin lag time. Lag time is defined as the time in hours for 50 percent of the total volume of runoff of the unit hydrograph to occur. The basin lag time can be approximated for ungaged watersheds by the use of the lag relationship presented on plate 27. The basin-n-value is a proportionality factor in the equation for lag time which permits adjustment of lag time depending on type of ground cover and surface characteristics affecting basin response to effective rainfall. Examples of unit hydrographs developed using this procedure can be seen on plates 36 through 39. It should be noted that base flow and interflow components of runoff do not represent significant portions of observed flood hydrographs in the Phoenix area. Hence, no separate provision is made for base flow and interflow in flood hydrograph determination; they are included directly in the unit hydrograph.

(1) S-Graphs. The concentration of excess rainfall in the Phoenix area, can be described by two S-Graphs, a valley and a mountain. These two S-graphs are presented on plates 28 and 29. The Phoenix Valley S-Graph was derived from reconstitutions at the following locations: New River at Bell Road; Skunk Creek near Phoenix; Cave Creek at Phoenix; Agua Fria Tributary at Youngtown and Queen Creek Tributary at Apache Junction. Similarly the Phoenix Mountain S-Graph was derived from the New River near Rock Springs and New River at New River reconstitutions. The use of the Phoenix Mountain or Valley S-Graph adequately describes the time distribution of runoff from watersheds in the Phoenix region.

(2) Basin n Values. Basin-n-values derived from the reconstituted unit hydrographs were used as a guide to establishing the following standard project flood basin-n-values:

Mountain	—	n = 0.040 - 0.045	depending on ruggedness of terrain
Foothill	—	n = 0.035	
Valley	—	n = 0.030	
Lightly urbanized valley	—	n = 0.020 - 0.025	depending on extent of development
Highly urbanized	—	n = 0.015	

The basin n-values presented above are only to serve as a general guide. Adjustments to these n-values are made on the basis of the influence of any surface characteristics which affect the lag time of the watershed.

c. Rainfall Loss Rate Function. Based on the HEC loss rate parameters (see paragraph 4-04a) derived from the reconstitutions in the Phoenix region (see table 4), the following values were chosen:

STRKR = 0.40 DLTKR = 1.5 RTIOL = 2.0 ERAIN = 0

A graphical representation of this loss rate function is shown on plate 30. The initial portion of this loss rate function has a high loss rate indicative of a dry watershed in which the loss of interception and depression storage as well as high initial inflitration rate must be satisfied before rainfall excess becomes available for runoff. The more slowly decreasing portion of the loss rate function seeks to describe the slowly decreasing inflitration rate as a function of accululated loss. The HEC loss rate function accounts for the effects of urbanization by reducing the effective loss rate in direct proportion to the percent impervious cover. Included on plate 30 is the loss rate function adjusted for 0.5 inches of antecedent rainfall loss. The HEC loss rate function parameters then become STRKR = 0.38, DLTKR = 1.0, RTIOL = 2.0, and ERAIN = 0. This lower overall loss rate function is used in conjunction with standard project local storm to determine standard project flood. Also shown is the loss rate function used for general storm standard project flood computation. Loss rate parameters were established as STRKR = 0.30, DLTKR = 1.0, RTIOL = 2.0, ERAIN = 0. Paragraphs 4-02d and 4-03c describe the antecedent ground conditions chosen for local and general storm standard project floods. Appropriate ajustments to the dry watershed loss rate function were made on the basis of the established antecedent ground conditions.

d. Base flow and Snowmelt. Base flow is considered negligible for this study area as runoff occurs only as a direct response to high intensity rainfall. Allowance for snowmelt is inappropriate in this region for storms occurring in the summer season.

4-05. FLOOD ROUTING.

a. A technique of hydrologic flood routing in the Phoenix, Arizona region must be general enough to handle a wide range of discharge and channel conditions. Of primary importance is the ability of a routing procedure to accurately describe the attenuation of a flood wave when a large amount of channel storage is encountered as in the flat valley areas in and around Phoenix. For this reason the Muskingum flood routing method was chosen for flood routing in natural channels. The Successive-Average Lag method was reserved for use in well defined project channels where overbank storage was assumed not to be significant. Reservoir routing was accomplished by the Modified Puls routing procedure. Each of these flood routing methods is described in detail in "Routing of Floods Through River Channels", EM 110-2-1408, Corps of Engineers, 1 March 1960.

b. From topographic maps produced by aerial mapping, cross sections published in flood plain information studies, USGS topographic maps, and field observations, representative cross sections were determined for each channel reach under natural conditions. The Interim Report on Survey for Flood Control, Phoenix, Arizona and Vicinity was used to establish both channel locations and cross sections in the proposed plan of improvement.

Flood wave travel time in a reach is determined by dividing reach length by average peak flow velocity. Manning's formula for normal flow and an appropriate channel cross section are used to compute the average peak flow velocity for the reach. The average peak discharge for a reach is taken as the mean of peak discharge at the upstream and downstream limits of the reach. The Muskingum storage constant K is approximated by the travel time of a flood wave through a reach. The Successive Average Lag routing method is based on empirical observations of flood wave attenuation in which the choice of routing coefficients is a function of flood wave travel time and the unit time used for the routing computation.

c. The Muskingum K value may be determined directly from a computation of flood wave travel time in a specific reach, but the Muskingum X value does not lend itself to direct analysis. It was therefore necessary to define the Muskingum X coefficient in terms of three parameters: channel slope, cross sectional shape, and discharge. In a steep (slope equals 0.005 or greater) channel in which the discharge is confined to an efficient cross section with little overbank flow, the Muskingum X value was taken as 0.4. In a mildly sloped channel with a large portion of the flow in the overbanks, the Muskingum X value was taken as 0.2 or lower. All routing reaches were considered individually and the Muskingum X coefficient prorated between the two extremes described for each flood computation. A comparison between typical Muskingum values used and the results of an optimization analysis of Muskingum coefficients for the floods of September 1970 is presented in table 5.

4-06. CHANNEL PERCOLATION. Losses due to streambed percolation in the Phoenix area were determined using observed flood data for the September 4-6, 1970 storm. The record of water stage behind Cave Creek Dam as well as the observed flood hydrograph for Cave Creek at Phoenix allowed computation of channel percolation. The recession limb of the observed flood hydrograph for Cave Creek at Phoenix leveled out at a constant flow of 290 c.f.s. between 1800 hours of September 5, and 2400 hours September 7. Normally channel percolation would rapidly diminish the tail end of recession flow. In this case the steady outflow from Cave Creek Dam was the source of the constant flow. Using the stage recorder chart for the upstream face of the dam, and applying the orifice formula to the four foot square ungated outlet, the outflow was found to be 400 c.f.s. for the same period that 290 c.f.s. was recorded downstream. This meant 110 c.f.s. was lost to channel

percolation in the 11.7 mile channel reach. Assuming an average wetted channel width equal to 75 feet, a loss rate of 1.05 c.f.s. per wetted acre was computed. With this observed percolation rate on Cave Creek, the following percolation rates were chosen for the Phoenix region: main channel equals 1.25 c.f.s. per wetted acre, and overbank equals 0.50 c.f.s. per wetted acre. The higher value for main channel percolation was chosen because the 1.05 c.f.s. per wetted acre figure was for a 290 c.f.s. discharge. Higher discharges would produce higher hydrostatic heads and thus a higher percolation rate. The overbank material is less pervious than the stream bed deposits, hence the percolation rate for the overbank area was taken as 0.50 c.f.s. per wetted acre. The effect of channel percolation on a flood hydrograph is computed by reducing the entire flood hydrograph by the ratio of the discharge lost in channel percolation to the average peak discharge in the reach. The discharge lost to channel percolation is found by applying the percolation rates established above to the natural channel cross section when the flow is at the average peak discharge.

4-07. COMPUTATION OF STANDARD PROJECT FLOOD.

a. Stream System Analysis.

(1) The stream system analysis approach to computation of design floods involves division of a study area into subbasins, which are homogeneous with respect to hydrologic and meteorologic factors, and routing and combining the flood hydrographs generated from each subbasin to determine the design flood at a desired concentration point. Subdividing a watershed into subbasins permits more accurate modeling of the runoff process, as variations in topography, urbanization, and rainfall, as well as channel shape and slope may be incorporated into the hydrologic description of the basin.

(2) Standard project flood is computed by centering the standard project precipitation in the most critical flood producing manner. Application of the rainfall loss rate function described previously to standard project precipitation enables determination of the rainfall excess hyetograph. The rainfall excess hyetograph is then applied to the subbasin unit hydrograph to produce the subbasin flood hydrograph. Routing and combining of all subbasin flood hydrographs to the desired concentration point completes the computation of standard project flood. Computer programs such as the HEC Combining and Routing (723-G1-L2320) greatly facilitate stream system computations of design floods.

b. Standard Project Flood Discharges.

(1) Standard project flood peak discharges, computed as described in foregoing paragraphs, are presented for three plans: (a) present no-project conditions on plate 31; (b) future no-project conditions on plate 32; and (c) the authorized plan, in the future with the CAP, in table 6 (locations shown on plate 33). All supporting information including subarea map (plate 34), schematic flow diagram (plate 35), subarea drainage characteristics (table 7), channel reach characteristics (table 8), as well as typical flood routing coefficients (table 5) is presented for the present no-project case. Other plans or alternatives would be set up as stream systems in an analogous manner.

(2) Standard project flood hydrographs for the authorized Cave Buttes, Adobe, and New River damsites as well as the existing Cave Creek Dam are shown on plates 36, 37, 38, and 39, respectively. It should be noted that the SPF at Cave Buttes damsite was computed assuming the existing Cave Creek Dam breached. The standard project local summer storm produced the maximum peak runoff rate at all authorized damsites. The standard project local summer storm produced the maximum volume of runoff for Adobe Dam, while the

August 1951 general summer storm produced the maximum volume of runoff for Cave Buttes and New River Dams. For the authorized dam outlet sizes (5,400 cfs for Cave Buttes Dam, 2,000 cfs for Adobe Dam, and 1,000 cfs for New River Dam) these same respective storms produced the maximum design storage, except for Cave Buttes Dam which is governed by the local summer storm. However, the proper volume flood for reservoir design storage is heavily dependent on the choice of reservoir outlet size. The floods produced by the two standard project precipitation (SPP) general storms are shown for Cave Buttes damsite on plates 40 and 41. The flood produced by the SPP August 1951 general storm is shown for New River damsite on plate 42.

(3) Standard project flood peak discharges computed using the hydrologic procedures updated from the Interim Report as outlined in this report are approximately 20 percent higher than the Interim Survey Report values for subbasins less than about 200 square miles. Agreement between the two sets of standard project floods becomes progressively closer as drainage area size increases.

V - SYNTHESIS OF PROBABLE MAXIMUM FLOOD

5-01. GENERAL. The probable maximum flood (PMF) is defined as the flood that would result if the probable maximum precipitation for the drainage area were to occur at a time when ground conditions were conducive to maximum runoff. Probable maximum flood as its name implies, is an estimate of the upper bound of flood potential on a watershed. Such a hypothetical flood is necessary for proper design of dam and debris basin spillways.

5-02. PROBABLE MAXIMUM PRECIPITATION. Probable maximum precipitation (PMP) is considered the practical upper limit of available precipitable water over an area as estimated by the Hydrometeorological Branch of the National Weather Service. The preliminary draft of "Probable Maximum Thunderstorm Precipitation Estimates – Southwest States" dated August 1972 prepared by the National Weather Service and amended as per letter, dated March 29, 1973 (subject: "Probable Maximum Thunderstorm Precipitation Estimates for Southwest States," August 1972), was used to establish local storm PMP estimates for a unit time period of 15 minutes as well as the incremental PMP time pattern. PMP hyetographs for Cave Buttes Dam, Adobe Dam, New River Dam, and Cave Creek Dam drainage area are shown on plates 43, 44, 45, and 46 respectively. Total 6-hour local storm PMP for the authorized Cave Buttes, Adobe, and New River damsites were 7.55, 10.02, and 8.00 inches respectively. Probable maximum precipitation for the general storm was outlined in a letter from the National Weather Service, dated 29 September 1972 (subject: "Probable Maximum Precipitation Estimates for Drainages North of Phoenix, Arizona.") As with SPP, the local storm PMP with its high rainfall intensities produced the critical rainfall amounts as compared with general storm PMP for the relatively small drainage areas sizes of the damsites proposed in the authorized plan.

5-03. PROBABLE MAXIMUM FLOOD. Computation of PMF for the thunderstorm event is accomplished in the same manner as SPF with two exceptions. First, basin lag time is reduced by 15 percent to account for the reduction in time of concentration of rainfall excess characteristic of large floods where the hydraulic efficiency of the watershed is increased by high depths of flow. Secondly, the loss rate is taken as a constant equal to 0.15 inches per hour for the entire duration of the storm. This is a minimum loss rate deemed reasonable of a watershed saturated by antecedent rainfall. PMF hydrographs for Cave Buttes, Adobe, New River and Cave Creek damsites are shown on plates 43, 44, 45, and 46 respectively. For spillway design at the damsites the water surface elevation should be assumed at spillway crest at the beginning of PMP. The PMF at Cave Buttes damsite was computed assuming the existing Cave Creek Dam breached.

VI – DISCHARGE FREQUENCY ANALYSIS

6-01. GENERAL.

a. Present condition without project discharge frequency analysis in the Phoenix area involves determination of peak discharge frequency curves at points along major watercourses, many of which are gaged, and on smaller streams and tributaries most of which are ungaged. In addition is the problem of developing frequency curves for urbanized areas. Where stream gage records were available, a graphical frequency approach was used in combination with an understanding of the meteorologic and hydrologic characteristics of the region. A form of regional flood frequency analysis was developed to handle the problem of determining frequency curves for ungaged non-urbanized watersheds. An evaluation of two discharge frequency curves on urbanized watersheds in the region permitted development of frequency curves for ungaged urban watersheds.

b. Future condition without project discharge frequency analysis was accomplished using combinations of the same basic techniques outlined for the present condition case. Since urbanization takes place only on the lower one-third of both the New River-Skunk Creek and Cave Creek drainages, future condition discharge frequency analysis was done by modifying the present condition without project frequency curves depending on the degree of future urbanization. This approach to future condition discharge frequency curves applies only to gaged watersheds with non-homogeneous surface characteristics. Ungaged watersheds which are uniformly rural or urban are handled the same as for the present condition case.

c. Concentration points in the future condition with authorized plan are located mainly in the urbanized valley area and therefore discharge frequency analysis may be handled analogous to the no-project case (present or future) with due consideration to the outlet capacities of the authorized detention dams.

6-02. GRAPHICAL FREQUENCY ANALYSIS.

a. The graphical method of streamflow frequency analysis was used in Phoenix, as it allowed the best use of recorded and historical floodflows, as well as the opportunity to input engineering judgement based on the meteorologic and hydrologic characteristics of the region. Recorded and historical floodflows were plotted for each stream gage in the study area based on the median plotting positions in Beard's "Statistical Methods in Hydrology." Five long record stream gages (San Carlos River near Peridot, Gila River near Solomon, Salt River near Roosevelt, Hassayampa River at Box Damsite and Salt River at Granite Reef Dam) were compared for record consistency in order to estimate SPF exceedence frequency (see pls. 47 through 49a). Resulting SPF exceedence percentages varied as follows: San Carlos and Hassayampa – 0.2 to 0.5 percent, Granite Reef – 0.4 to 0.5 percent and Solomon – 0.3 percent. SPF for Roosevelt is not available. Variations were dependent on graphical or analytical curve fitting of the data. All stations except Granite Reef show consistency through similar standard deviations. Granite Reef is affected by storage and snowmelt. Station statistics are shown on each plate. This analysis indicates that an SPF exceedence frequency of 0.2 to 0.5 percent is reasonable for areas in this study.

b. An explanation of the behavior of the statistical parameters is now in order; however, consider first several basic facts:

(1) The upstream reaches of the study area consist of mountains ranging up to 5,000 feet in elevation.

(2) The downstream end of the study area consists of a large alluvial fan with its population centers.

(3) Major storm run-off occurs largely from summer storms of both the general and cloudburst types.

(4) Orographic lifting causes the storms to occur more frequently in the mountain areas than in the valley areas.

With these basic facts in mind, consider their influence on the four parameters of the frequency curves in Phoenix.

c. The mean annual peak discharge is expected to increase as drainage area increases. This is true, but only for the mountain areas. When moving to concentration points in the valley, the mean actually decreases with increased drainage area size. The reason for this phenomena is that storms tend to center in the mountains, thereby producing their high discharges in the mountains. Upon reaching the valley areas flood flows are attenuated by flatter, wider channels and the volume of runoff is reduced by channel percolation. Concentration points located large distances from the mountains may have large drainage areas; however, the attenuation effects on flood flows originating in the mountains becomes increasingly pronounced.

d. The standard deviation increases as one proceeds downstream. This is caused by the higher incidence of zero or low flows at downstream points because upstream flood flows are attenuated and percolated before they can reach downstream. In a given year, run-off may have occurred only in the mountains due to orographically induced rainfall. For large drainage areas comprised mainly of valley type area, large discharges result from general type storms or widespread thunderstorms which occur less frequently.

e. Skew of the frequency curves becomes more negative as valley drainage area size increases due to the necessity of bending the steeply sloped lower portions of the frequency curve in a concave downward manner to intersect the less frequent floods, which have an upper bound defined by PMF.

f. Standard project flood for concentration points in or near the mountains occurs more frequently than for valley concentration points because of the tendency of storms to center in the mountains. By contrast standard project flood for valley concentration points far removed from the mountains and having a large drainage area would require a direct "hit" by a large storm, thereby making standard project flood less frequent.

g. Discharge frequency curves drawn using the procedure outlined above, for stream gages located along Cave Creek, Skunk Creek, New River, and the Agua Fria River are presented on plates 50 through 57 inclusive. Refer to plate 8 for the location of these stream gages. It should be noted that these discharge frequency curves form the basis for discharge frequency estimates for both present and future without project conditions.

6-03. FLOOD FREQUENCY REGRESSION ANALYSIS.

a. Some form of regionalization of flood frequency curves in the Phoenix area is necessary to determine discharge frequency curves for ungaged watersheds. A regional frequency analysis using stream gages in the Phoenix-Tucson region as well as a study using stream gages in the Phoenix region only were attempted but failed to yield usable results because of several factors: short stream gage records, the complications due to runoff from localized thunderstorm events, and the lack of a long term stream gage in the immediate

Phoenix valley area which correlated well with the short term gages. Also, a straight forward analytical approach to frequency curves provided unreasonably high results for the less frequent events because zero flows in several of the records caused extremely high standard deviations. Hence, a straight forward regression analysis, with each return frequency flood on gaged watersheds determined by graphical analysis as a dependent variable and various hydrologic and meteorlogic parameters as independent variables, was performed.

b. Streamflow records for a total of 43 gages (see tables 9 and 10) in south central Arizona were compiled. The drainage area sizes range from 0.13 to 417 square miles. Graphical frequency curves were drawn for each of these gages using the procedures outlined in Beard's "Statistical Methods in Hydrology." Zero flows were ranked along with the positive flows, hence the entire period of record was utilized for each stream gage. The standard project flood computed using the local standard project storm and precipitation-runoff relationships outlined in previous paragraphs was plotted on each frequency curve between a 200- and 500-year return period. The 100-year, 50-year, 25-year, 10-year, 5-year, and 2-year peak discharges at each gage were then used as the dependent variables in the regression analysis.

c. The following independent parameters for each watershed were determined: drainage area in square miles, average basin elevation in feet, normal annual precipitation in inches, percentage mountain area in basin, basin response (drainage area divided by lag time) in square miles per hour, shape factor (drainage area divided by length squared), 5-year 6-hour rainfall in inches, 5-year 24-hour rainfall in inches, 50-year 6-hour rainfall in inches, 50-year 24-hour rainfall in inches, 10-year 6-hour rainfall in inches, 10-year 24-hour rainfall in inches, 25-year 6-hour rainfall in inches, 25-year 24-hour rainfall in inches, basin n-value, length of longest watercourse in miles, length from centroid of the area to the gage in miles, and lag time in hours. A logarithmic transformation of independent variables was made for all correlation runs.

d. The regression analysis was performed by using HEC computer program 704-G9-L2020-Multiple Regression Package, which makes successive runs in which it eliminates the independent variable having the least significance based on the partial determination coefficients. After an initial run of the multiple regression program, it was decided to reduce the number of independent variables to three: drainage area, basin slope, and 5-year 24-hour rainfall (see tables 9 and 10). After the next run, watersheds were divided into two groups based on drainage area size in order to achieve better correlation results. The basins were grouped from zero to 15 square miles and from 15 to 417 square miles. The regression equations derived for the two drainage area groups as well as the adjusted correlation coefficient (R-Bar) and the coefficient of determination (R-Bar Squared) are listed for each return period in table 11. R-Bar Squared varied from 0.5087 for the 5 year return period flood to 0.6697 for the 100 return period flood for drainage areas between 15 and 417 square miles. In a similar manner, R-Bar Squared ranged from 0.2513 for the 5 year flood to 0.7621 for the 100 year flood for drainage area sizes between 0 and 15 square miles. R-Bar Squared (between 0 and 1) expresses the percentage of variance of the dependent variable explained by the regression equation and is adjusted for the sample size. The reasonably good correlation demonstrated above was felt adequate for the determination of discharge frequency curves on ungaged watersheds in the Phoenix region. Practical application for the Phoenix region has shown that the ranges of drainage area sizes for which the regression equations give consistent results are zero to 35 square miles and 35 to 417 square miles. This is noted in table 11. The "gray area" near 35 square miles should be investigated with both regression equations.

6-04. DISCHARGE FREQUENCY ANALYSIS FOR URBANIZED AREAS.

a. Urbanization of a watershed can significantly alter the runoff characteristics and hence discharge frequency relationship of a basin. As urbanization takes place, natural ground and soil are replaced with impervious materials in the form of roads, roof tops, sidewalks and parking lots. The result is that incident rainfall, which originally infiltrated into the natural ground cover, now runs off with little or no rainfall loss. Not only does more volume run off than under natural conditions, but the basin response to rainfall is generally faster because of storm drain systems and the increased hydraulic efficiency of paved surfaces. The net result of urbanization in terms of discharge frequency analysis is the generation of more runoff from the same series of storm events over what would be observed on an identical rural watershed. This phenomena produces a more positively skewed discharge frequency curve.

b. In southern Arizona there are two stream gages located on catchments with significant percentages of impervious cover. The gages are Agua Fria Tributary at Youngtown (USGS Gage No. 9-5137) and Tucson Arroyo at Vine Avenue (USGS No. 9-4830). The discharge frequency curve for the Youngtown stream gage (pl. 58) is representative of a valley watershed in Phoenix with approximately 40 percent impervious cover. The discharge frequency curve for the Tucson stream gage (pl. 59) is indicative of a more highly urbanized watershed, (60 percent impervious cover); however, the normal annual precipitation in and around Tucson is higher than the Phoenix area. The average of the ratios of the n-year flood to standard project flood for these two watersheds was used for determining discharge frequency curves for urbanized basins in the Phoenix region.

n-Year Flood	Percent of SPF for an urbanized watershed
SPF	100
100	45
50	32
25	21
10	12
5	7
2	3

6-05. DISCHARGE FREQUENCY ANALYSIS FOR WATERSHEDS WITH NON-HOMOGENEOUS SURFACE CHARACTERISTICS.

a. Along the lower reaches of New River for the future condition and along Cave Creek for both present and future conditions, the problem arises of determining non-project discharge frequency curves for watersheds which are rural in the upstream areas and urban in the downstream areas. The non-homogeneity of the surface cover (rural vs.urban) in these instances means that a combination of the discharge frequency analysis procedures discussed in previous paragraphs must be used.

b. After determining individual discharge frequency curves at a desired concentration point, resulting from each separate area (i.e.,rural and urban), two combined discharge frequency curves are drawn. One combined frequency curve assumes total independence of runoff from each of the two contributing areas, hence the frequency of occurrence of each peak discharge from both areas is summed to produce the combined frequency curve. A simplified way to visualize this approach is to imagine that storms center either over the

urban area or over the rural area, but never over both at the same time. The other combined frequency curve assumes total dependence in which all storms center over both areas, therefore, the peak discharge at the concentration point will be the sum of runoff contributions from both areas. Obviously neither of these approaches is wholly correct. The approach taken in this study was to draw a median discharge-frequency curve between these two extremes and thereby account for peak discharges which result from both storms with partial area coverage and storms which cover the entire watershed.

6-06. DISCHARGE-FREQUENCY VALUES. Utilizing the procedures described in the foregoing paragraphs on discharge frequency analysis, discharge frequency values were computed for all pertinent concentration points in each of the three plans investigated. The present condition without project discharge frequency values are presented in table 12 (see pl. 31 for concentration point locations), future condition without project discharge frequency values in table 13 (see pl. 32 for concentration point locations), and the authorized plan in the future with CAP discharge frequency values in table 6 (concentration points locations shown on pl. 33).

6-07. CAVE CREEK DAM FAILURE ANALYSIS. It should be noted that the discharge frequency analysis along Cave Creek (table 12) has been extended past the SPF to the 500-year flood event. The reason for this was to include an analysis of the probable mode of failure of Cave Creek Dam and the resulting flood hazard. Specific data on the failure potential of Cave Creek Dam will be given in DM No. 3, General Design Memorandum, Phase 1 of Plan Formulation for New River and Phoenix City Streams.

a. Although the dam will be overtopped for floodflows greater than approximately a 50-year flood, the dam was assumed to remain intact for the range of floods up to but not including the 500-year flood. In the case of the 500-year flood, the resistance of the dam foundation to an 8-hour overtopping is such that the dam would be rendered unsafe and failure was assumed to occur. The 500-year flood hydrograph was taken as SPF scaled up by the ratio of the 500-year inflow peak to the SPF peak discharge. Plate 60 shows the 500-year inflow hydrograph to Cave Creek Dam and the outflow hydrograph after reservoir routing through the dam under present conditions. Erosion of the foundation along the downstream edge of the dam would not be uniform, therefore sudden and complete failure of the dam (along its entire length) would not occur. A more reasonable failure mode would be that erosion of the foundation materials under four bays near the center of the dam would cause vertical displacement and shearing of this section away from the rest of the structure. This would leave a 4-bay breach in the structure (176 feet in width).

b. The outflow hydrograph from the dam was determined in the following manner:

(1) The peak outflow from the dam was calculated using the equation for a full depth-partial width breach found in "Military Hydrology Bulletin No. 9: Flow Through a Breached Dam," Corps of Engineers, 1957, Page 23.

(2) The peak outflow from the dam was assumed to occur in one minute after the failure. The recession leg of the hydrograph was assumed to be a straight line and was adjusted so that the volume under the triangular hydrograph was equal to the storage in the reservoir at the time of failure.

(3) Failure was assumed to occur after the dam had been overtopped for approximately 7.75 hours, or one time period prior to the point at which flow over the dam ceased. At this point, storage behind the dam was 12,470 acre-feet and the water surface was 52 feet above the bottom of the dam. Plate 61 shows the outflow hydrograph from the breached dam.

c. Routing of the outflow hydrograph was accomplished using the Muskingum routing technique. The Muskingum "X" value was set at zero which approximates a reservoir type routing. The stream channel used in flood routing from Cave Creek Dam to the Arizona Canal was deepened and widened from the present condition cross sectional shape to account for the effects of the 8-hour overtopping of the 500-year flood preceeding the dam failure. From the Arizona canal to the Salt River the cross-sectional shape of Cave Creek was assumed to remain unchanged by flood flows which occurred prior to the passage of the dam failure flood hydrograph. The presence of large amounts of impervious cover in conjunction with the fact that no defined channel exists provided the basis for the assumption that no significant change in channel cross sectional shape occurs in this reach. The 500-year flood values along Cave Creek, presented in table 12, were determined by the dam failure analysis outlined above.

VII – SEDIMENT PRODUCTION

7-01. The rate of sediment production in the study area was established from a determination of sediment deposited in Cave Creek Dam during a 47-year period (1923 to 1970). The sediment production rate was found to be 0.24 acre-feet per square mile per year. However, since the survey made in 1970 does not include debris produced during the September 1970 flood, the sediment production rate was increased to 0.30 acre-feet per square mile per year to compute sediment storage allowances at damsites. The 100-year sediment production for the proposed damsites is presented in table 14.

VIII – ADEQUACY OF DESIGN FLOODS AND SEDIMENT STORAGE

8-01. STANDARD PROJECT FLOOD. The standard project flood, as developed, is of a magnitude that would be exceeded only on rare occasions. Because of the lack of long term streamflow records, the adequacy of standard project flood is best determined from a comparison with an enveloping curve of peak discharges shown in plate 62 and table 15. The standard project floods generated at the damsites north of Phoenix are greater than the enveloping curve of observed peak flows in the region. Standard project flood peak discharges are on the order of 40 percent of the PMF peak discharges, which is reasonable for Phoenix and vicinity streams. SPF peak estimates reflect a recurrence interval of from 200 to 500 years in this region.

8-02. PROBABLE MAXIMUM FLOOD. The adequacy of probable maximum flood for the damsites is best indicated by the severity of the various hydrologic factors (storm magnitude, precipitation-intensity pattern, and loss rate) on which the flood estimate is based. The occurrence of any of these factors in the severity assumed would be infrequent, and obviously a flood resulting from the combination of all of these conditions would be very severe. An indication of the adequacy of probable maximum flood peak discharges is shown by the relatively high plotting positions on the enveloping curves of peak discharges shown on plate 62.

8-03. SEDIMENT STORAGE. The 100-year sediment storage, as developed, is reasonable for the proposed damsites in that the 0.3 acre-foot per square mile per year value was derived from a reservoir in the immediate study area.

TABLE 1

Precipitation Stations

No.	Name Station	Agency No.	Gage* (Type)	Elev. (ft.)	Latitude (deg.)	Latitude (min.)	Longitude (deg.)	Longitude (min.)	Period of Record
1.	Alhambra 2 NE, AZ	WB 0104	S	1135	33	31	112	07	1/46 to 9/73
2.	Beardsley, AZ	WB 0660	S	1265	33	40	112	23	1/50 to 9/73
3.	Black Canyon 4 NE, AZ	USGS 7	R		34	07	112	06	5/62 to 9/73
4.	Carefree, AZ	WB 1282	S	2530	33	49	111	54	11/62 to 9/73
5.	Carefree, AZ	USGS 10	R		33	49	111	55	7/64 to 9/73
6.	Castle Hot Springs Hotel, AZ	WB 1353	S	2100	33	59	112	22	3/59 to 9/73
7.	Chandler, AZ	WB 1511	S	1212	33	18	111	50	11/12 to 9/73
8.	Deer Valley, AZ	WB 2462	S	1235	33	35	112	08	1/50 to 9/73
9.	Ft. McDowell, AZ	USGS 11	R		33	35	111	40	5/62 to 9/73
10.	Falcon Field, AZ	WB 2927	S	1320	33	26	111	45	2/47 to 9/73
11.	Granite Reef Dam, AZ	WB 3621	S	1325	33	31	111	42	7/09 to 9/73
12.	Griggs 3 W, AZ	WB 3702	S	1160	33	30	112	29	1/50 to 9/73
13.	Horseshoe Dam, AZ	WB 4182	S	2020	33	59	111	43	2/48 to 9/73
14.	Lake Pleasant, AZ	WB 4770	S	1600	33	51	112	16	3/59 to 9/73
15.	Laveen, 3 SSE, AZ	WB 4829	S	1115	33	20	112	09	3/48 to 9/73
16.	Litchfield Park, AZ	WB 4977	S	1030	33	30	112	22	8/17 to 9/73
17.	Mesa, AZ	USGS 14	N		33	36	111	50	9/64 to 9/73

TABLE 1 (Continued)

Precipitation Stations

No.	Name Station	Agency No.	Gage (Type)	Elev. (ft.)	Latitude (deg.)	Latitude (min.)	Longitude (deg.)	Longitude (min.)	Period of Record
18.	Mesa Experiment Farm, AZ	WB 5467	S	1230	33	25	111	52	8/18 to 9/73
19.	Mummy Mountain, AZ	WB 5765	S	1422	33	31	111	57	11/69 to 9/73
20.	New River, AZ	USGS 16	R		33	54	112	09	12/62 to 9/73
21.	Paradise Valley 2, AZ	WB 6246	S	1375	33	34	111	58	5/55 to 9/73
22.	Skunk Creek	USGS 33	R		33	44	112	08	9/67 to 9/73
23.	Phoenix 13 N, AZ	USGS 22	N		33	38	112	03	6/69 to 9/73
24.	Phoenix 11 NNW, AZ	USGS 20	N/R		33	36	112	07	7/61 to 9/73
25.	Phoenix 11 NNE, AZ	USGS 21	N		33	37	112	03	8/62 to 9/73
26.	Phoenix 6 NE, AZ	USGS 19	N		33	30	112	01	4/63 to 9/73
27.	Phoenix 1 N, AZ	USGS 17	N		33	28	112	04	1/62 to 9/73
28.	Phoenix-Weber, AZ		R	1140	33	30	112	03	
29.	Phoenix Indian School, AZ	WB 6476	S	1115	33	30	112	04	6/28 to 9/73
30.	Phoenix Airport, AZ	WB 6481	S/R	1117	33	26	112	01	1/38 to 9/73
31.	Phoenix 6 S, AZ	USGS 18	N		33	21	112	05	4/61 to 9/73
32.	Rock Springs, AZ	USGS 26	R		34	03	112	09	10/64 to 9/73
33.	Scottsdale 8 N, AZ	USGS 29	R		33	36	111	51	5/61 to 9/73
34.	Scottsdale, AZ	WB 7661	S	1227	33	30	111	55	3/68 to 9/73
35.	Scottsdale 3 S, AZ	USGS 28	N		33	28	111	56	5/61 to 9/73
36.	South Mountain Park, AZ	USGS 30	R		33	21	112	06	10/61 to 9/73
37.	South Phoenix, AZ	WB 8112	S	1155	33	23	112	04	6/61 to 9/73
38.	Tempe, AZ	WB 8489	S	1150	33	26	111	56	1/26 to 9/73

TABLE 1 (Continued)

Precipitation Stations

No.	Name Station	Agency No.	Gage (Type)	Elev. (ft.)	Latitude (deg.)	Latitude (min.)	Longtitude (deg.)	Longtitude (min.)	Period of Record
39.	Tempe Citrus Exp. Station, AZ	WB 8499	S	1180	33	23	111	58	8/43 to 9/73
40.	Thunderbird Airport, AZ	USGS 34	R		33	36	111	55	10/61 to 9/73
41.	Tolleson 1 E, AZ	WB 1025	S	1025	33	27	112	14	9/51 to 9/73
42.	Youngtown, AZ	USGS 25D	R		33	35	112	18	10/61 to 9/73
43.	Youngtown, AZ	WB 9634	S	1135	33	35	112	18	10/64 to 9/73
44.	Wagoner, AZ	USGS 30	R		34	18	112	33	

*S — Standard 8" Non-Recording Gage
N — Other Non-Recording Gage
R — Recording Gage

Note: See plate 8 for precipitation station locations

TABLE 2

Stream Gages

Letter	Stream Gage Name	USGS No.	Drainage Area (sq. miles)	Period of Record	Maximum Discharge (c.f.s.)	Date of Maximum Discharge
A	Indian Bend Wash near Scottsdale, AZ	09512100	142	1961 to present	20,000	June 22, 1972
B	Salt River Tributary in South Mountain Park, at Phoenix, AZ	09512200	1.75	1961 to present	670	Sept. 4, 1965
C	Cave Creek near Cave Creek, AZ	09512300	121	1963 to present	12,400	Dec. 19, 1967
D	Cave Creek at Phoenix, AZ	09512400	252	1957 to present	4,080	Dec. 19, 1967
E	Agua Fria River near Rock Springs, AZ	09512800	1,130	1970 to present	40,100	Sept. 5, 1970
F	New River near Rock Springs, AZ	09513780	67.3	1962 to present	18,600	Sept. 5, 1970
G	New River at New River, AZ	09513800	85.7	1960 to present	19,500	Sept. 5, 1970
H	New River at Bell Road, near Peoria, AZ	09513835	187	1965 to present	14,600	Dec. 19, 1967
I	New River near Glendale, AZ	09513910	323	1961 to present	19,800	Dec. 19, 1967
J	Agua Fria River at Avondale, AZ	09513970	2,013	1960 to present	20,600	Sept. 6, 1970
K	Hassayampa River at Box damsite, near Wickenburg, AZ	09515500	417	1946 to present	58,000	Sept. 5, 1970
L	Skunk Creek near Phoenix, AZ	09513860	64.6	1960 to present	11,500	Aug. 1, 1964
M	Agua Fria River Tributary at Youngtown, AZ	09513700	0.13	1961 to 1968	73	Oct. 16, 1964

Note: See plate 8 for stream gage locations

TABLE 3

Stream Gages and Storms
Used in Flood Reconstitutions

Stream gage Name	USGS No.	Drainage Area	Storm Dec. 1967	Sept. 1970	June 1972
			Peak Discharge		in cfs.
1. New River near Rock Springs	9-5137.8	67.30	10500	18600	
2. New River at New River	9-5138	85.70	12500	19500	
3. New River at Bell Road	9-5138.35	187.00	14600	11900	
4. New River at Glendale	9-5139.1	323.00	19800	19200	
5. Agua Fria at Avondale	9-5139.1	718.00	20000	20600	
6. Skunk Creek near Phoenix	9-5138.6	64.60	6800	9650	
7. Cave Creek at Phoenix	9-5124	70.00	4080	780	
8. Indian Bend Wash near Scottsdale	9-5121	142.00	2000	1120	20000
9. Queen Creek Trib. at Apache Junct (Part 1)	9-4792	0.51	28	138	
(Part 2)		0.51		84.5	
10. Agua Fria Trib. at Youngtown (Part 1)	9-5137	0.13		15.8	
(Part 2)		0.13		40.5	

TABLE 4

"Best – Fit" Loss Rate and Unit Hydrograph Parameters
for Flood Reconstitutions

Gage	No.	Date	HEC Loss Function Parameters (1) STRKR	ERAIN	DLTKR	RTIOL	Clark Unitgraph (2) Parameters TC	R	TC+R	R/(TC+R)	TC/R	Antecedent Rainfall (Inches)
NR near RS	1	12/67	0.01*	0	0	2.00	0.52	3.00	3.53	0.85	0.17	3.3
NR at NR	2	12/67	0.05*	0	0	2.00	6.22	0.44	6.66	0.07	14.14	3.7
NR at BR	3	12/67	0.12*	0	0	2.00	8.90	5.15	14.05	0.37	1.73	3.5
NR at GL	4	12/67	0.12*	0	0	2.00	13.15	4.86	18.01	0.27	2.71	3.3
AF at AV	5	12/67	0.22*	0	0	2.00	19.54	2.56	22.10	0.12	7.63	3.2
SC near PH	6	12/67	0.25	0	0	2.00	3.40	0.62	4.02	0.15	5.48	3.3
CC at PH	7	12/67	0.30	0	0	2.00	7.53	1.95	9.48	0.21	3.86	3.3
IBW near SD	8	12/67	0.31	0	0	2.00	10.99	4.00	14.99	0.27	2.75	3.5
QCT at AJ	9	12/67	0.34	0	0	2.00	1.09	0.18	1.27	0.14	6.6	3.2
NR at RS	10	9/70	0.26	0	0.82	2.00	1.66	2.01	3.67	0.55	0.83	0
NR at NR	11	9/70	0.37	0	2.61	2.00	3.09	0.85	3.94	0.22	3.64	0
NR at BR	12	9/70	0.41	0	1.01	2.00	6.67	1.92	8.59	0.22	3.47	0
NR at GL	13	9/70	0.38	0	1.32	2.00	10.76	1.72	12.48	0.14	6.26	0
AF at AV	14	9/70	0.52	0	0.43	2.00	14.66	1.60	16.26	0.10	9.16	0
SC near PH	15	9/70	0.41	0	1.68	2.00	2.00	0.90	2.90	0.31	2.22	0
CC at PH	16	9/70	0.48	0	1.41	2.00	7.49	3.18	10.67	0.30	2.36	0
IBW near SD	17	9/70	0.74	0	1.79	2.00	5.42	6.79	12.21	0.56	0.80	0
AFT at YT No. 1	18	9/70	2.73	0	6.83	2.00	1.27	0.21	1.48	0.14	6.05	0
AFT at YT No. 2	19	9/70	0.66	0	2.27	2.00	0.70	0.68	1.38	0.49	1.03	0
QCT at AJ No. 1	20	9/70	1.02	0	3.16	2.00	1.37	0.19	1.56	0.12	7.21	0
QCT at AJ No. 2	21	9/70	1.12	0	5.30	2.00	1.07	0.18	1.25	0.14	5.94	0
IBW near SD	22	6/72	0.13	0	0	2.00	0.55	3.30	3.85	0.86	0.17	0.8

TABLE 4 (Continued)

"Best – Fit" Loss Rate and Unit Hydrograph Parameters
for Flood Reconstitutions

AFT	–	Agua Fria Tributary
NR	–	New River
RS	–	Rock Springs
BR	–	Bell Road
GL	–	Glendale
AV	–	Avondale
AF	–	Agua Fria
SC	–	Skunk Creek
PH	–	Phoenix
CC	–	Cave Creek
IBW	–	Indian Bend Wash
SD	–	Scottsdale
QCT	–	Queen Creek Tributary
AJ	–	Apache Junction
YT	–	Youngtown

* A significant amount of snowpack above approximately 3,000 feet elevation existed immediately prior to the period of heavy rainfall of Dec. 19, 1967. The observed rainfall loss rate was less than watershed without snowpack and proportional to the percentage of drainage area above elevation 3,000 feet.

(1) The HEC loss function parameters represent a purely mathematical function as defined in paragraph 4-04a of the text.

(2) The Clark Unit Hydrograph parameters are time of concentration (TC) and the storage coefficient (R).

TABLE 5

Muskingum Routing Coefficients For Present Without Project Conditions
Storm Centered Above CP 119 (736.62 sq. mi.)

Channel Reach	ITQR[1]	NRCHS[2]	1970** Flood	AMSKK[3]	1970** Flood	X[4]	1970** Flood	CQLOS[5]
Skunk Creek and New River System								
8 R 7*	15	8		0.25		0.3		0.022
17 R 6	15	9		0.25		0.4		0.029
167 R 5	15	9	21	0.25	0.25	0.3	0.30	0.026
5 R 2	15	3		0.25		0.2		0.008
138 R 19	15	3		0.25		0.4		0.017
4 R 19	15	9		0.25		0.3		0.047
19 R 2	15	18		0.25		0.2		0.055
2 R 1	15	5	8	0.25	0.25	0.2	0.25	0.010
1 R 16	15	4		0.25		0.2		0.008

Storm Centered Above CP 112 (174.44 sq. mi.)

Cave Creek System

Modified Puls Routing Through Cave Creek Dam
Then Muskingum Routing to CP 113

14 R 13	15	2	0.25	0.3	0.005
13 R 12	15	5	0.25	0.3	0.015
12 R 11	15	2	0.25	0.3	0.006
11 R 10	60	2	0.25	0.1	0.017
10 R 9	60	3	0.25	0.0	0.021

[1]ITQR = Routing interval in minutes.
[2]NRCHS = Number of successive reaches to be routed with identical routing specification.
[3]AMSKK = K coefficient for Muskingum routing.
[4]X = X coefficient for Muskingum routing.
[5]CQLOS = Loss as ratio of remaining outflow.

REF: HEC Generalized Computer Program "Hydrograph Combining and Routing" No. 23-J2-L132 August 1966.

* This symbolizes the reach from subarea or concentration point "A" to subarea or concentration point "B" ("A" R "B").

** Values are Muskingum coeffieients derived from recorded floodflows of September 1970 for New River at Rock Springs, New River at New River, New River at Bell Road, New River near Glendale and Agua Fria River at Avondale.

Refer to plates 34 and 35 for identification of reaches.

TABLE 6

Standard Project Flood, 100-Year and 50-Year Peak Discharges For Authorized Plan Under Future Conditions with Central Arizona Project (In Thousands of Cubic Feet Per Second)

Concentration Point	SPF	100 YR	50 YR
1001	160	72	51
1002U*	145	65	46
1002D *	155	69	49
1002R*	34	15	11
1003U*	120	55	39
1003D*	135	60	43
1003R*	25	11	8.1
1007	155	70	50
1010	33	15	11
1012	125	55	39
1013U*	76	53	39
1013D*	2.6	2.2	1.9
1014	53	24	17
1015	72	32	23
1016U*	30	14	9.6
1016D*	60	27	19
1017U*	65	48	36
1017D*	1.9	1.6	1.4
1018	31	14	9.9
1019	14	6.3	4.5
1020	19	8.3	5.9
1021	26	12	8.3
1023	26	12	8.3
1024	13	5.8	4.1
1025	23	10	7.3
1026U*	86	61	45
1026D*	0.75	0.63	0.54
1027	10	4.4	3.1
1028	30	14	9.6
1030U*	3.6	1.7	1.2
1030D*	0.20	0.18	0.17
1031	6.4	2.9	2.1

TABLE 6 (Continued)

Standard Project Flood, 100-Year and 50-Year Peak Discharges For Authorized Plan Under Future Conditions with Central Arizona Project (In Thousands of Cubic Feet Per Second)

Concentration Point	SPF	100 YR	50 YR
1033	27	12	8.8
1036	130	58	41
111U*	115	52	37
111D*	125	57	41
121	40	18	13
131U*	2.3	1.1	0.74
131D*	6.6	3.0	2.1
145	24	11	7.5
152	160	73	52
154	110	50	35
163	23	10	7.2

*U = Upstream; D = Downstream; R = Right of confluence

Note: Refer to plate 33 for concentration locations.

TABLE 7

Subarea Drainage Characteristics
Present Without Project Conditions

Subarea	Drainage Area (Sq. Mi.)	L* (mi.)	Lca* (Mi.)	Slope* (Ft./Mi.)	n	Present Impervious Cover	LAG* (Hrs.)	Unit Hydrograph Peak (cfs)
1	112.36	24.50	9.60	25.3	0.035	21	3.62	5700
2	86.33	16.10	10.70	18.9	0.030	13	2.91	3600
3	8.67	9.10	4.80	86.8	0.030	5	1.30	1000
4	72.00	23.10	11.20	85.1	0.034	5	2.90	4300
5	24.72	10.20	5.00	20.6	0.029	12	1.75	2500
6	36.58	11.50	5.15	46.4	0.030	5	1.64	3800
7	6.51	6.40	3.30	22.6	0.030	5	1.27	820
8	103.65	24.90	13.20	132.1	0.033	5	2.83	6600
9	46.15	12.50	5.90	75.6	0.055	50	2.97	2800
10	20.03	7.50	4.30	45.3	0.055	48	2.39	1400
11	9.79	7.10	3.96	80.3	0.032	11	1.19	1400
12	40.03	23.70	10.24	79.4	0.030	5	2.53	2800
13	20.12	18.10	7.20	116.7	0.030	5	1.85	1800
14	174.44	32.10	17.10	88.5	0.038	5	4.28	7400
15	53.24	17.90	10.40	21.2	0.030	5	2.94	3200
16	11.34	6.92	3.20	12.3	0.030	5	1.45	1300
17	167.84	37.80	18.00	92.4	0.037	5	4.48	6700
18	238.50	32.00	14.00	80.9	0.032	5	3.43	12000
19	18.99	8.90	3.90	33.7	0.030	5	1.42	2300
20	69.04	17.00	8.00	93.5	0.031	5	2.03	5900

* See plate 27 for explanation.
Refer to plate 34 for location of subareas.

TABLE 8

Channel Characteristics For Present Without Project Conditions

Channel Reach	Length (miles)	Invert Slope (ft./ft.)	Present Manning's N Value Channel	Overbank	Channel Shape	Channel Type
8 R 7*	6.29	0.0044	0.030	0.040	Trap.	Natural
17 R 6	8.64	0.0049	0.030	0.040	Trap.	Natural
4 R 19	5.27	0.0031	0.030	0.040	Trap.	Natural
18 R 20	5.88	0.0006	0.026	0.034	Trap.	Natural
14 R 13	2.35	0.0077	0.030	0.040	Trap.	Natural
8 R 1	17.55	0.0036	0.040	0.040	Trap.	Natural
167 R 5	7.20	0.0033	0.030	0.040	Trap.	Natural
19 R 2	10.12	0.0027	0.030	0.040	Trap.	Natural
12 R 11	2.50	0.0064	0.030	0.040	Trap.	Natural
5 R 2	2.46	0.0027	0.030	0.040	Trap.	Natural
2 R 1	3.03	0.0019	0.030	0.040	Trap.	Natural
1 R 16	2.92	0.0023	0.030	0.040	Trap.	Natural
11 R 10	5.12	0.0043	0.100	0.100	Trap.	Urban
10 R 9	4.78	0.0026	0.100	0.100	Trap.	Urban
138 R 19	5.57	0.0054	0.030	0.040	Trap.	Natural
13 R 12	6.14	0.0057	0.030	0.040	Trap.	Natural
12 R 1	17.36	0.0038	0.040	0.040	Trap.	Natural
20 R 138	Zero					
3 R 138	Zero					
6 R 167	Zero					
7 R 167	Zero					

* This symbolizes the reach from subarea or concentration point "A" routed to subarea or concentration point "B". ("A" R "B")

Refer to plates 34 and 35 for identification of reaches.

TABLE 9

Stream gages and Physical Parameters (D.A. = 0 to 15 Square Miles)

Observation Number	Stream gage Number	Station	Authority	Period of Record	Drainage Area (sq. mi.)	Slope (ft./mi.)	5-Year 24-hour Rainfall (inches)
1	94792	Queen Creek Tributary at Apache Junction	USGS	1961-71	0.51	67	1.98
2	95012	Mesquite Creek near Mormon Flat Dam	USGS	1963-67	4.18	239	2.77
3	95100.7	West Fork Sycamore Creek ABV McFairland Canyon near Sunflower	USGS	1966-71	4.58	1437	3.74
4	95100.8	West Fork Sycamore Creek near Sunflower	USGS	1962-71	9.80	603	3.73
5	95101	East Fork Sycamore Creek near Sunflower	USGS	1962-71	4.49	1030	3.70
6	95101.8	Rock Creek near Sunflower	USGS	1963-71	15.00	340	2.80
7	95122	Salt River Trib. in South Mountain Park	USGS	1961-71	1.75	397	2.20
8	95127	Agua Fria River Trib. No. 2 near Rock Springs	USGS	1963-70	1.00	400	2.82
9	95137	Agua Fria River Trib. at Youngtown	USGS	1961-68	0.13	16	1.98
10	95138.2	Deadman Wash at Black Canyon Highway) near New River	USGS	1960-71	11.10	319	2.48
11	95158	Hartman Wash near Wickenburg	USGS	1964-71	5.57	125	2.23
12	95166	Ox Wash near Morristown	USGS	1963-71	6.31	150	2.40
13	95172	Centennial Wash Tributary near Salome	USGS	1963-71	3.58	288	2.18
14	95196	Rainbow Wash Tributary near Buckeye	USGS	1963-71	3.45	122	2.07
15	95201	Military Wash near Sentinel	USGS	1963-71	8.70	25	1.98
16	95202.3	Crater Range Wash near Ajo	USGS	1963-71	1.49	184	2.23
17	—	Safford W-V	ARS*	1939-71	1.13	130	2.00
18	—	Safford W-IV	ARS*	1939-71	1.19	54	2.00
19	—	Safford W-II	ARS*	1939-71	1.07	324	2.00
20	—	Safford W-I	ARS*	1939-71	0.81	80	2.00

*(ARS) — Agricultural Research Service, U.S. Department of Agriculture. Years in "Period of Record" are calendar years.

TABLE 10
Stream gages and Physical Parameters (D.A. = 15 to 417 Square Miles)

Observation Number	Stream gage Number	Station	Authority	Period of Record	Drainage Area (sq. mi.)	Slope (ft./mi.)	5-year 24-hour Rainfall (inches)
1	94988.7	Rye Creek near Gisela	USGS	1966-71	122	287	3.42
2	95101.5	Sycamore Creek near Sunflower	USGS	1962-70	53.4	368	3.70
3	95102	Sycamore Creek near Fort McDowell	USGS	1961-72	165	206	3.25
4	95121	Indian Bend near Scottsdale	USGS	1961-72	142	64	2.15
5	95123	Cave Creek near Cave Creek	USGS	1958-72	121	127	2.70
6	95137.8	New River near Rock Springs	USGS	1962-72	67.3	141	2.90
7	95138	New River at New River	USGS	1961-72	85.7	145	2.84
8	95138.35	New River (at Bell Road) near Peoria	USGS	1963-72	187	83	2.45
9	95138.6	Skunk Creek near Phoenix	USGS	1961-72	64.6	89	2.40
10	95138.9	New River (at Grand Avenue) at Peoria	USGS	1960-71	317	88	2.42
11	95139.1	New River near Glendale	USGS	1961-72	323	74	2.40
12	95142	Waterman Wash near Buckeye	USGS	1964-71	403	84	2.25
13	95155	Hassayampa River near Wickenburg	USGS	1938,1946-72	417	108	2.80
14	95168	Jack Rabbit Wash near Tonopah	USGS	1964-71	137	78	2.17
15	95172.8	Tiger Wash near Aguila	USGS	1963-71	85.2	47	2.17
16	95197.5	Bender Wash near Gila Bend	USGS	1963-71	68.8	43.4	2.26
17	95197.6	Sauceda Wash near Gila Bend	USGS	1963-71	126	62	2.23
18	94685	San Carlos River near Peridot	USGS	1916,1930-71	320	127	2.70
19	94800	Santa Cruz River near Lochiel	USGS	1949-71	80.6	57	2.00
20	94815	Sonoita Creek near Patagonia	USGS	1930-71	209	300	2.58
21	94840	Sabino Creek near Tuscon	USGS	1932-71	35.5	434	2.80
22	94831	Tanque Verde Creek near Tucson	USGS	1960-71	43.0	103	2.56
23	94850	Rincon Creek near Tucson	USGS	1953-71	44.8	638	2.70

TABLE 11

Regional Flood Frequency Regression Equations

Regression Equations for Drainage Area Sizes 0 to 35 Square Miles

Return Period Years	C1	C5	C9	Constant	R Squared	R Bar Squared
100	6580.04	0	20226.14	-3972.32	0.7871	0.7621
50	4539.83	0	13523.12	-2461.47	0.7919	0.7674
25	2987.84	0	7889.85	-1179.13	0.7447	0.7147
10	1541.98	0	2793.95	-85.86	0.5547	0.5023
5	815.86	0	643.13	+283.51	0.3301	0.2513
2	—	—	—	—	0	0

for Drainage Area Sizes 35 to 417 Square Miles

100	41750.34	-10966.70	214649.68	-107165.82	0.7148	0.6697
50	30366.44	-13048.91	185458.49	-79977.25	0.7086	0.6626
25	20596.45	-10846.21	139951.40	-55538.28	0.6680	0.6156
10	11674.26	-5949.96	80139.74	-32835.46	0.6123	0.5511
5	6779.87	-2717.80	45087.47	-20399.96	0.5757	0.5087
2	2641.09	30.45	12027.60	-8540.39	0.4335	0.3440

Note: General regression equation is:

$$Q = C1 (\log DA) + C5 (\log S) + C9 (\log 5R24) + Constant$$

Where: DA = drainage area in square miles
5R24 = 5 year 24 hour rainfall in inches
S = basin slope in ft/mile
C1 = DA regression coefficient
C5 = S regression coefficient
C9 = 5R24 regression coefficient
Constant = regression constant
Q = return period peak discharge in c.f.s.

Example:

New River at Bell Road near Peoria, Arizona USGS Gage 9-1538.35
DA = 187 square miles; S = 83.4 ft. per mi.; 5R24 = 2.45 inches

Return Period Years	Actual Flow cfs (see plate 55)	Computed Flow cfs
100	53,000	50,000
50	40,000	36,000
25	28,000	25,000
10	15,000	13,000
5	7,500	7,300
2	1,500	2,200

TABLE 13

Discharge Frequency Values For Future
Without Project Conditions

Discharge In C.F.S.

Exceedence Interval

Location	2 year	5 year	10 year	25 year	50 year	100 year	SPF
Gage 95123 Cave Creek near Cave Creek, AZ	2000	6500	12000	22000	33000	45000	61000
Cave Creek at Cave Creek Dam. Upstream face. CP112U	1700	8500	17000	33000	47000	63000	86000
Cave Creek at Cave Creek Dam. Downstream face. CP112D	600	700	700	16000	35000	55000	80000
Cave Creek at CP113	600	800	1100	14000	32000	53000	78000
Cave Creek at Greenway Road CP114	600	1300	2400	12000	28000	46000	74000
Gage 95124 Cave Creek at Arizona Canal CP115	700	1700	3500	9500	21000	39000	50000
Cave Creek at Grand Canal CP116	700	1700	3500	10000	21000	38000	46000
Cave Creek at Buckeye Road CP117	1300	2800	5000	12000	21500	35000	37000
Gage 95138.6 Skunk Creek near Phoenix	1100	5500	11000	20000	30000	39000	54000
Skunk Creek at Hedgpeth Hills CP101	1200	6600	13000	23000	34000	45000	60000
Skunk Creek above New River Confluence CP102	1200	6600	13000	23000	34000	45000	60000
Gage 95137.8 New River near Rock Springs	2400	9000	15000	25000	33000	41000	49000
Gage 95138 New River at New River	2800	10000	18000	28000	37000	45000	55000
New River near West Wing Mountain CP103	1500	7400	15000	28000	39000	53000	76000
Gage 95138.35 New River at Bell Road near Peoria CP104	1500	7400	15000	28000	39000	53000	75000
New River below Confluence with Skunk Creek CP167	1700	8400	17000	31000	44000	58000	86000
Gage 95139.1 new River near Glendale CP105	2200	8100	15000	28000	39000	53000	82000
Auga Fria River CP106	1000	5200	10000	19000	26000	35000	52000

TABLE 13 (Continued)

Discharge Frequency Values For Future
Without Project Conditions

Discharge in C.F.S.

Exceedence Interval

Location	2 year	5 year	10 year	25 year	50 year	100 year	SPF
Auga Fria River CP109	1600	8000	16000	29000	40000	54000	78000
Auga Fria River CP110	3400	8800	16000	29000	41000	55000	90000
Auga Fria River CP118	3400	8800	16000	28000	40000	54000	89000
Gage 95139.7 Auga Fria River at Avondale CP119	3400	8800	16000	28000	39000	53000	88000

Refer to plate 32 for concentration point locations.

TABLE 14

100-Year Sediment Production at Damsites

Dams	Drainage Area (sq. mi.)	100-Year Sediment Production (ac. - ft.)
Cave Buttes Dam	195	5900
Adobe Dam	75.6	2300
New River Dam	164	4900

TABLE 15
Peak Discharges for Major Recorded Floods
in Phoenix and Vicinity

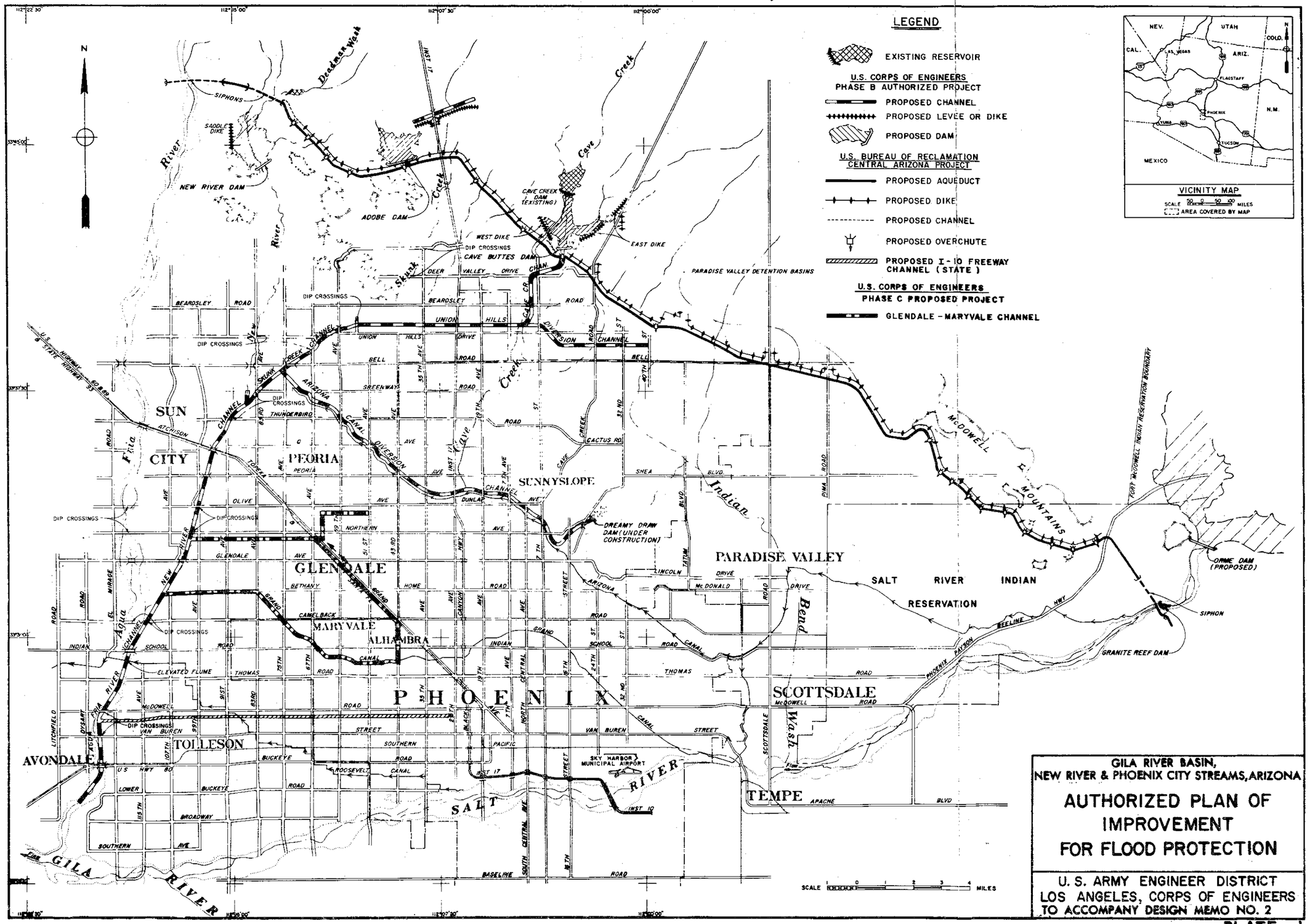
Number	Location	Date	Drainage Area (sq. mi.)	Peak Discharge (c.f.s.)	Discharge (c.f.s./sq. mi.)
1	Shea Wash, at Shea Blvd. NR Scottsdale	6/22/72	1.79	945	528
2	Shea Wash Trib. No. 3, at Shea Blvd. NR Scottsdale	6/22/72	0.09	86	956
3	Shea Wash Trib. No. 2, at Shea Blvd. NR Scottsdale	6/22/72	0.14	103	736
4	Shea Wash Trib. No. 1, at Shea Blvd. NR Scottsdale	6/22/72	0.12	80	667
5	Indian Bend Wash Trib. No. 1, at Tatum Blvd. in Paradise Valley	6/22/72	0.026	88	3,400
6	Indian Bend Wash Trib. No. 2, at Tatum Blvd. in Paradise Valley	6/22/72	0.075	144	1,920
7	Indian Bend Wash, at Camelback Country Club in Paradise Valley	6/22/72	83	14,500	175
8	Indian Bend Wash (at Indian Bend Road) NR Scottsdale (USGS Gaging Station)	6/22/72	142	20,000	141
9	Cudia City Wash NR Phoenix, 1,000 ft. upstream from McDonald Drive	6/22/72	2.16	4,200	1,940
10	Cudia City Wash Trib., at 40th St. and Rancho Drive	6/22/72	0.08	219	2,750
11	Dreamy Draw at Phoenix, at 16th St.	6/22/72	1.24	860	694
12	Hassayampa R. at Box Damsite (NR Wickenburg)	Sept. 1970	417	58,000	139
13	New River NR Glendale	Sept. 1970	323	19,200	59
14	New River NR Rock Springs	Sept. 1970	67.3	18,600	276

TABLE 15 (Continued)

Peak Discharges for Major Recorded Floods
in Phoenix and Vicinity

Number	Location	Date	Drainage Area (sq. mi.)	Peak Discharge (c.f.s.)	Discharge (c.f.s./sq. mi.)
15	New River at Glendale	Dec. 1967	323	19,800	61
16	Skunk Cr. NR Phoenix	Dec. 1964	64.6	11,500	178
17	New River at New River	Dec. 1970	85.7	19,500	228
18	Cave Cr. NR Cave Cr.	Dec. 1967	121	12,400	102
19	New River NR Rock Springs	Dec. 1967	67.3	10,600	158
20	New River at Peoria	Aug. 1943	187	38,000	203
21	Agua Fria R. at Avondale	Jan. 1916	2,400	105,000	44
22	Agua Fria R. at Avondale	Nov. 1919	2,400	105,000	44
23	Sycamore Cr. NR Ft. McDowell	Sept. 1970	165	24,200	147
24	Queen Cr. at Whitlow Ranch Dam Site NR. Superior	Aug, 1954	144	42,900	298
25	Rock Creek NR Sunflower	Dec. 1965	15.2	1,900	125
26	E. Fork Sycamore Cr. NR Sunflower	Sept 1970	4.49	1,940	432
27	W.Fork Sycamore Cr. NR Sunflower	Sept. 1970	9.8	3,480	355

Refer to plate 58 for enveloping curve.

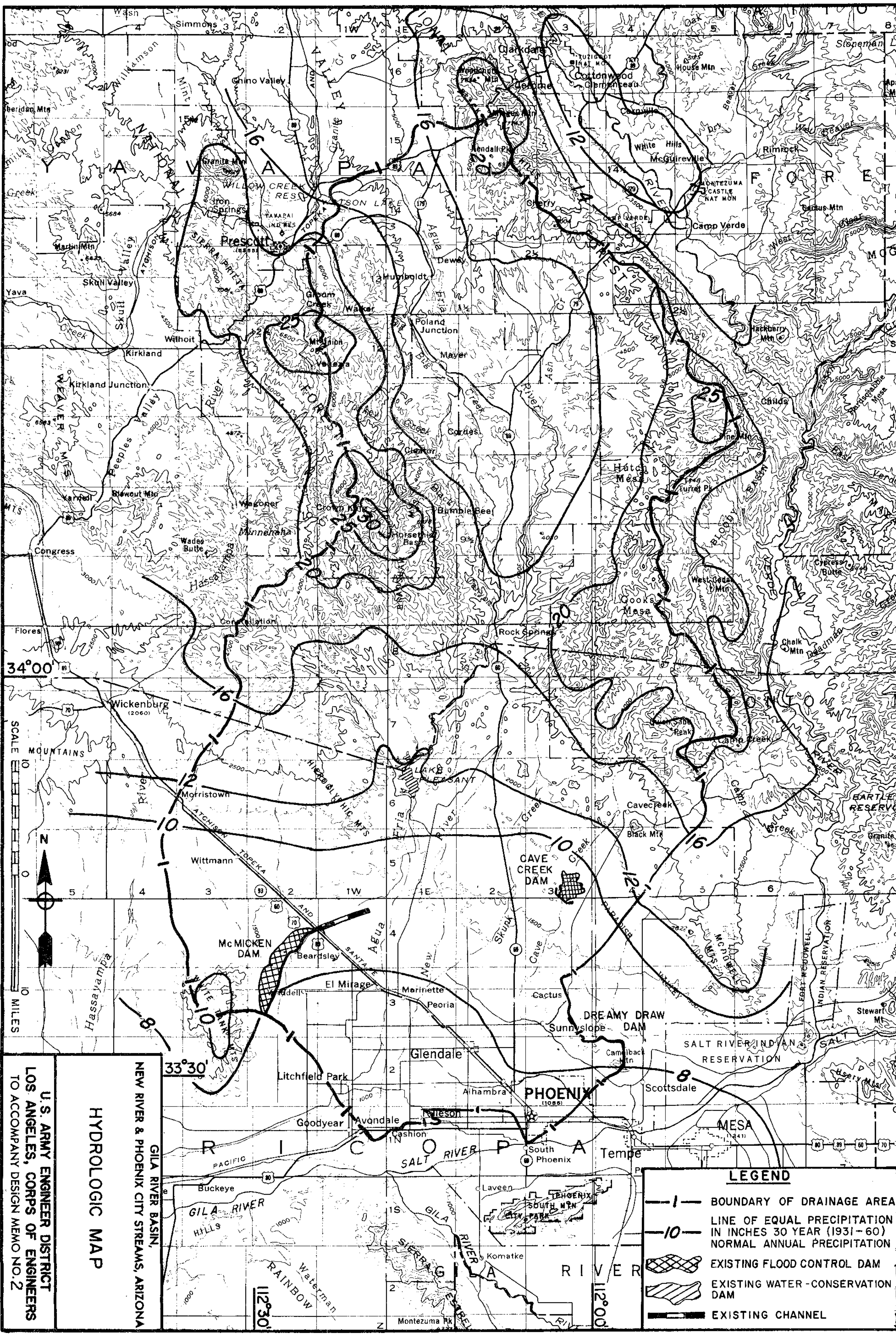


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

AUTHORIZED PLAN OF IMPROVEMENT FOR FLOOD PROTECTION

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 1

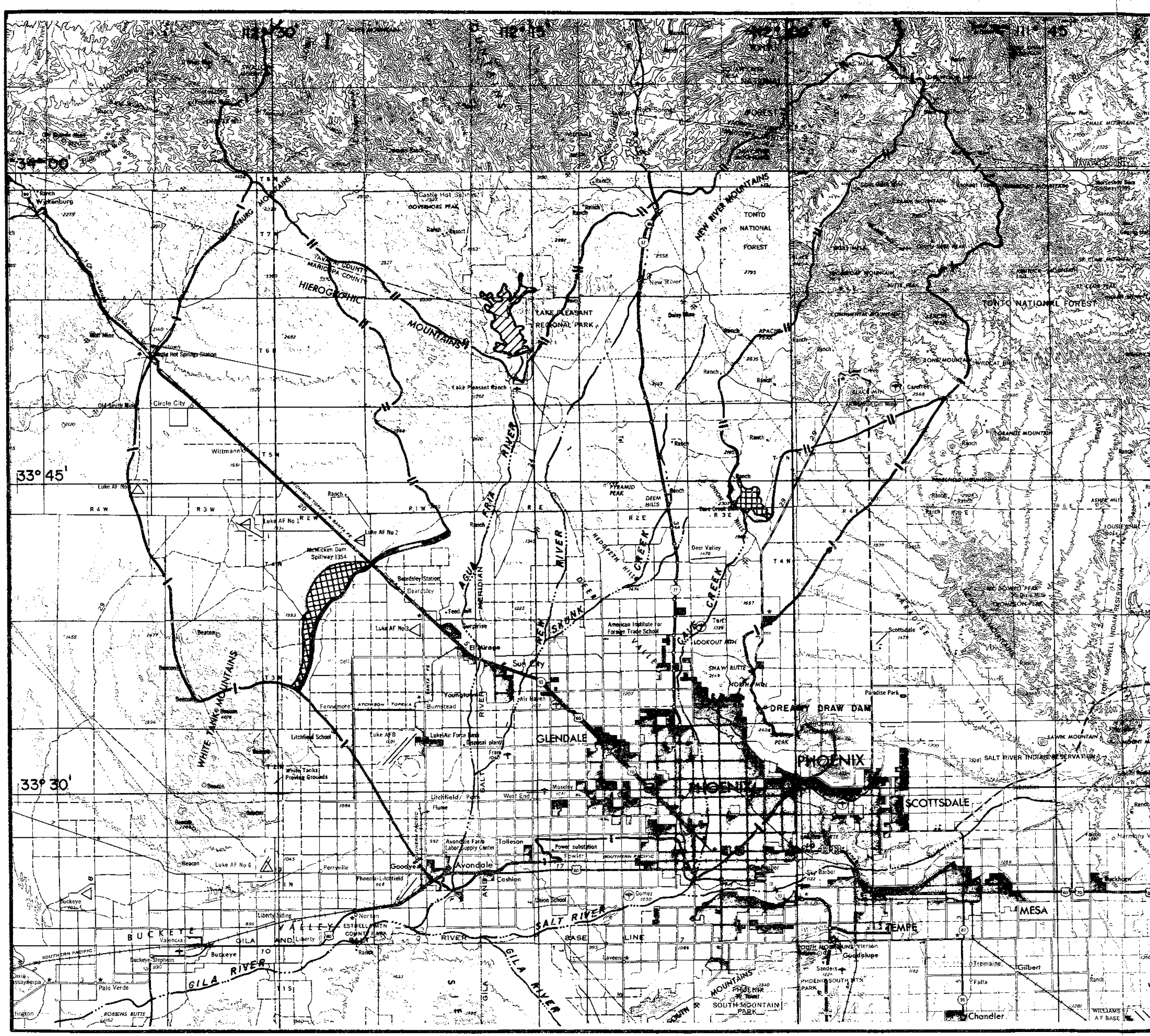


U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

HYDROLOGIC MAP

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

PLATE 2



LEGEND

BOUNDARY OF DRAINAGE AREA

BOUNDARY OF DRAINAGE SUBAREA

EXISTING FLOOD CONTROL DAM

EXISTING WATER CONSERVATION DAM

EXISTING CHANNEL

SCALE

5 0 5 10 Miles

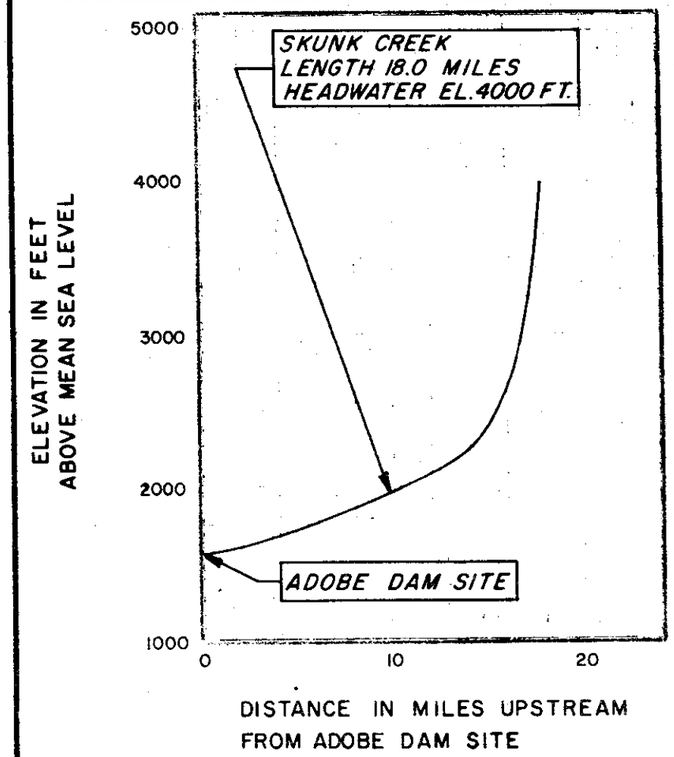
N

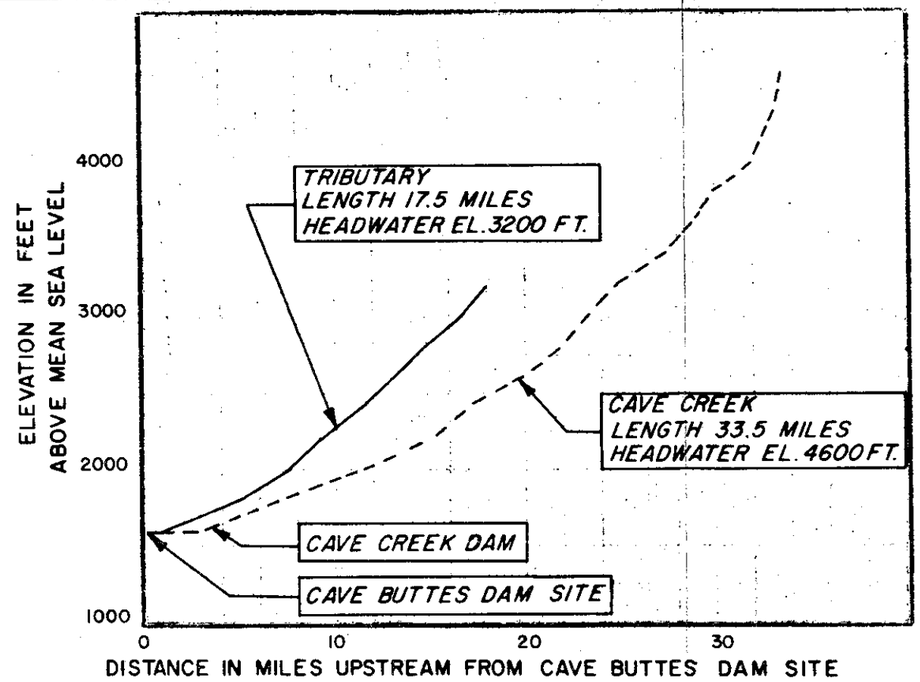
GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

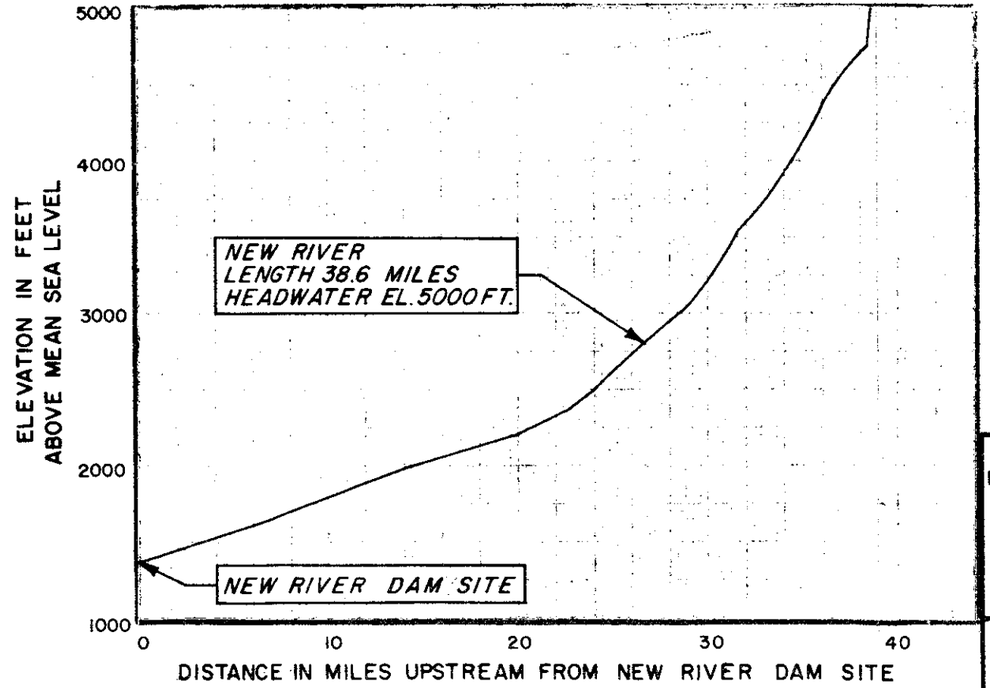
TOPOGRAPHIC MAP

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 3



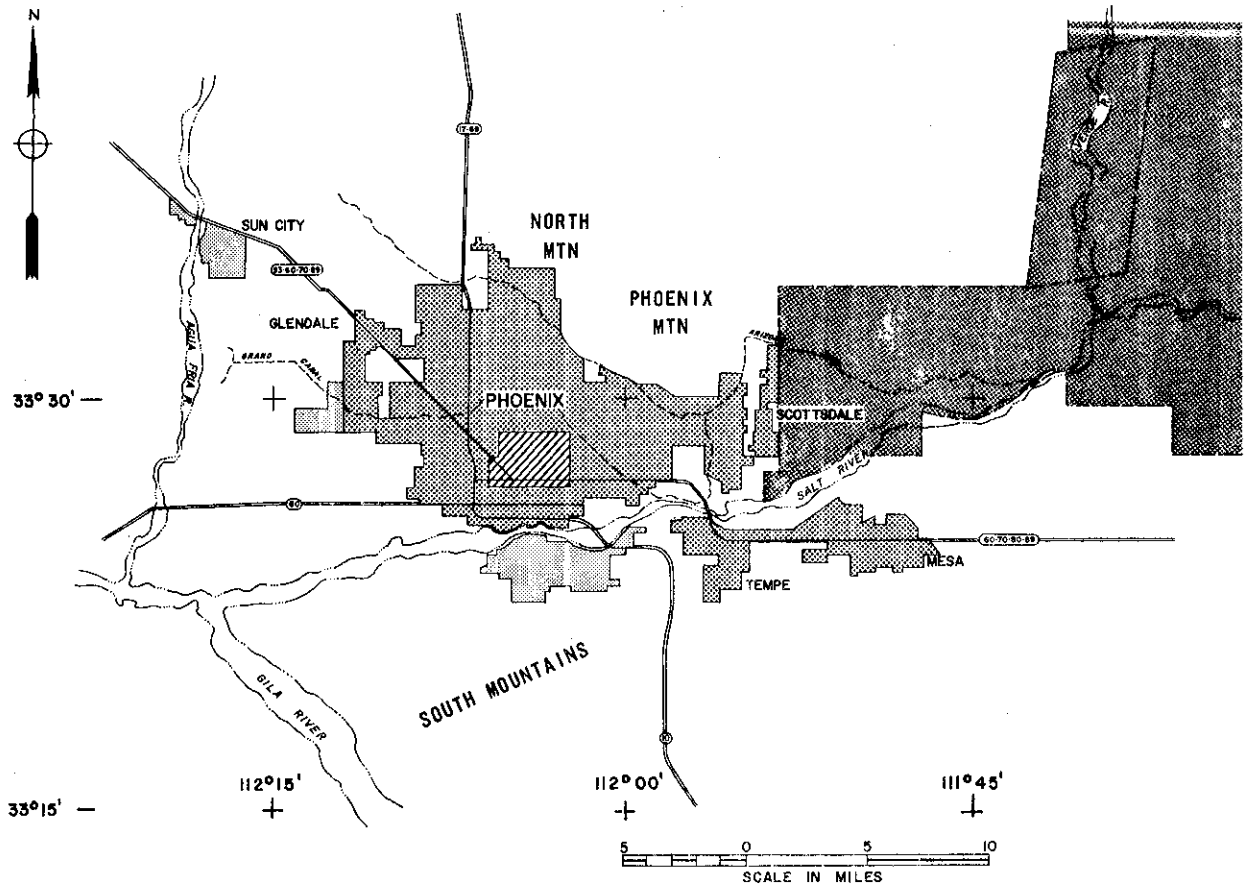




GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

STREAMBED PROFILES

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



IMPERVIOUS COVER

USE 5 PERCENT FOR UNSHADED AREAS

80% 65% 50%

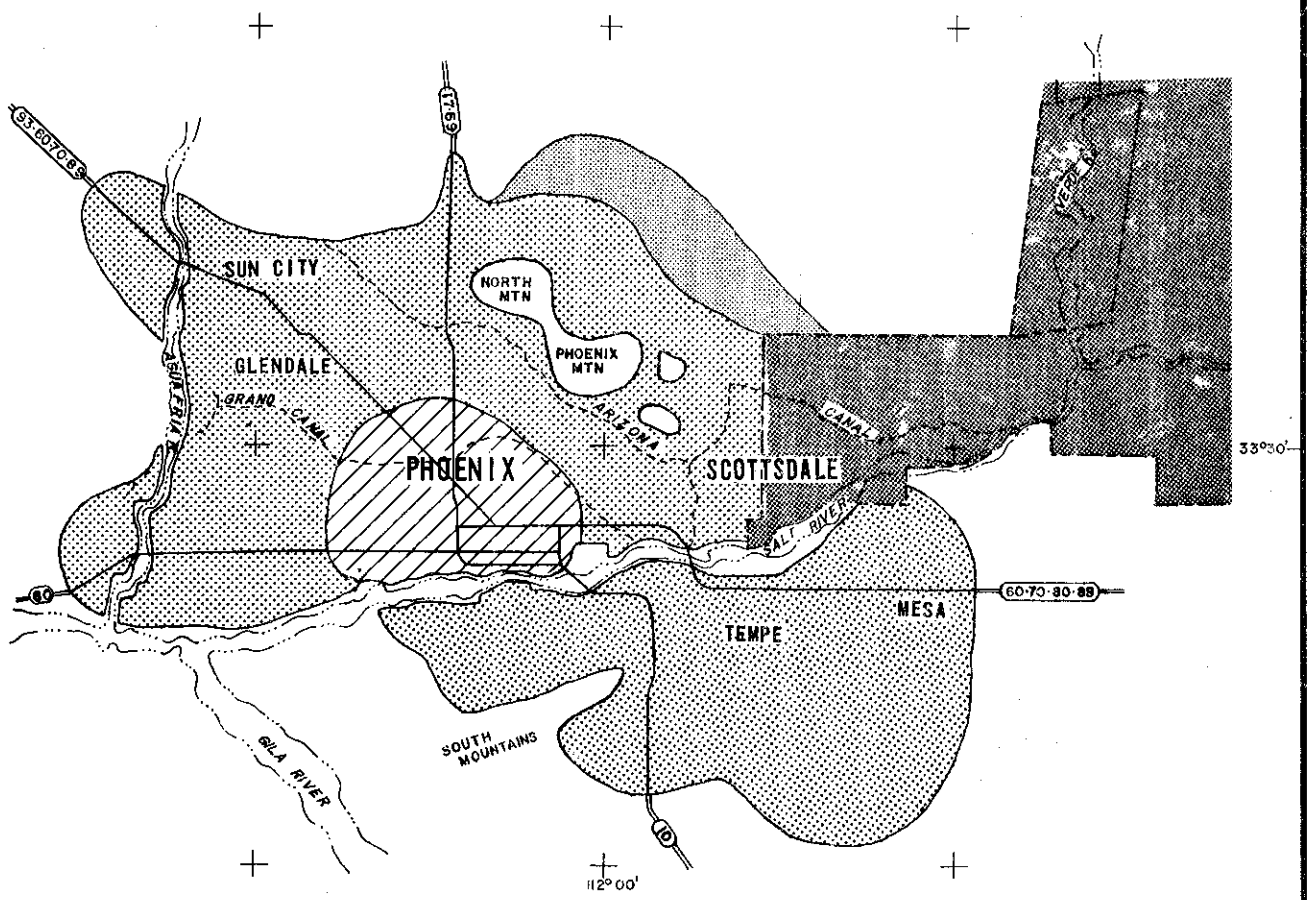
5% INDIAN RESERVATION LANDS

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

PERCENT IMPERVIOUS COVER

PRESENT CONDITIONS
(1977)

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



IMPERVIOUS COVER

USE FIVE PERCENT FOR UNSHADED AREAS

85% 65% 40%

5% INDIAN RESERVATION LANDS (DEVELOPMENT POTENTIAL UNCERTAIN)

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

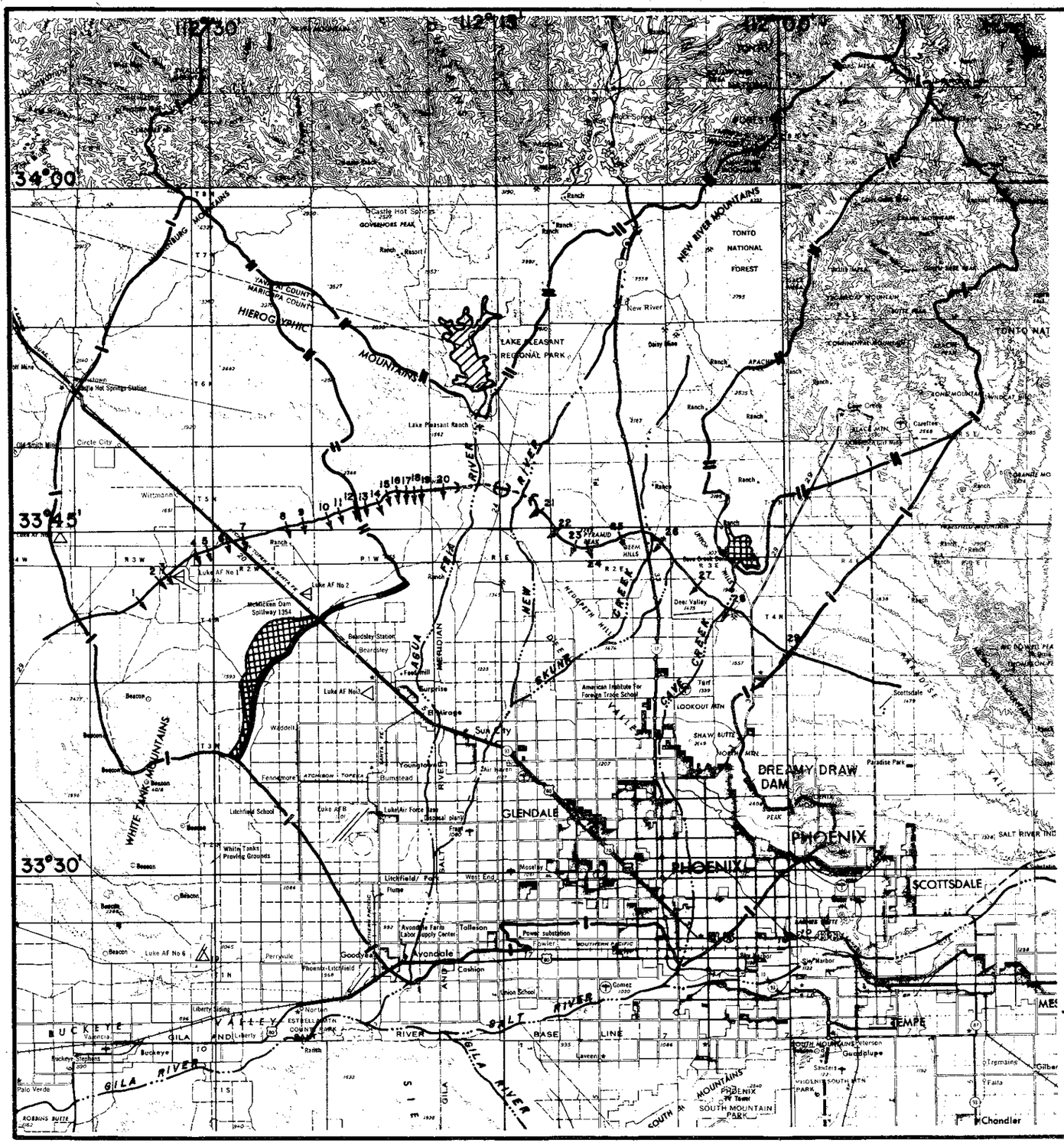
PERCENT IMPERVIOUS COVER

FUTURE CONDITIONS
(2077)

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

5 0 5 10
SCALE IN MILES

PLATE 6



CROSS-DRAINAGE STRUCTURES

#	Drainage Area (Sq. Mi.)	USBR 50-Year Peak Discharge (CFS)
1	16.10	4140
2 & 3	56.07	7410
4	17.52	4330
5	13.54	3825
6	16.69	4215
7	12.54	3675
8 & 9	34.30	5930
10	1.87	1375
11	22.52	4860
12	0.88	1270
13	3.39	2930
14	0.48	890
15	0.55	840
16	3.17	2380
17	0.19	475
18	0.17	425
19	1.16	1160
20	0.37	725
21	0.24	515
22	1.48	885
23	0.88	950
24	0.49	430
25	71.20	2000
26	3.65	4420
27	3.05	3510
28	206.80	9395
29	9.53	4180

LEGEND

—I— BOUNDARY OF DRAINAGE AREA

—II— BOUNDARY OF DRAINAGE SUBAREA

(FLOW) CROSS DRAINAGE STRUCTURE

SIPHON

SCALE IN MILES

10 0 10

N

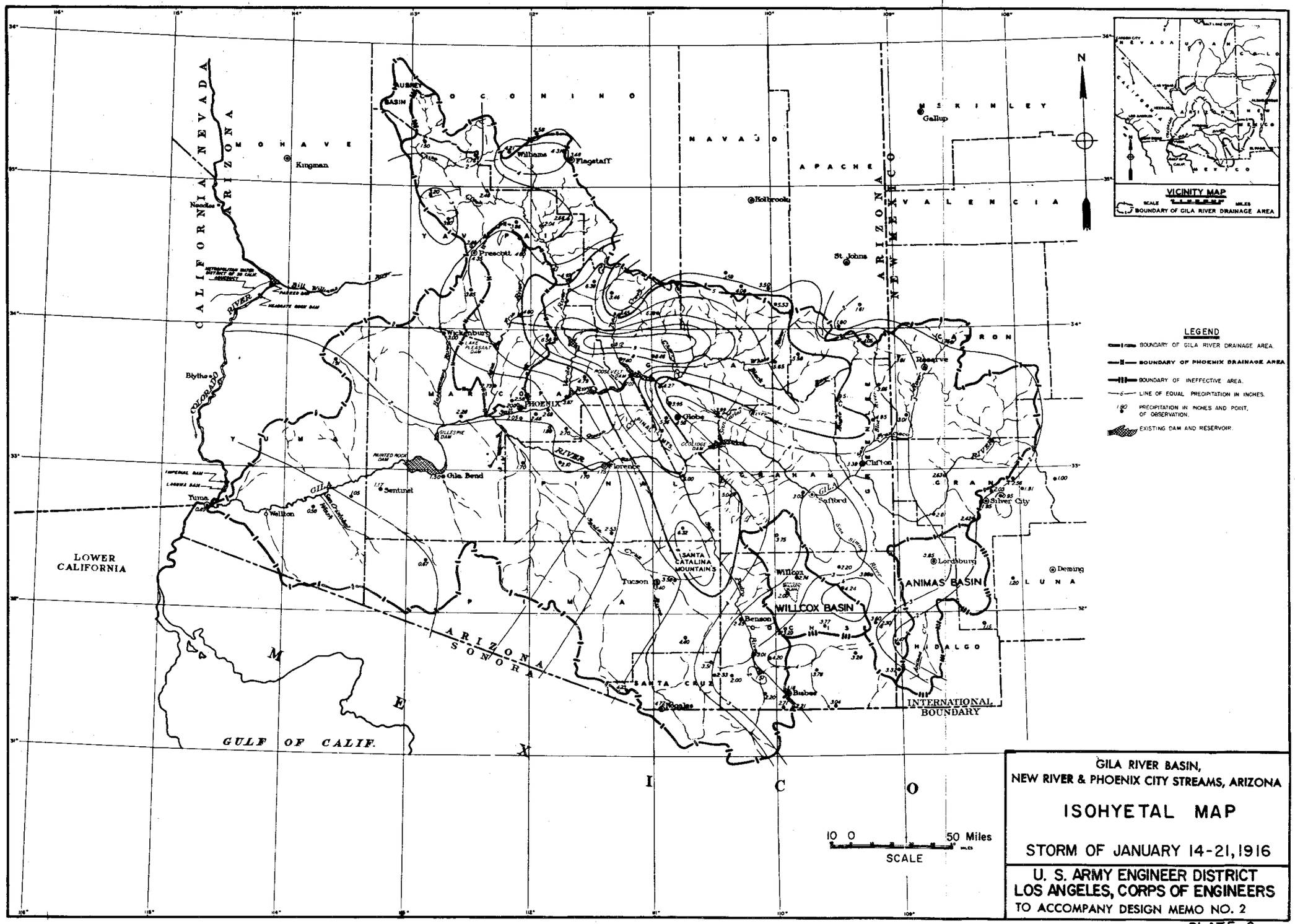
NOTE:
3 Jan.72
ALIGNMENT

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

CENTRAL ARIZONA PROJECT
(CAP)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 7



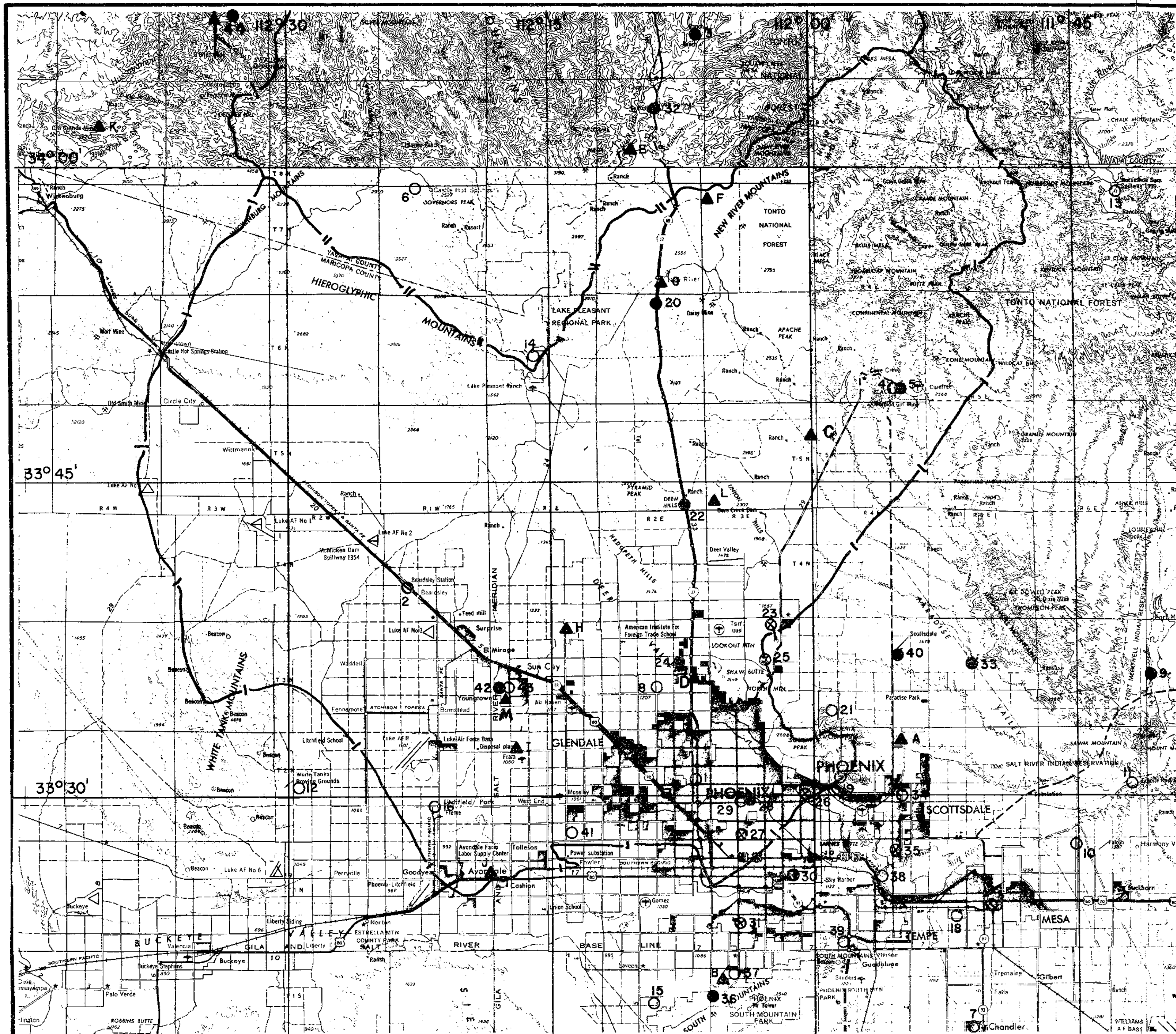
GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

ISOHYETAL MAP

STORM OF JANUARY 14-21, 1916

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 9



LEGEND

—I— BOUNDARY OF DRAINAGE AREA.

—II— BOUNDARY OF DRAINAGE SUBAREA.

● RECORDING GAGE

○ STANDARD 8" NON-RECORDING GAGE.

⊗ OTHER NON-RECORDING GAGE.

◐ RECORDING + STANDARD 8" NON-RECORDING GAGES.

▲ STREAMGAGE

N

SCALE IN MILES

5 0 5 10

NOTE: REFER TO TABLES 1 & 2

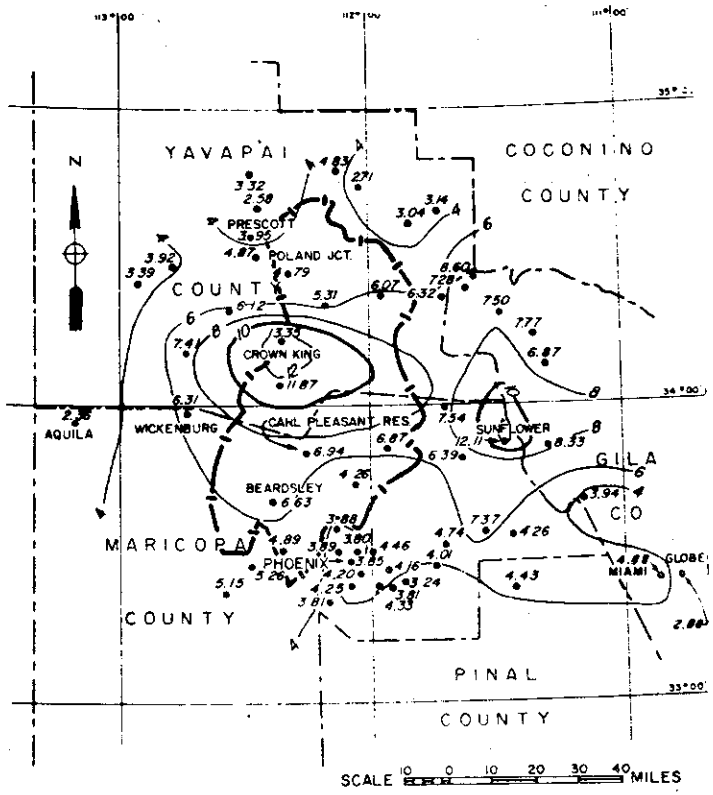
GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

PRECIPITATION AND STREAMGAGE STATIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 8



LEGEND

—1— BOUNDARY OF DRAINAGE AREA.

—8— LINE OF EQUAL PRECIPITATION IN INCHES.

● 6.31 RECORDED PRECIPITATION DEPTH IN INCHES.

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

ISOHYETAL MAP

STORM OF AUGUST 26-29, 1951

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 10

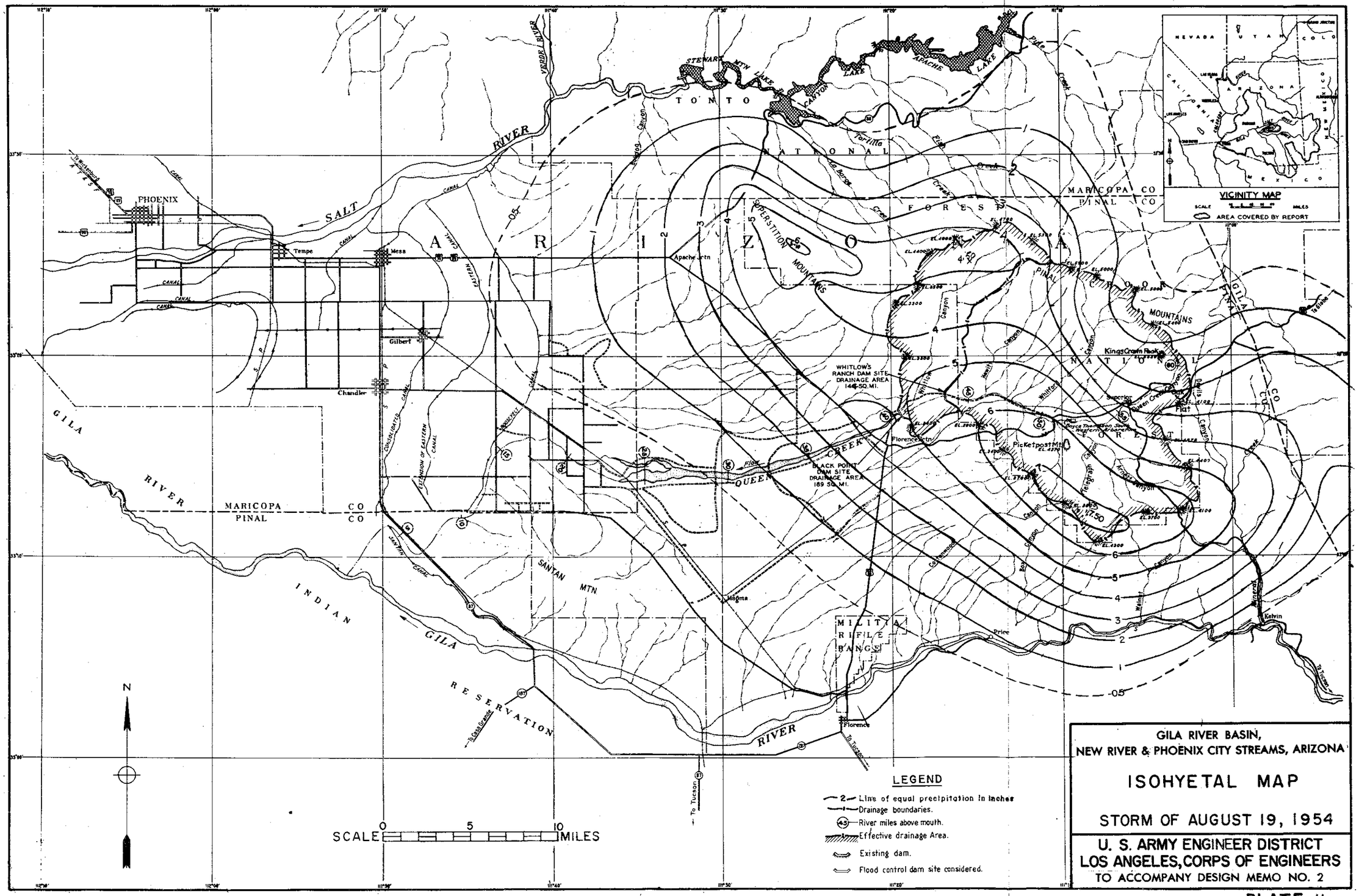
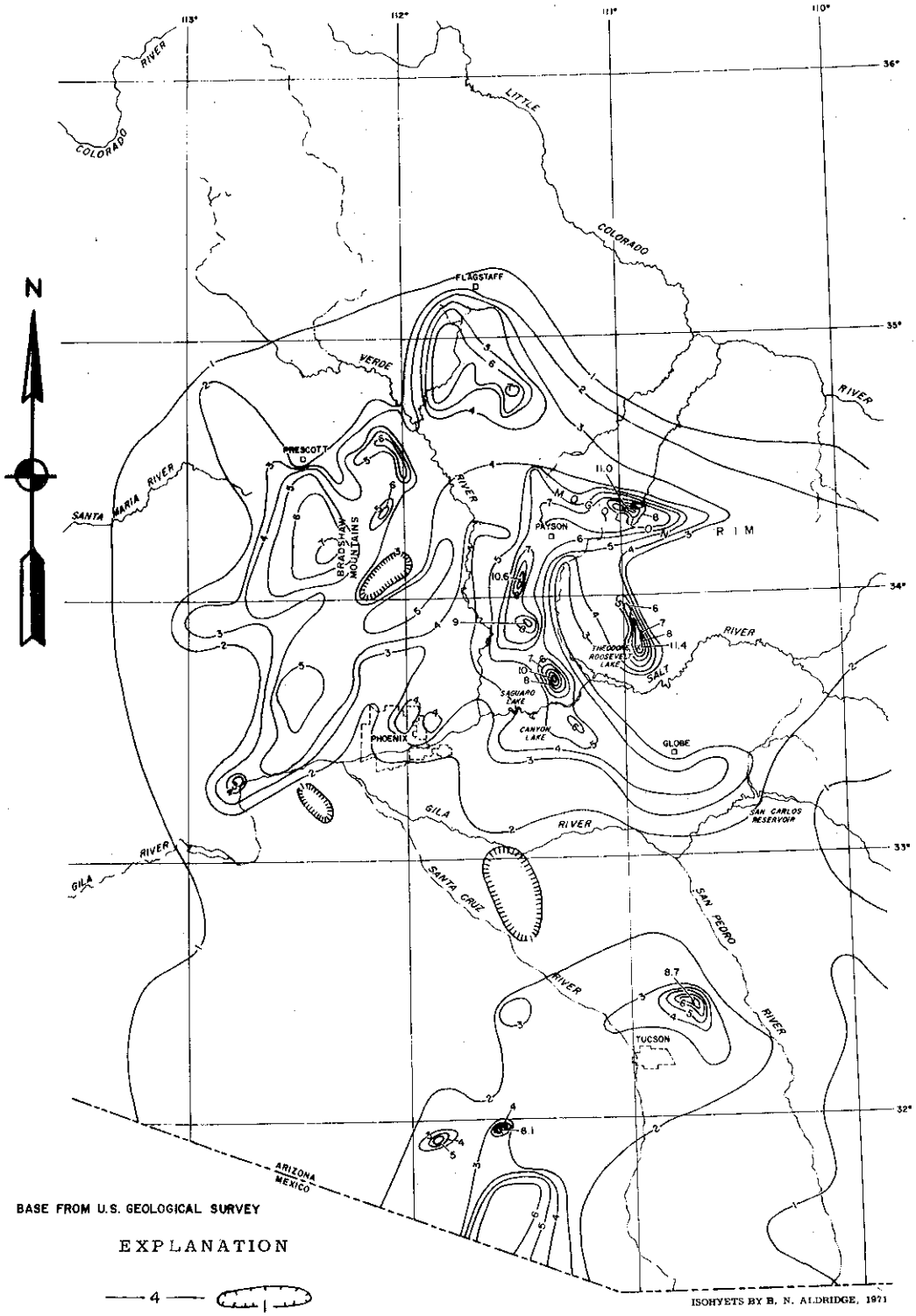

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
ISOHYETAL MAP
STORM OF AUGUST 19, 1954
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2
LEGEND
2 Line of equal precipitation in inches
Drainage boundaries.
4.5 River miles above mouth.
Effective drainage Area.
Existing dam.
Flood control dam site considered.
SCALE 0 5 10 MILES
N
VICINITY MAP
AREA COVERED BY REPORT
PHOENIX
Tempe
Mesa
Gilbert
Chandler
Apache Jctn
Florence Jctn
Hayden
Superior
Kelvin
Florence
SALT RIVER
GILA RIVER
INDIAN RESERVATION
TONTO NATIONAL FOREST
PINAL MOUNTAINS
SUPERSTITION MOUNTAINS
SANTAN MTN
MILITARY RIFLE RANGE
WHITLOW'S RANCH DAM SITE DRAINAGE AREA 144 SQ. MI.
BLACK POINT DAM SITE DRAINAGE AREA 189 SQ. MI.
MARICOPA CO
PINAL CO
GILA CO
To Tucson
To Casa Grande
To Globe

PLATE II



BASE FROM U.S. GEOLOGICAL SURVEY

ISOHYETS BY B. N. ALDRIDGE, 1971

EXPLANATION

—4— ISOHYETS

SHOW AMOUNT OF RAINFALL; INTERVAL, 1 INCH. HACHURES INDICATE LESS RAINFALL THAN VALUE SHOWN; SOME RAINFALL NEAR THE MEXICAN BOUNDARY OCCURRED BEFORE MIDNIGHT ON SEPTEMBER 3

x 11.4

SHOWS MAXIMUM RAINFALL WHERE ALL ISOHYETS CANNOT BE SHOWN

Scale 0 10 20 30 40 50 Miles

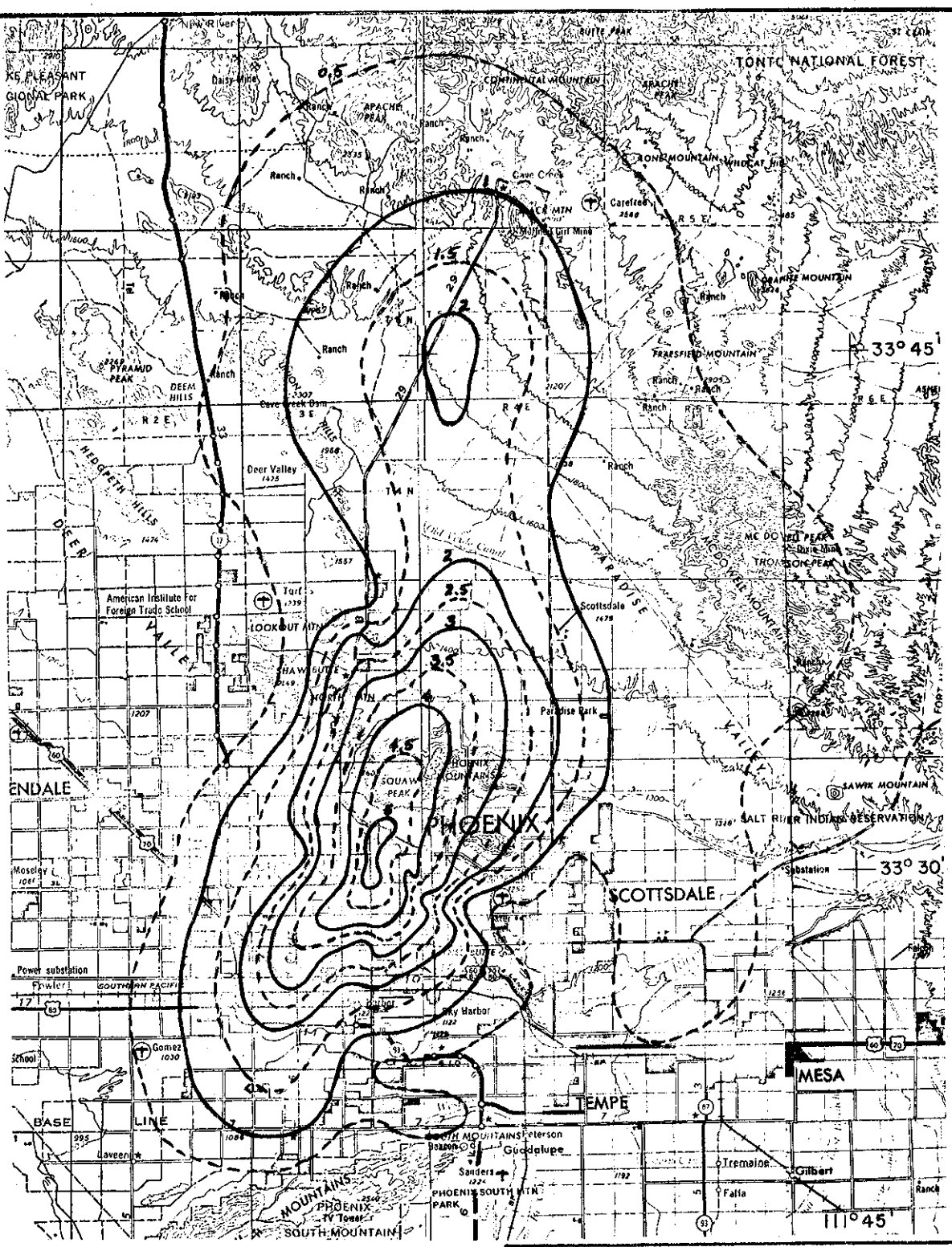
GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ISOHYETAL MAP

STORM OF SEPT. 4-6, 1970

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 12



N

SCALE IN MILES

1 0 4 8

LEGEND

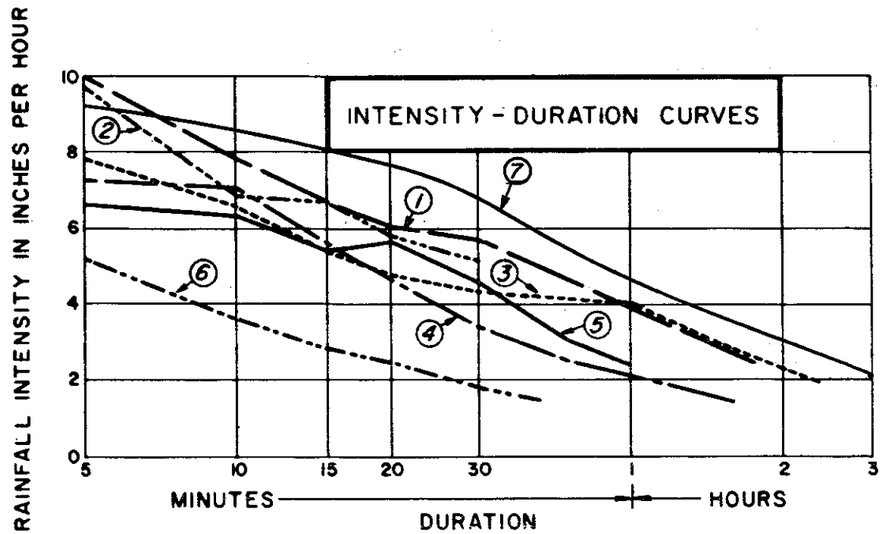
—2— Line of equal precipitation in inches

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

ISOHYETAL MAP
STORM OF JUNE 21-22, 1972

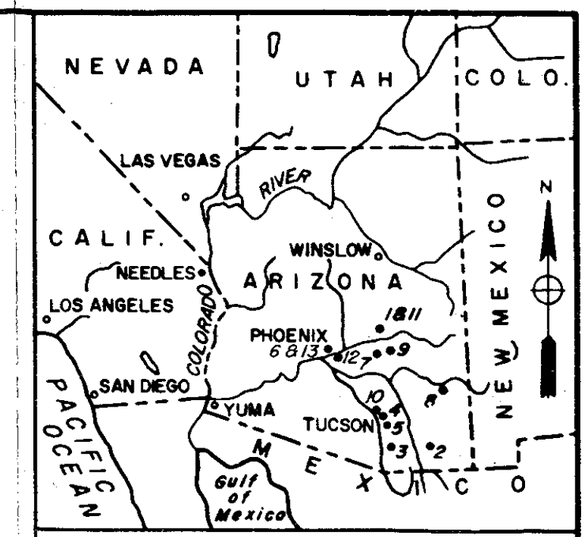
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 13



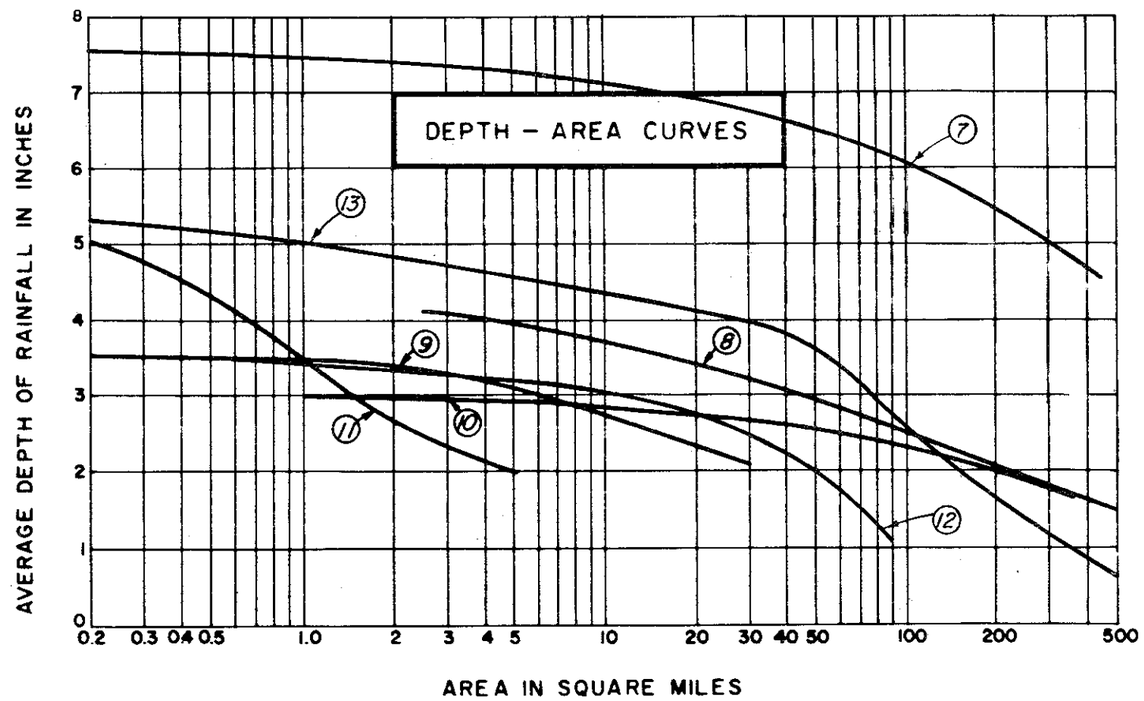
NOTE:

INTENSITY-DURATION CURVE NO. 7 REPRESENTS APPROXIMATELY THE VALUES AT THE STORM CENTER AND IS SYNTHESIZED FROM DATA AT VARIOUS GAGES WITHIN THE STORM, SUPPLEMENTED BY INTENSITY-DURATION VALUES FROM OTHER CENTRAL ARIZONA STORMS WITH VERY SHORT DURATIONS. DATA FOR OTHER INTENSITY-DURATION CURVES ARE FOR STATIONS WITHIN THE STORM AREA BUT NOT NECESSARILY AT THE STORM CENTER.



VICINITY MAP

SCALE 40 0 40 80 120 MILES

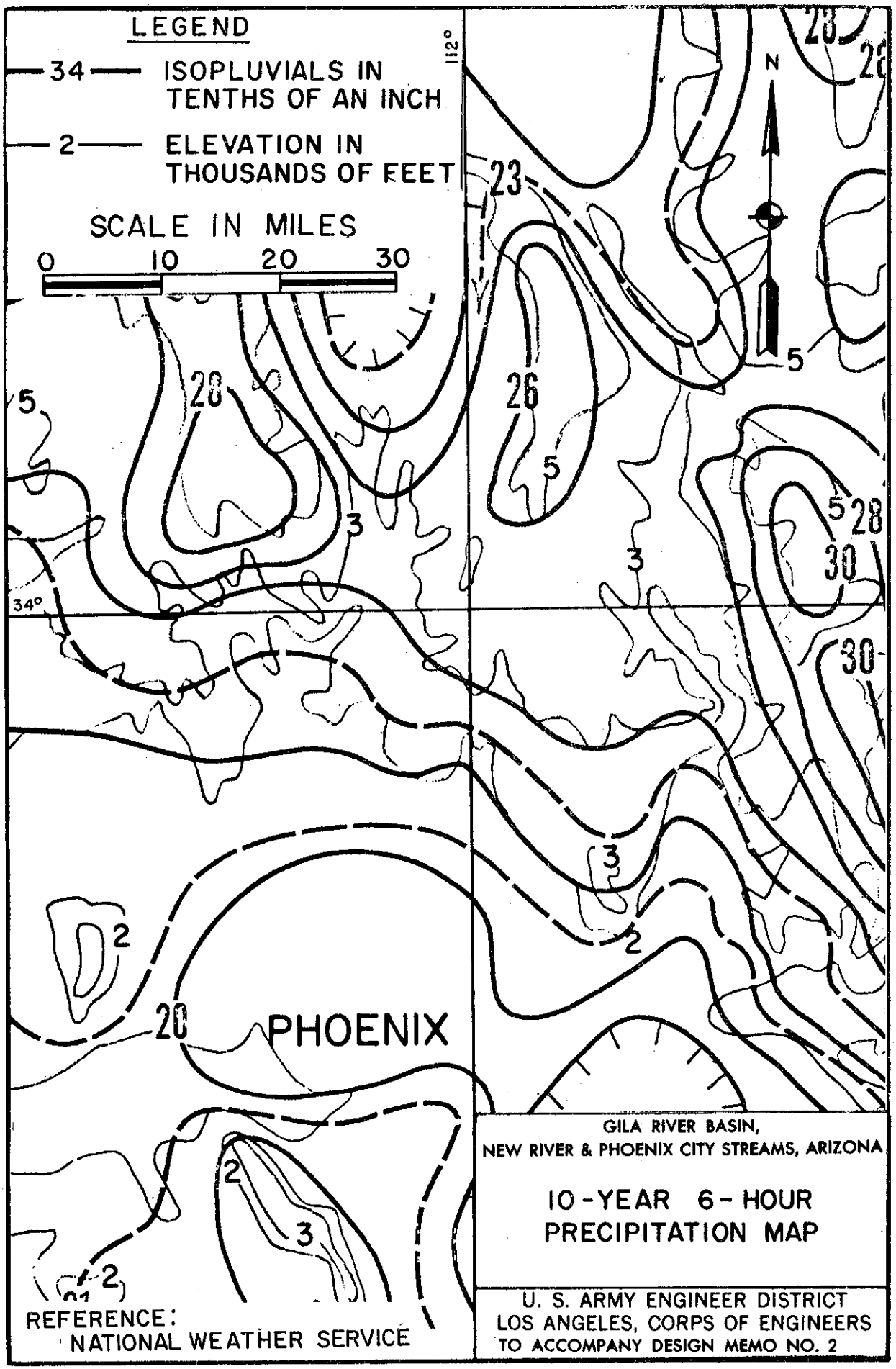


CURVE NO.	STORM			
	LOCATION	DATE	APPROXIMATE DURATION	
			HRS.	MIN.
1	PARKER CREEK	SEPT. 10, 1933	1	45
2	WALNUT GULCH	OCT. 4-5, 1954	0	30
3	SANTA RITA	JUNE 29, 1959	2	20
4	UNIV. OF ARIZONA	AUG. 13, 1940	1	35
5	TUCSON AIRPORT	SEPT. 24, 1943	1	0
6	PHOENIX	JULY 26, 1936	0	40
7	QUEEN CREEK	AUG. 19, 1954	7	0
8	THATCHER	SEPT. 16, 1939	1	30
9	GLOBE	JULY 29, 1954	1	0
10	TUCSON	SEPT. 24, 1943	3	0
11	PARKER CREEK	AUG. 5, 1939	2	20
12	TEMPE	SEPT. 14, 1969	1	0
13	PHOENIX	JUNE 22, 1972	2	0

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

INTENSITY - DURATION AND DEPTH - AREA CURVES

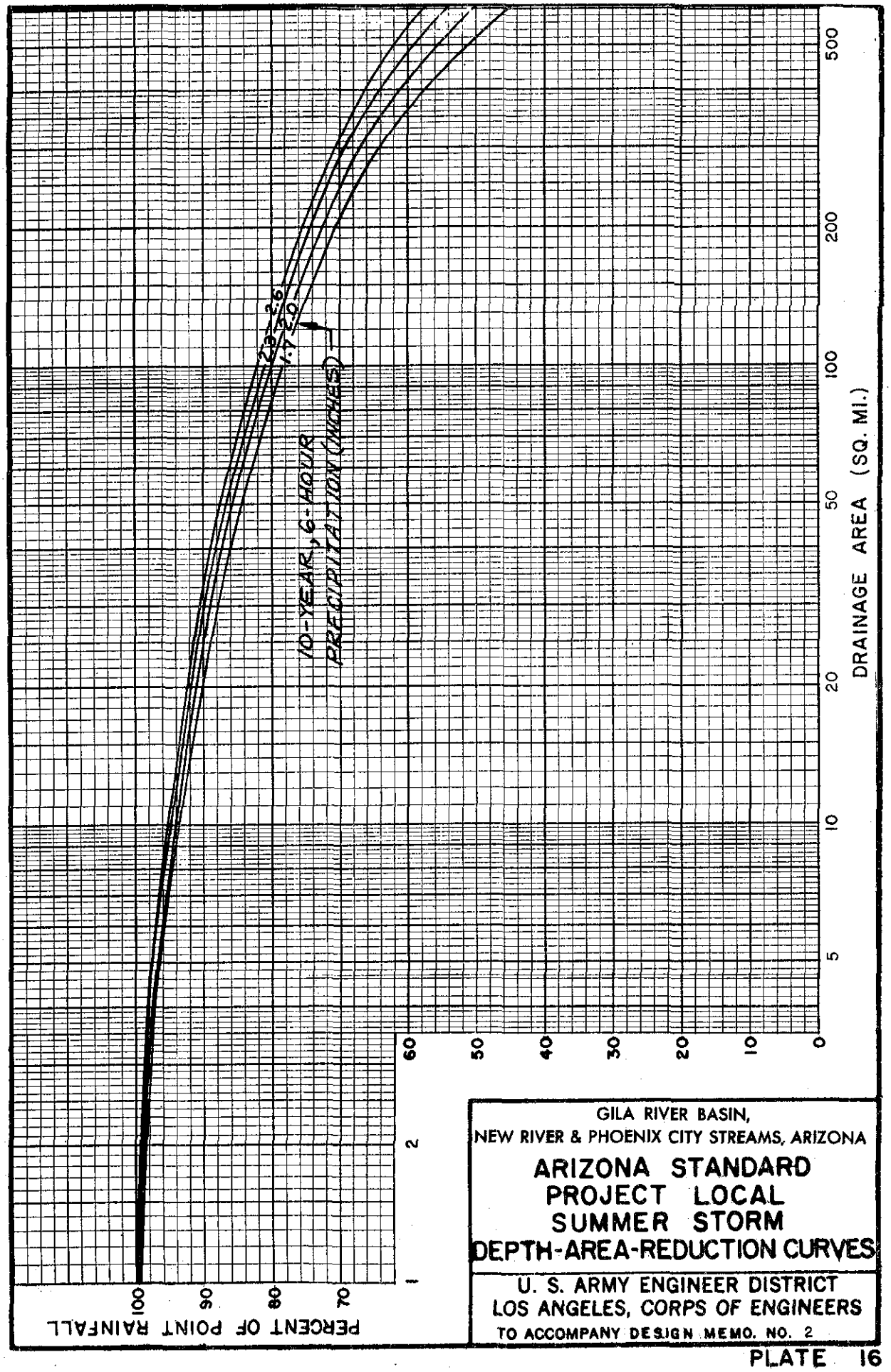
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

10-YEAR 6-HOUR
PRECIPITATION MAP

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

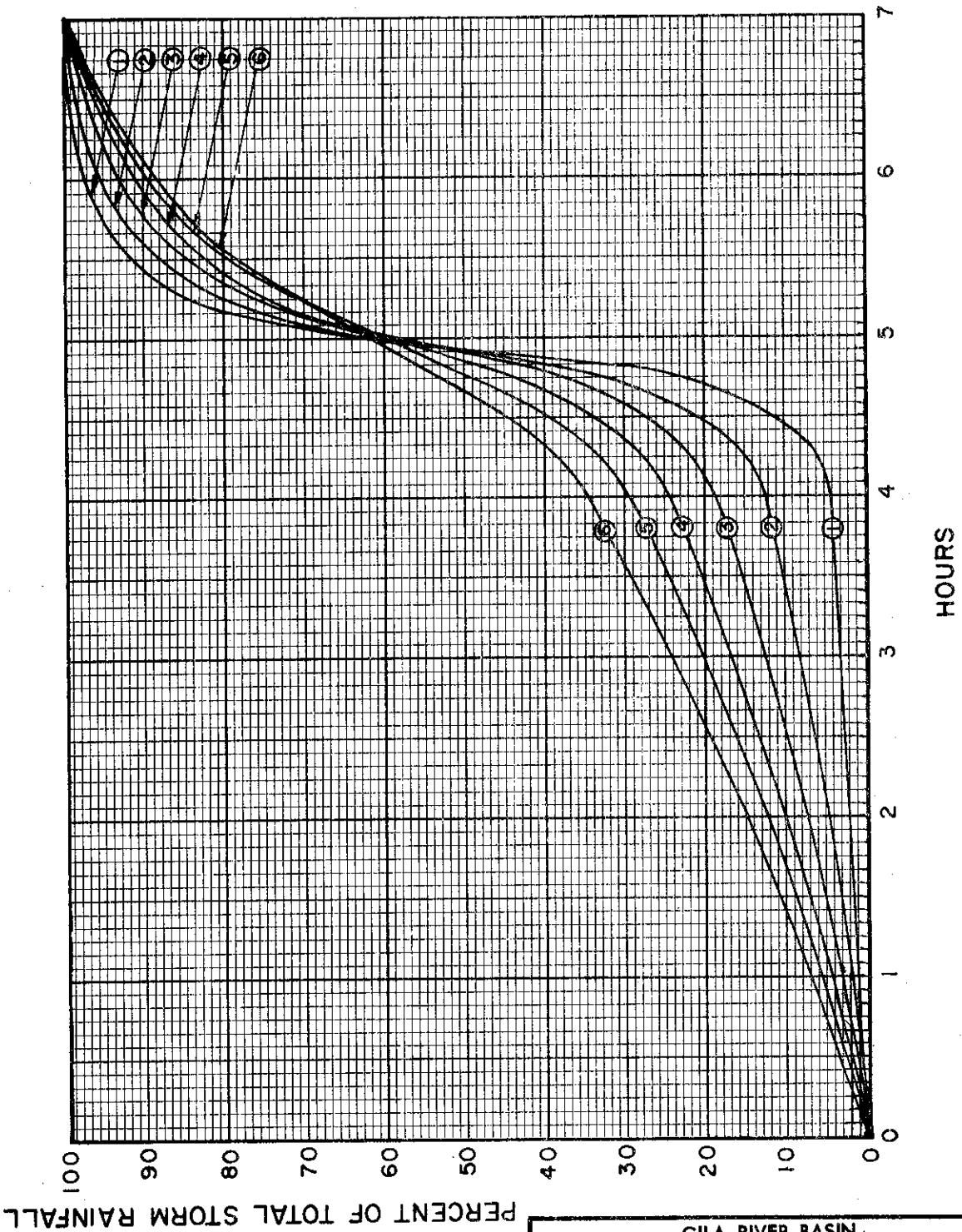


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

ARIZONA STANDARD PROJECT LOCAL SUMMER STORM DEPTH-AREA-REDUCTION CURVES

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO. NO. 2

PLATE 16

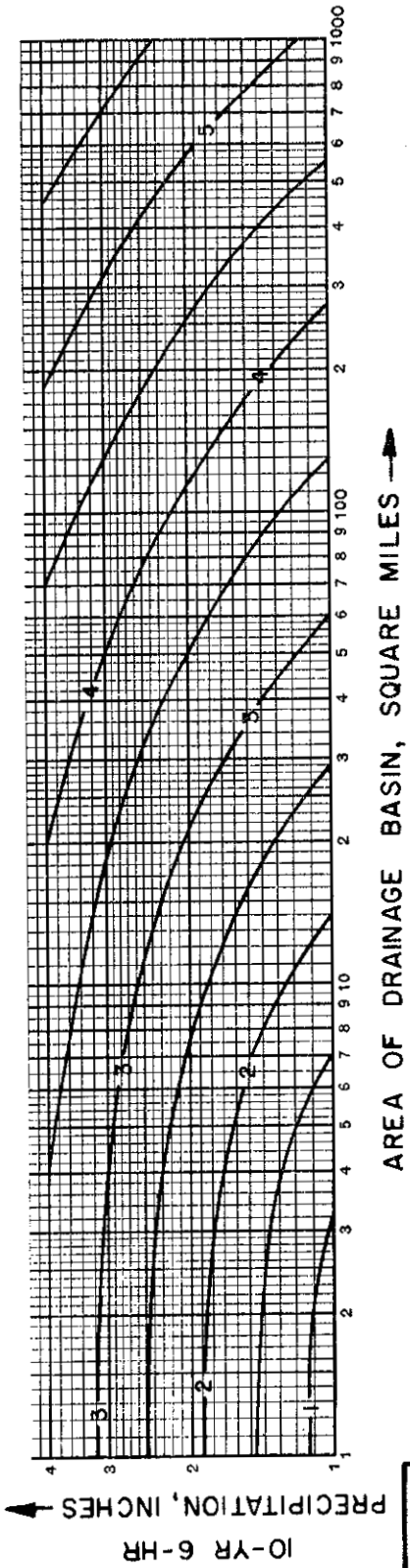


MAKE PATTERN NUMBER SELECTION ON PLATE 18.

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

ARIZONA STANDARD PROJECT LOCAL SUMMER STORM PRECIPITATION PATTERNS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

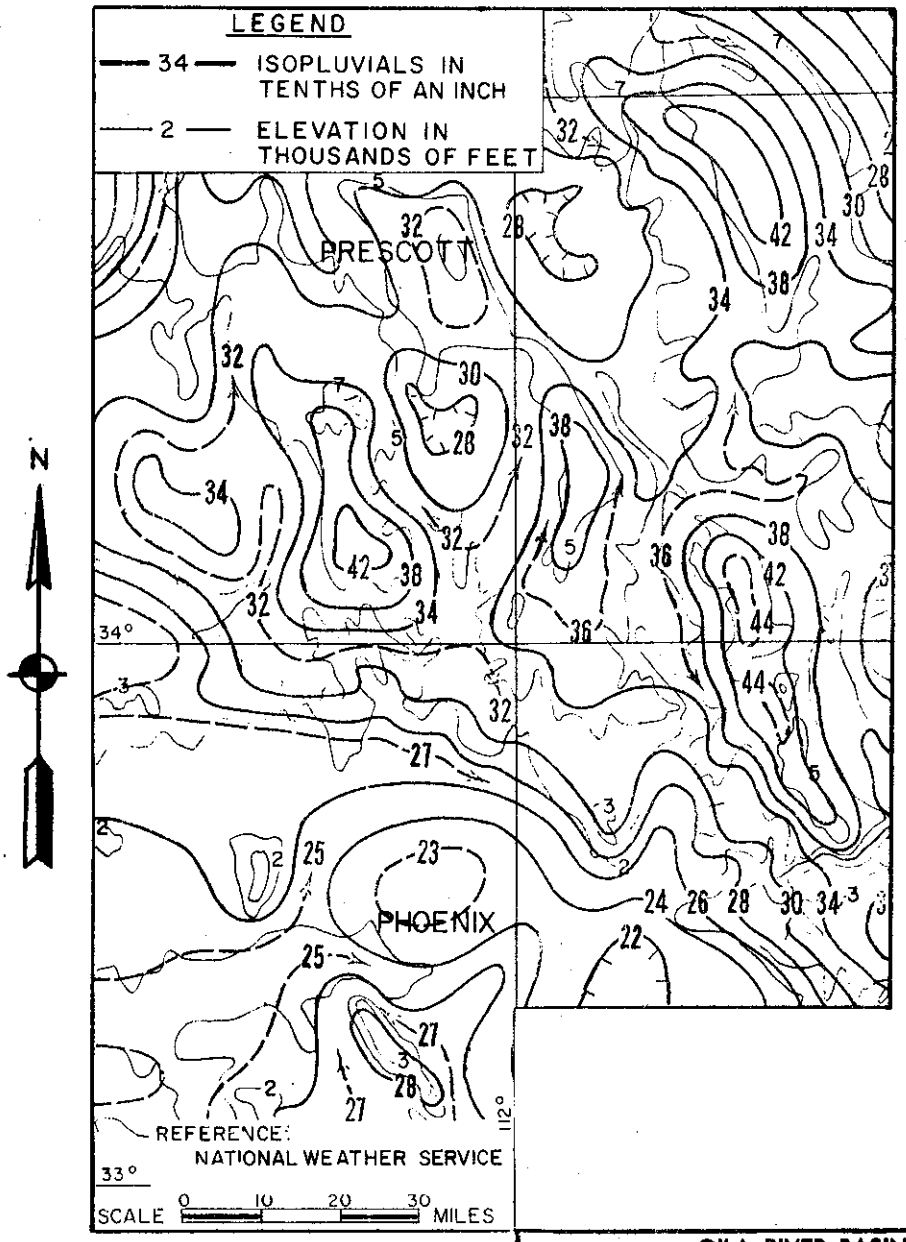


REFER TO PLATE 17
FOR ACTUAL PATTERN.

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

ARIZONA STANDARD PROJECT
LOCAL SUMMER STORM
PRECIPITATION-AREA-PATTERN
CURVES

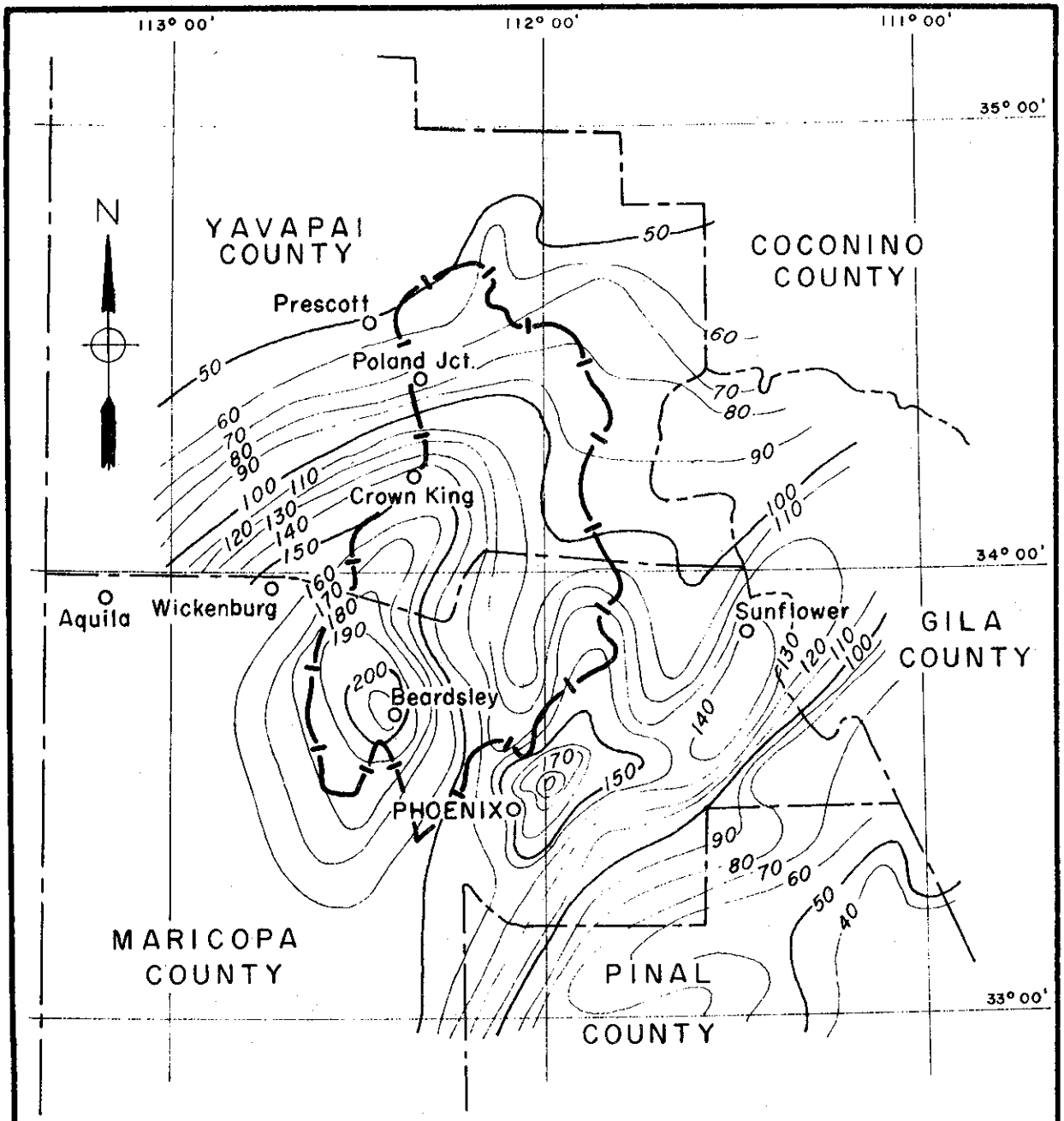
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY **DESIGN MEMO NO. 2**



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

10-YEAR 24-HOUR PRECIPITATION MAP

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



LEGEND

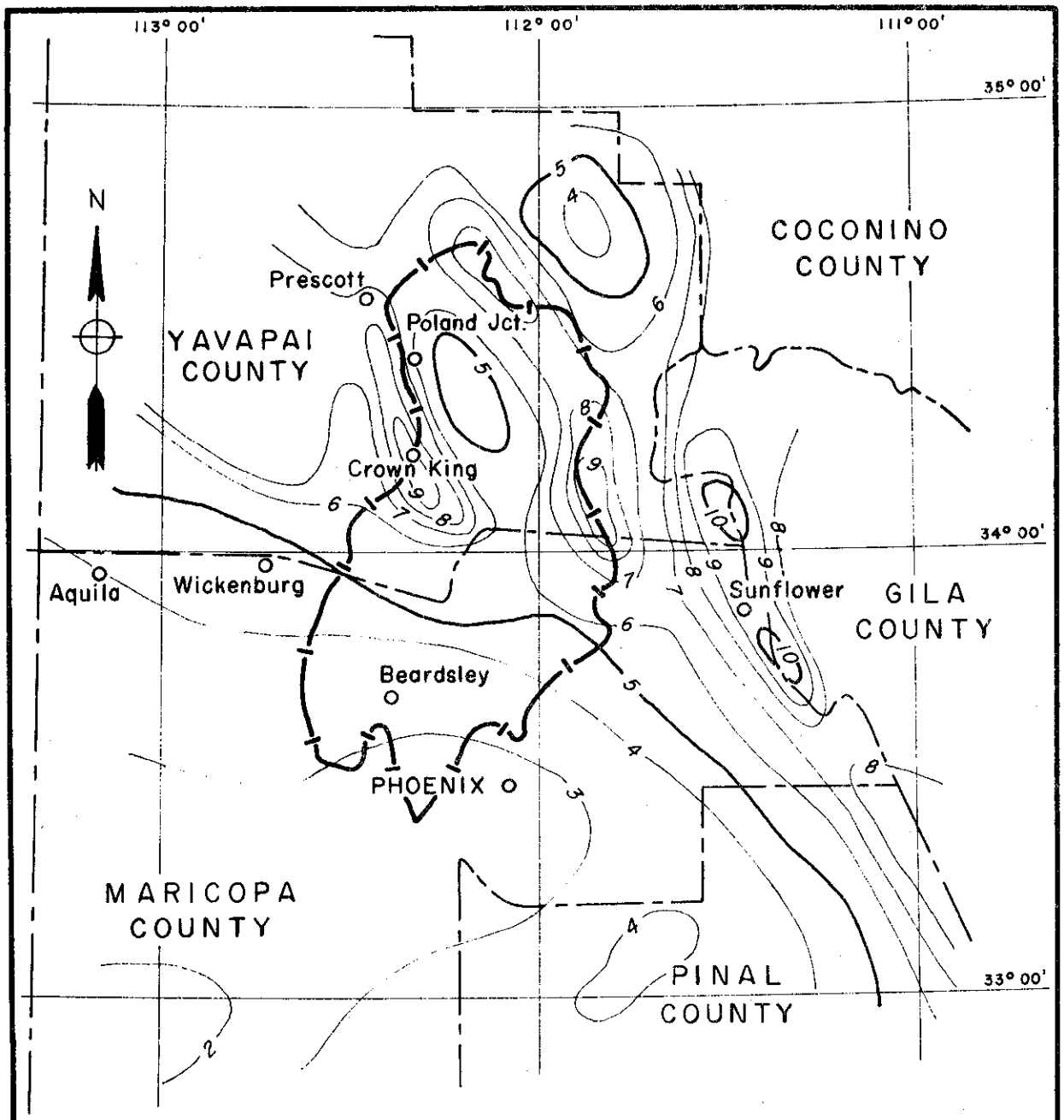
SCALE 10 0 10 20 30 40 MILES

—|— BOUNDARY OF DRAINAGE AREA

—140— LINE OF EQUAL PERCENTAGE

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
STORM OF AUGUST 26-29, 1951
ISOPERCENTUAL MAP
TOTAL STORM PRECIPITATION IN
PERCENT OF MEAN SUMMER SEASONAL
PRECIPITATION

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



LEGEND

—|— BOUNDARY OF DRAINAGE AREA

—140— LINE OF EQUAL PRECIPITATION IN INCHES

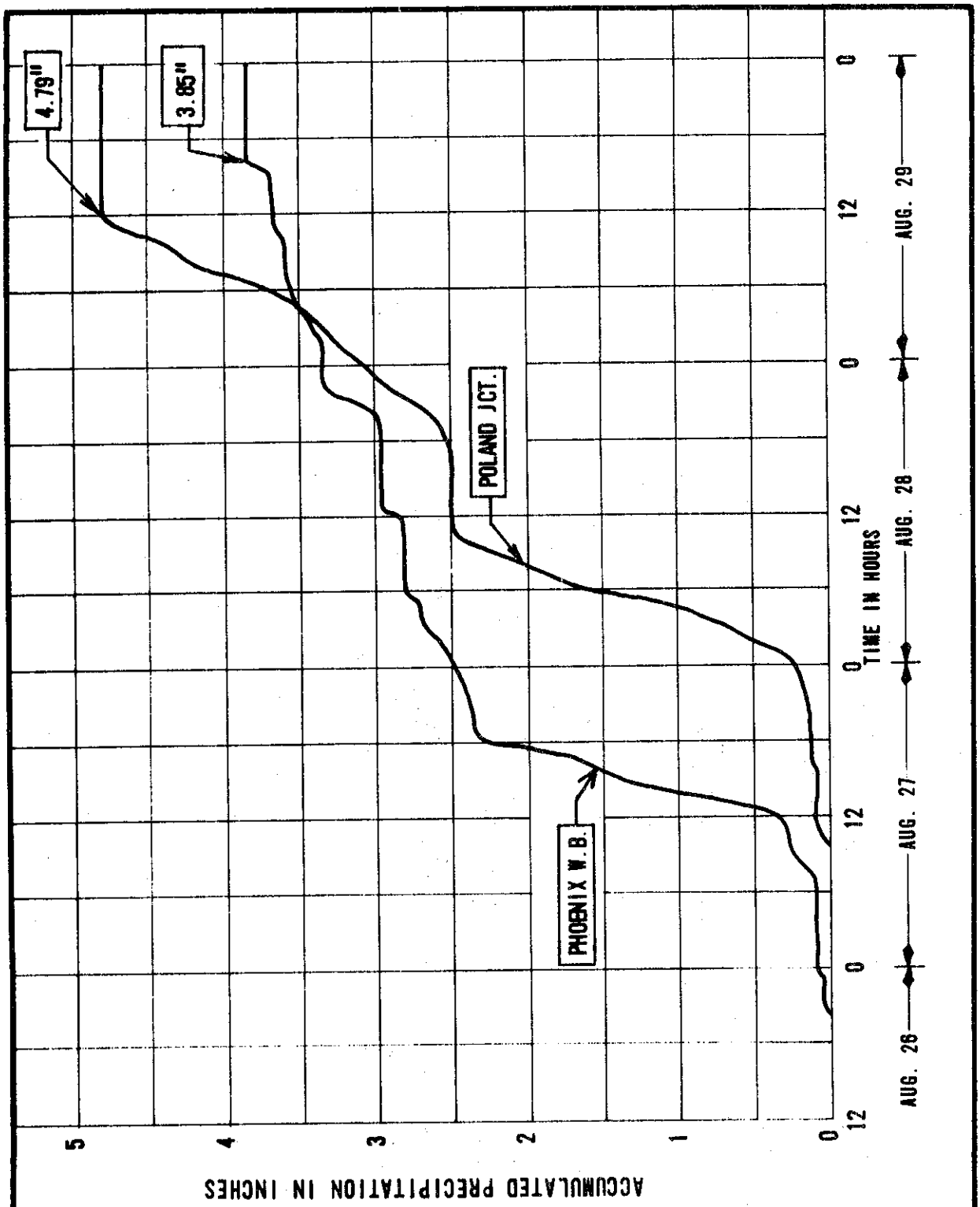
SCALE 10 0 10 20 30 40 MILES

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

ISOHYETS

MEAN SUMMER SEASONAL PRECIPITATION

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

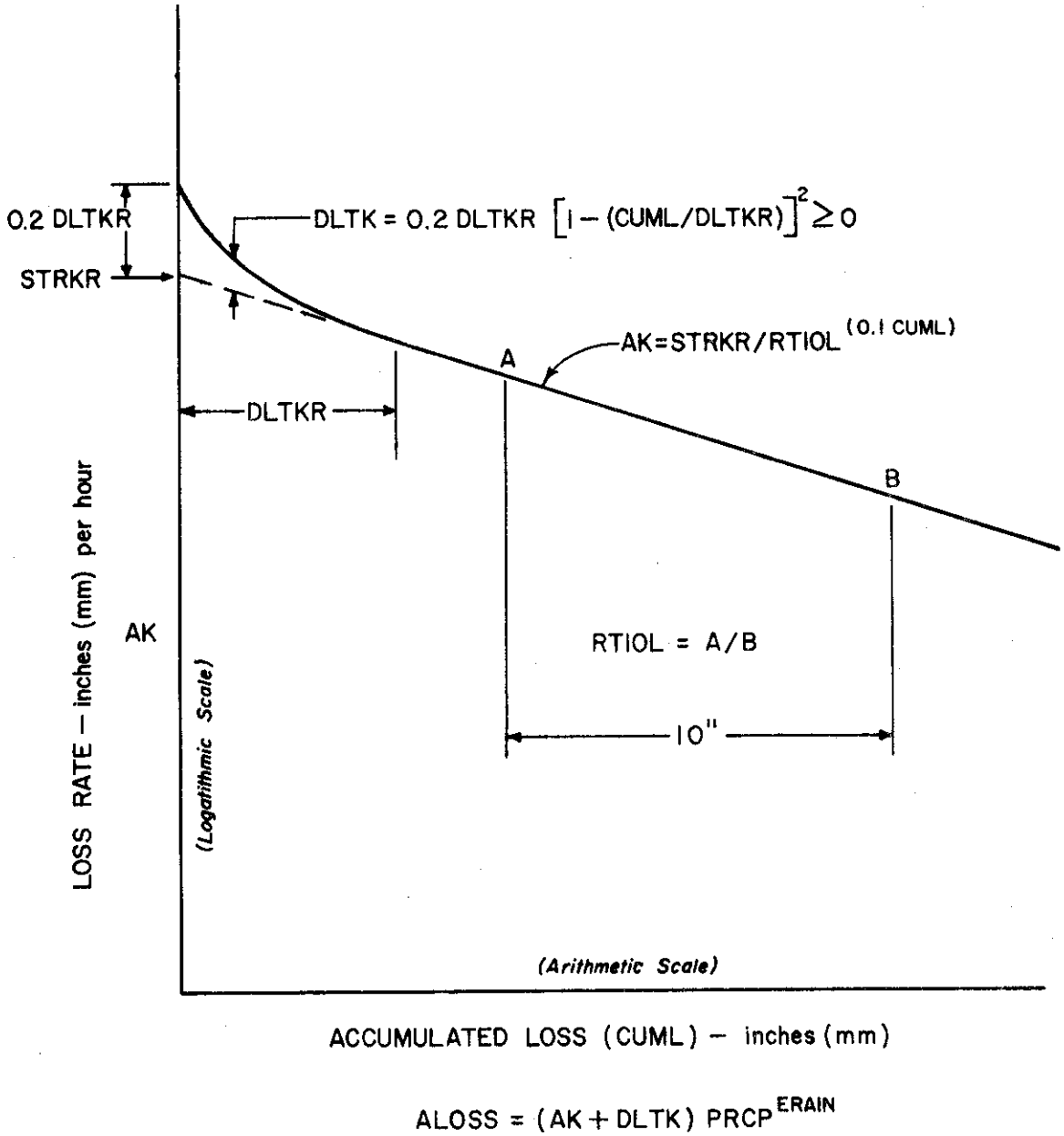


REFER TO PLATE 10
FOR STATION LOCATIONS.

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
STORM OF AUGUST 26-29, 1951

MASS CURVES OF PRECIPITATION

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



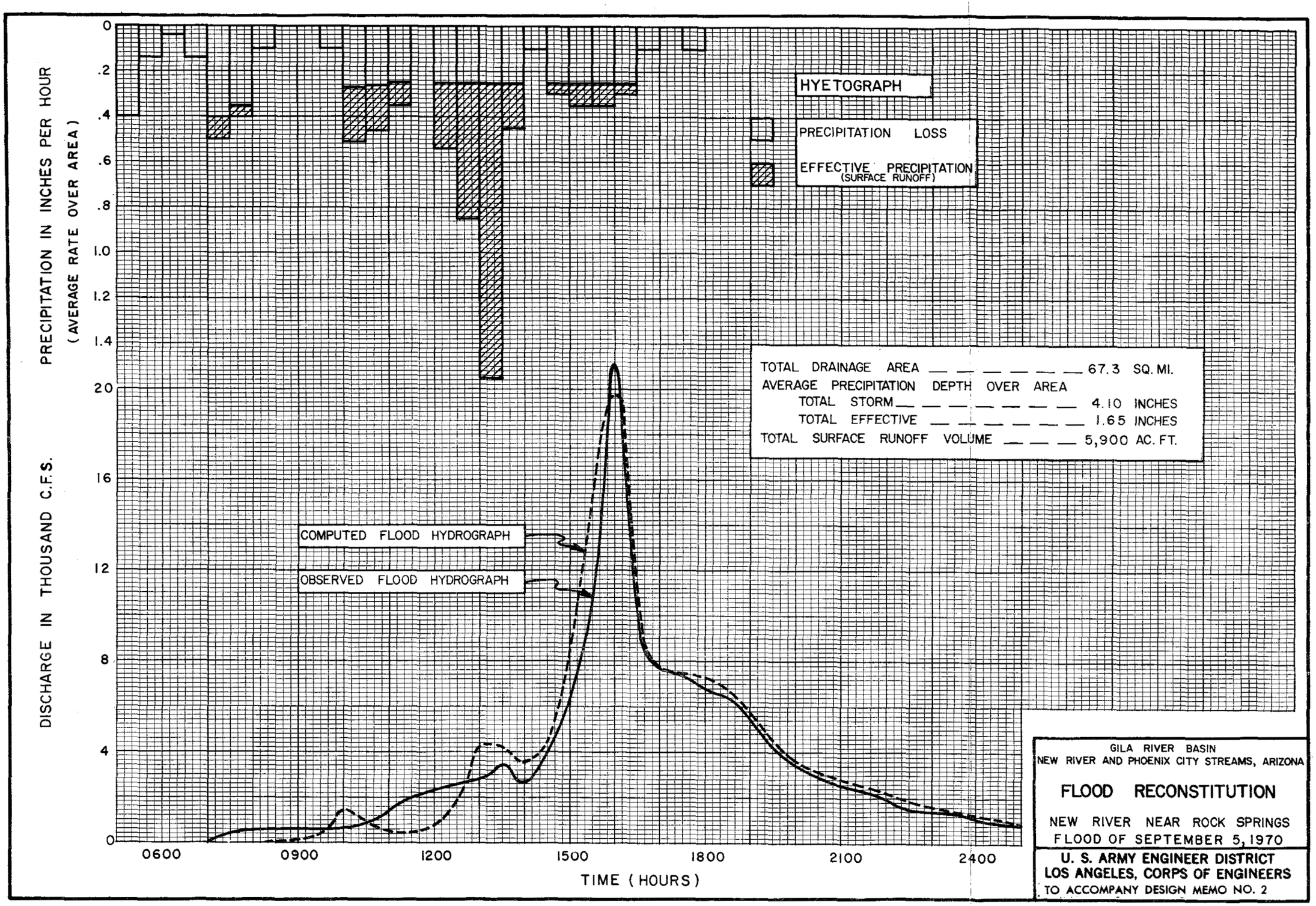
$$ALOSS = (AK + DLTK)\ PRCP^{ERAIN}$$

NOTE: DEFINITIONS OF VARIABLES GIVEN IN PARA. 4-04 OF TEXT.

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

H.E.C. LOSS RATE FUNCTION

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

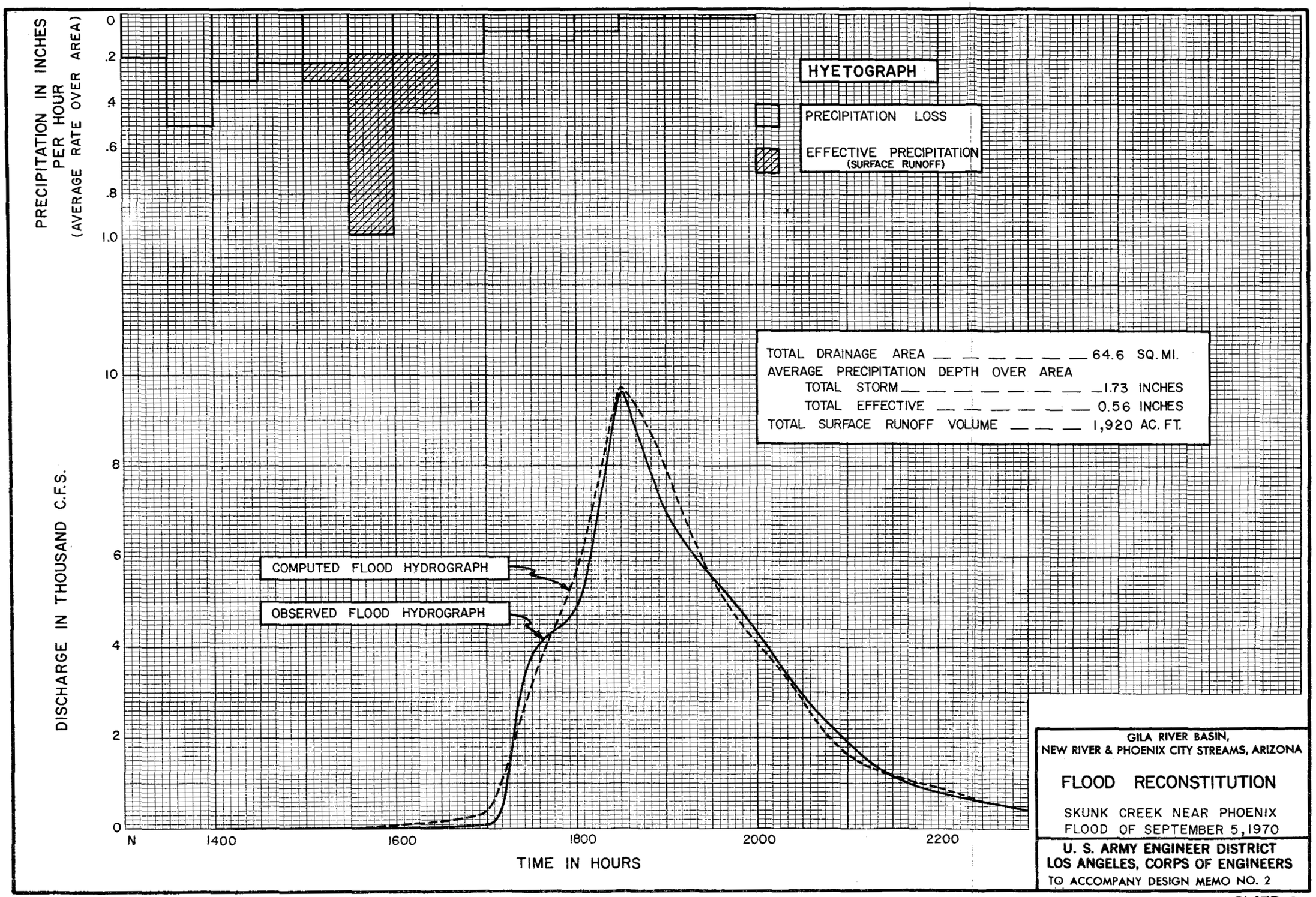


GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

FLOOD RECONSTITUTION

NEW RIVER NEAR ROCK SPRINGS
FLOOD OF SEPTEMBER 5, 1970

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

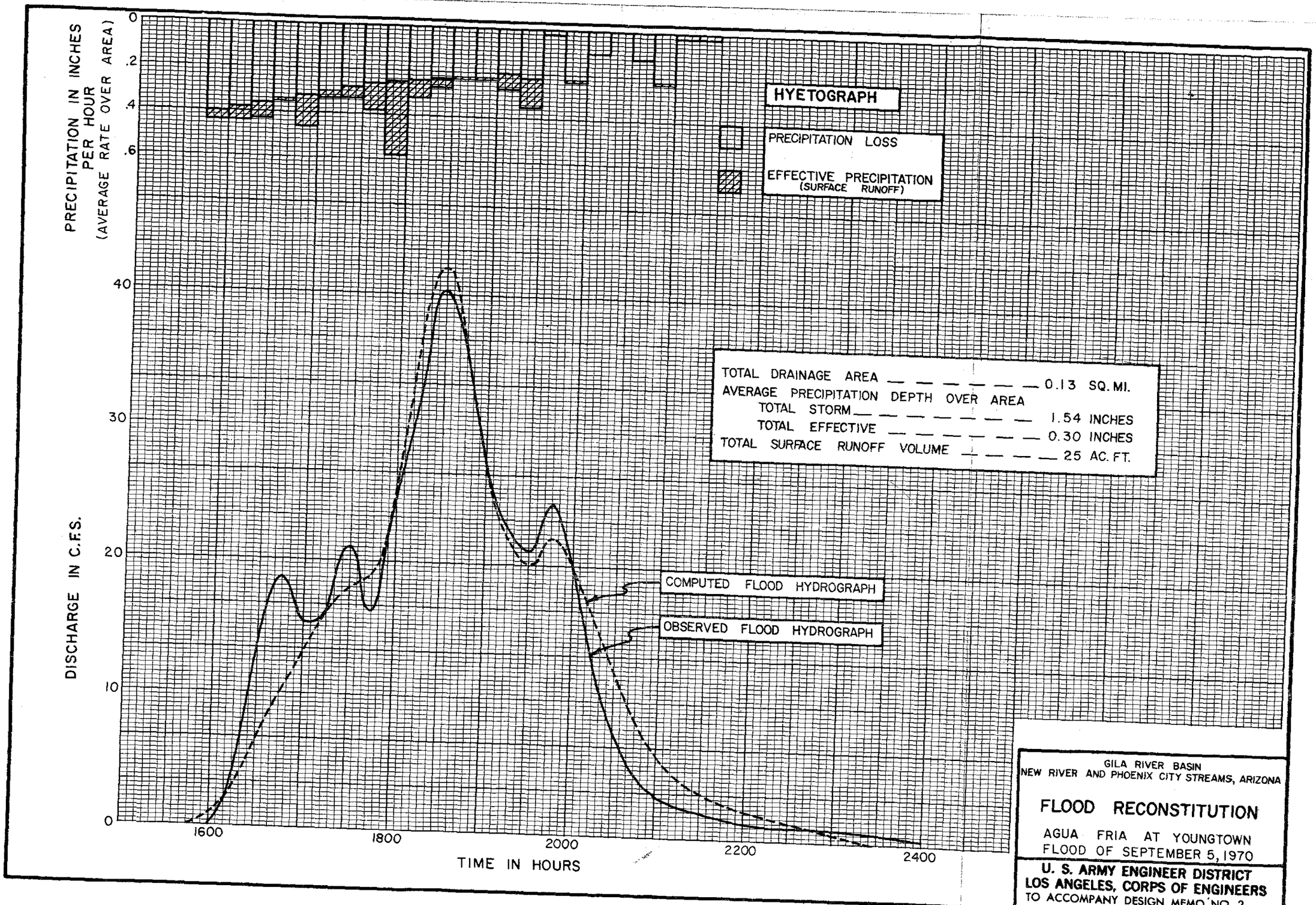


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

FLOOD RECONSTITUTION

SKUNK CREEK NEAR PHOENIX
FLOOD OF SEPTEMBER 5, 1970

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



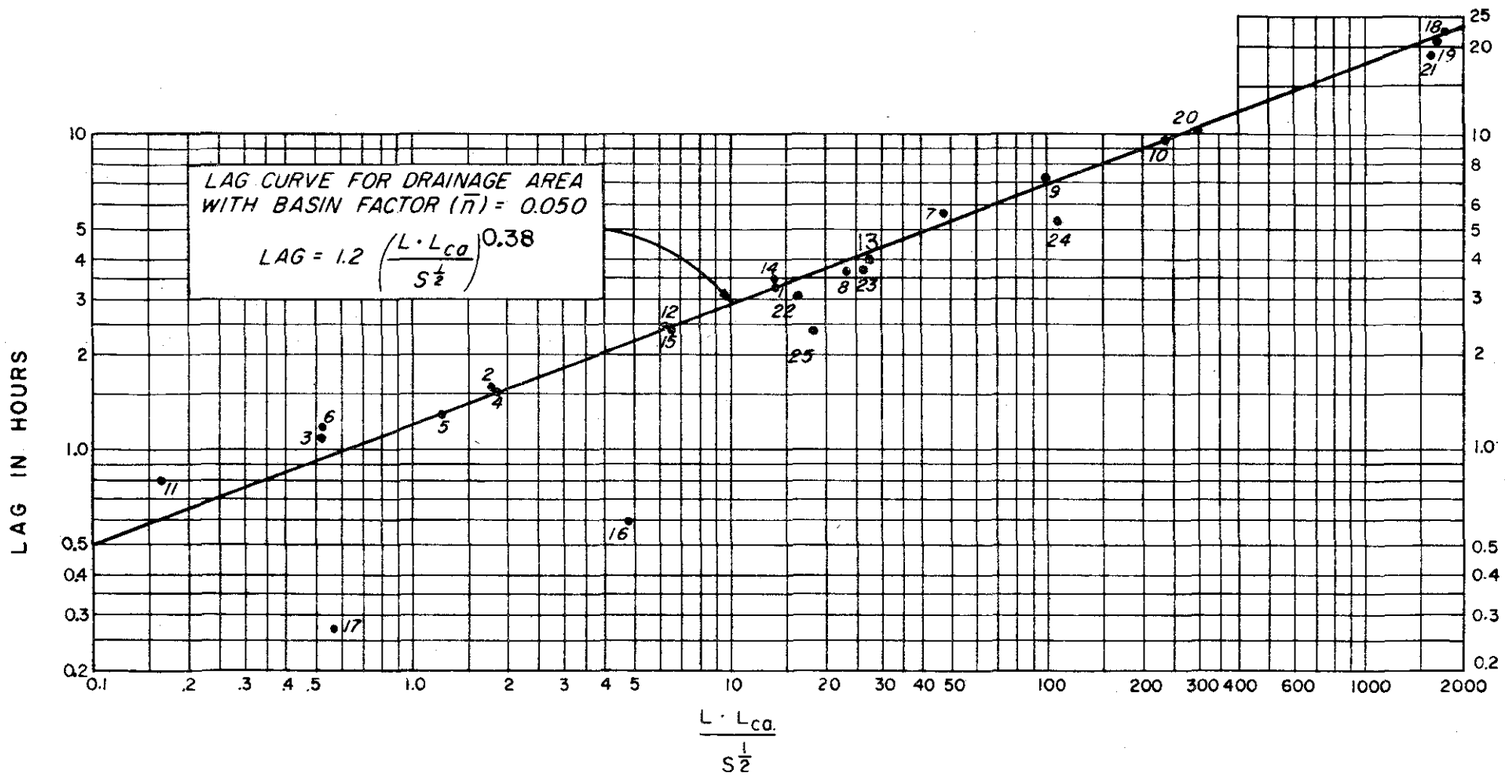
GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

FLOOD RECONSTITUTION

AGUA FRIA AT YOUNGTOWN
FLOOD OF SEPTEMBER 5, 1970

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

		CONTRIBUTING AREA SQ. MI.	L MILES	L_{ca} MILES	S FT./MI.	LAG HOURS	ESTIMATED $\bar{n}$
1.	SAN GABRIEL RIVER AT SAN GABRIEL DAM, CALIF.	162.0	23.2	11.6	350	3.3	0.050
2.	WEST FORT SAN GABRIEL RIVER AT COGSWELL DAM, CALIF.	40.4	9.3	4.3	450	1.6	.050
3.	SAN ANITA CREEK AT SANTA ANITA DAM, CALIF.	10.8	5.8	2.5	690	1.1	.050
4.	SAN DIMAS CREEK AT SAN DIMAS DAM, CALIF.	16.2	8.6	4.8	440	1.5	.050
5.	EATON WASH AT EATON WASH DAM, CALIF.	9.5	7.3	4.4	600	1.3	.050
6.	SAN ANTONIO CREEK NEAR CLAREMONT, CALIF.	16.9	5.9	3.0	1,017	1.2	.055
7.	SANTA CLARA RIVER NEAR SAUGUS, CALIF	355.0	36.0	15.8	140	5.6	.050
8.	TEMECULA CREEK AT PAUBA CANYON, CALIF.	168.0	26.0	11.3	150	3.7	.050
9.	SANTA MARGARITA RIVER NEAR FALLBROOK, CALIF.	645.0	46.0	22.0	105	7.3	.055
10.	SANTA MARGARITA RIVER AT YSIDORA, CALIF.	740.0	61.2	34.3	85	9.5	.055
11.	LIVE OAK CREEK AT LIVE OAK DAM, CALIF.	2.3	2.9	1.5	700	0.8	.070
12.	TUJUNGA CREEK AT BIG TUJUNGA DAM, CALIF.	81.4	15.1	7.3	290	2.5	.050
13.	MURRIETA CREEK AT TEMECULA, CALIF.	220.0	27.2	10.3	95	4.0	.050
14.	LOS ANGELES RIVER AT SEPULVEDA DAM, CALIF.	152.0	19.0	9.0	145	3.5	.050
15.	PACOIMA WASH AT PACOIMA DAM, CALIF.	27.8	15.0	8.0	315	2.4	.050
16.	ALHAMBRA WASH ABOVE SHORT STREET, CALIF.	14.0	9.5	4.6	85	0.6	.015
17.	BROADWAY DRAIN ABOVE RAYMOND DIKE, CALIF.	2.5	3.4	1.7	100	0.28	.015
18.	GILA RIVER AT CONNOR NO. 4 DAM SITE, ARIZ.	2840.0	131.0	71.0	29	21.5	.050
19.	SAN FRANCISCO RIVER AT JUNCTION WITH BLUE RIVER, ARIZ.	2000.0	130.0	74.0	32	20.6	.050
20.	BLUE RIVER NEAR CLIFTON, ARIZ.	790.0	77.0	37.0	65	10.3	.050
21.	SALT RIVER NEAR ROOSEVELT, ARIZ.	4310.0	160.0	66.0	45	18.6	.050
22.	NEW RIVER AT ROCK SPRINGS, ARIZ.	67.3	20.2	9.7	141	3.1	.045
23.	NEW RIVER AT NEW RIVER, ARIZ.	85.7	23.2	13.6	145	3.7	.045
24.	NEW RIVER AT BELL ROAD, ARIZ.	187.0	47.6	20.7	83	5.3	.037
25.	SKUNK CREEK NEAR PHOENIX, ARIZ.	64.6	17.6	10.0	89	2.4	.033



GUIDE FOR ESTIMATING BASIN FACTOR ($\bar{n}$)

$\bar{n}$=0.200: DRAINAGE AREA HAS COMPARATIVELY UNIFORM SLOPES AND SURFACE CHARACTERISTICS SUCH THAT CHANNELIZATION DOES NOT OCCUR. GROUND COVER CONSISTS OF CULTIVATED CROPS OR SUBSTANTIAL GROWTHS OF GRASS AND FAIRLY DENSE SMALL SHRUBS, CACTI, OR SIMILAR VEGETATION. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$=0.050: DRAINAGE AREA IS QUITE RUGGED, WITH SHARP RIDGES AND NARROW, STEEP CANYONS THROUGH WHICH WATERCOURSES MEANDER AROUND SHARP BENDS, OVER LARGE BOULDERS, AND CONSIDERABLE DEBRIS OBSTRUCTION. THE GROUND COVER, EXCLUDING SMALL AREAS OF ROCK OUTCROPS, INCLUDES MANY TREES AND CONSIDERABLE UNDERBRUSH. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$=0.030: DRAINAGE AREA IS GENERALLY ROLLING, WITH ROUNDED RIDGES AND MODERATE SIDE SLOPES. WATERCOURSES MEANDER IN FAIRLY STRAIGHT, UNIMPROVED CHANNELS WITH SOME BOULDERS AND LODGED DEBRIS. GROUND COVER INCLUDES SCATTERED BRUSH AND GRASSES. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$=0.015: DRAINAGE AREA HAS FAIRLY UNIFORM, GENTLE SLOPES WITH MOST WATERCOURSES EITHER IMPROVED OR ALONG PAVED STREETS. GROUND COVER CONSISTS OF SOME GRASSES WITH APPRECIABLE AREAS DEVELOPED TO THE EXTENT THAT A LARGE PERCENTAGE OF THE AREA IS IMPERVIOUS.

TERMINOLOGY

L = LENGTH OF LONGEST WATERCOURSE.

L_{ca} = LENGTH ALONG LONGEST WATERCOURSE, MEASURED UPSTREAM TO POINT OPPOSITE CENTER OF AREA.

S = OVER-ALL SLOPE OF LONGEST WATERCOURSE BETWEEN HEADWATER AND COLLECTION POINT.

LAG = ELAPSED TIME FROM BEGINNING OF UNIT PRECIPITATION TO INSTANT THAT SUMMATION HYDROGRAPH REACHES 50% OF ULTIMATE DISCHARGE.

$\bar{n}$ = VISUALLY ESTIMATED MEAN OF THE n (MANNING'S FORMULA) VALUES OF ALL THE CHANNELS WITHIN AN AREA.

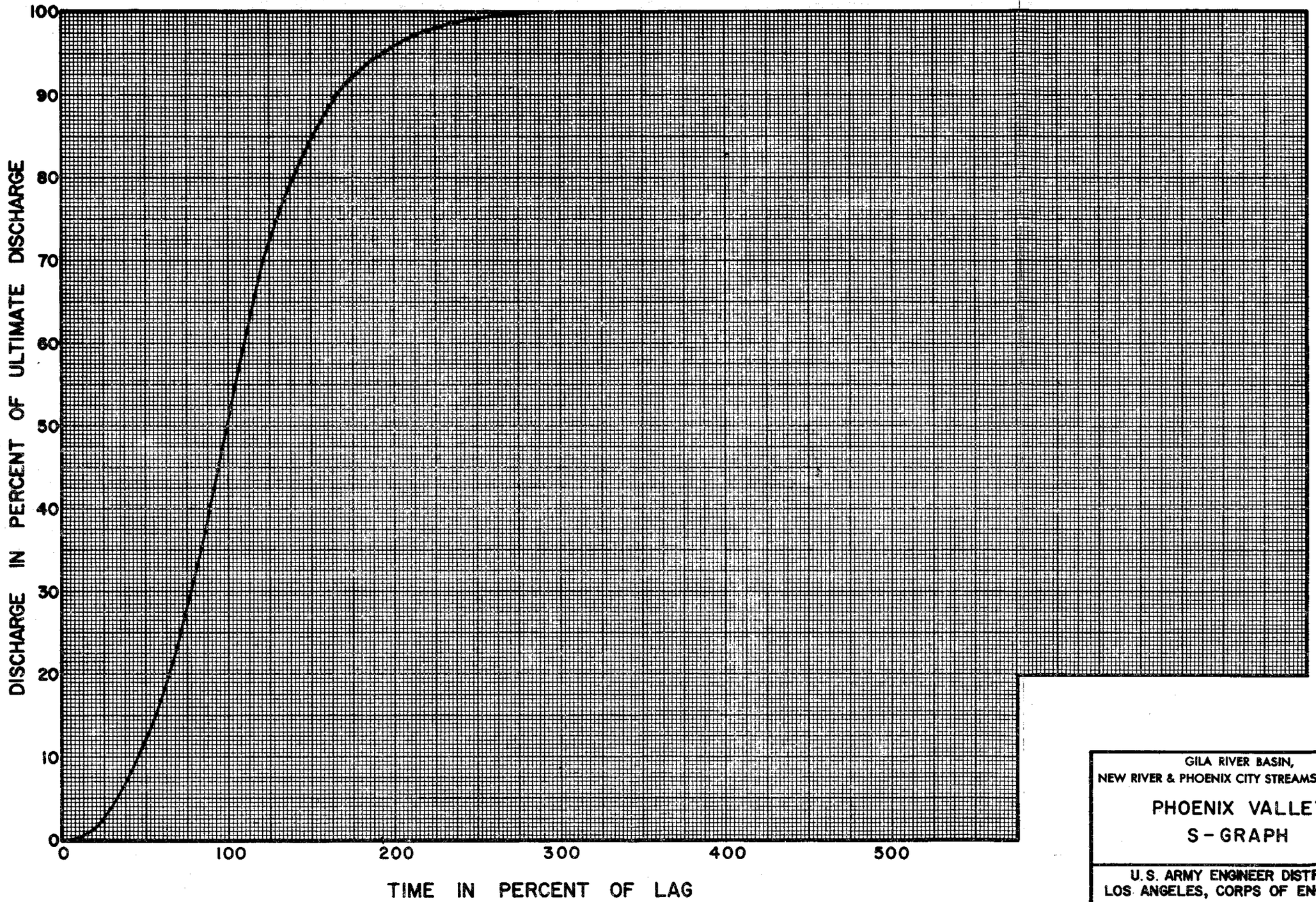
NOTE:
TO OBTAIN THE LAG (IN HOURS) FOR ANY AREA, MULTIPLY THE LAG OBTAINED FROM THE CURVE BY:

$$\frac{\bar{n}}{0.050} \text{ OR } 20\bar{n}$$

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

LAG RELATIONSHIPS

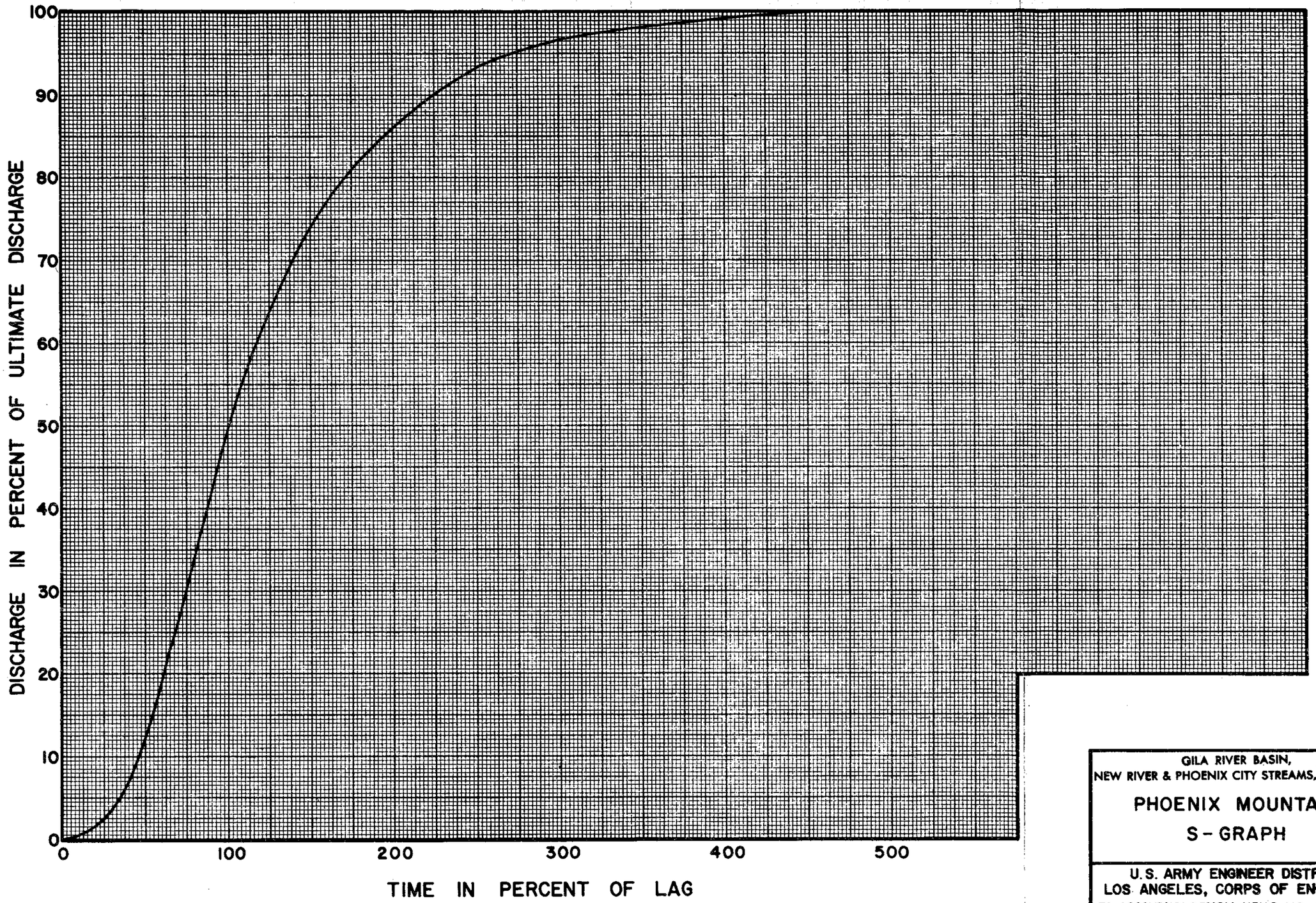
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

PHOENIX VALLEY
S-GRAPH

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

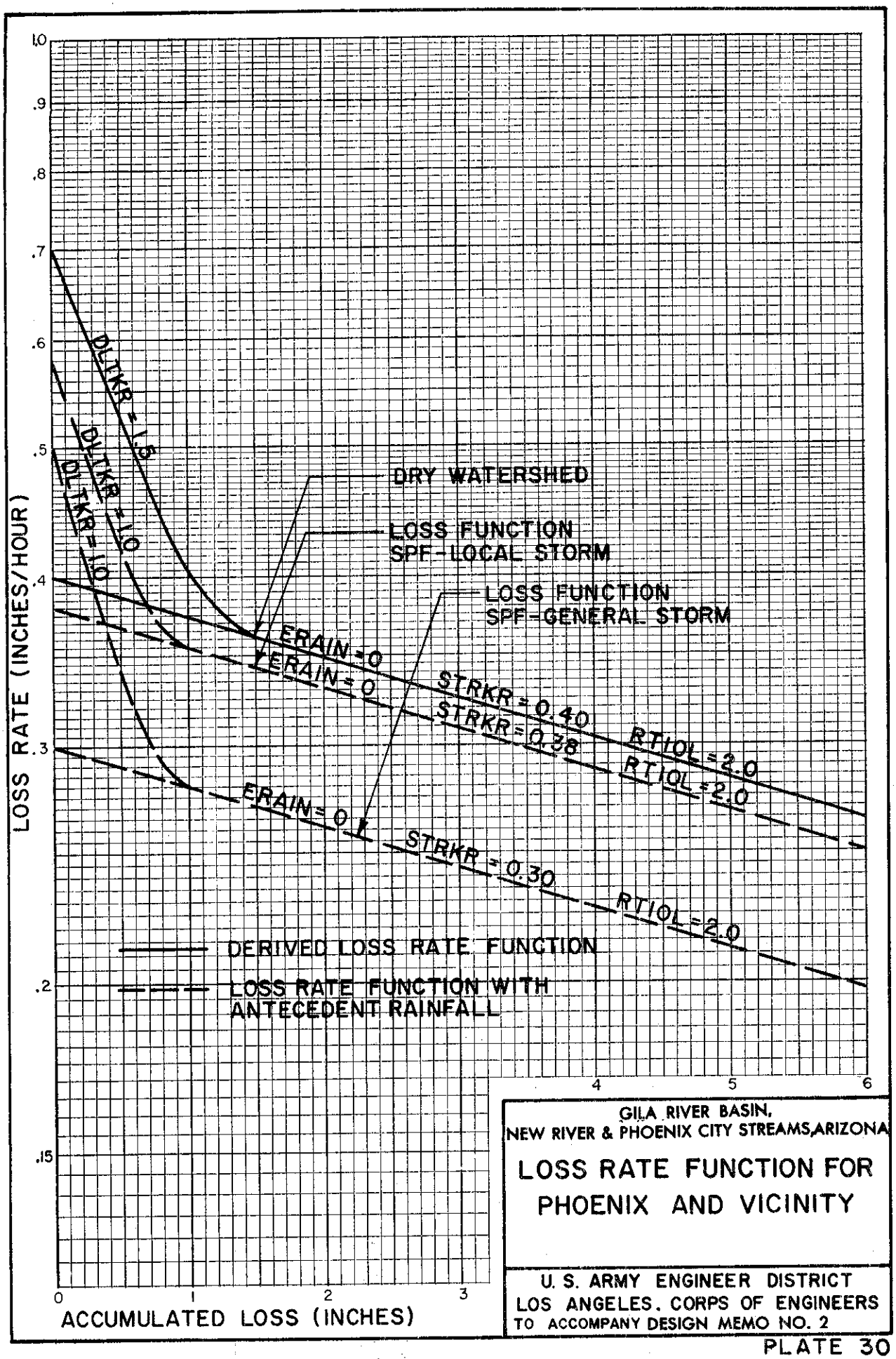


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

PHOENIX MOUNTAIN

S - GRAPH

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

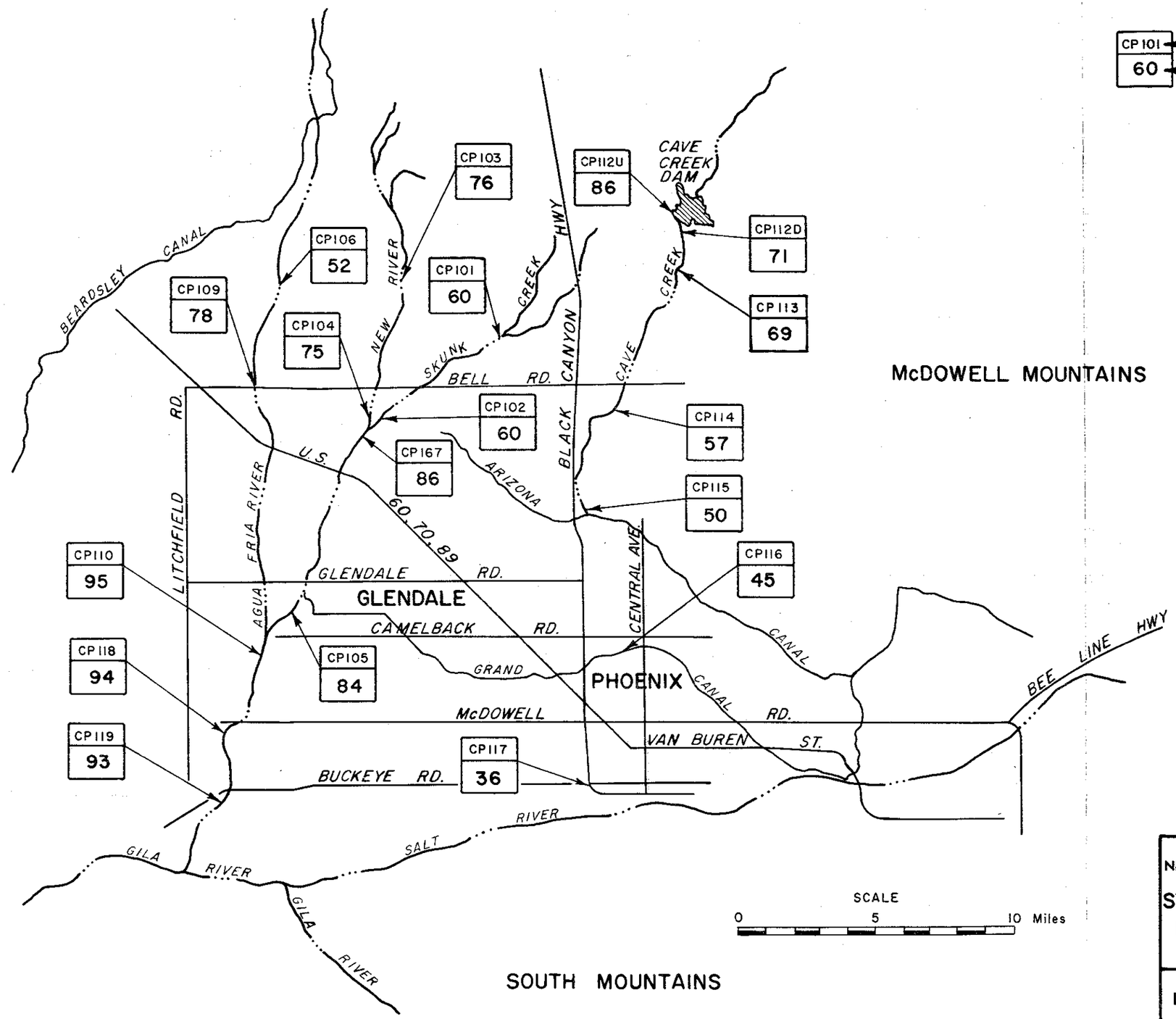


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

LOSS RATE FUNCTION FOR
PHOENIX AND VICINITY

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 30



LEGEND

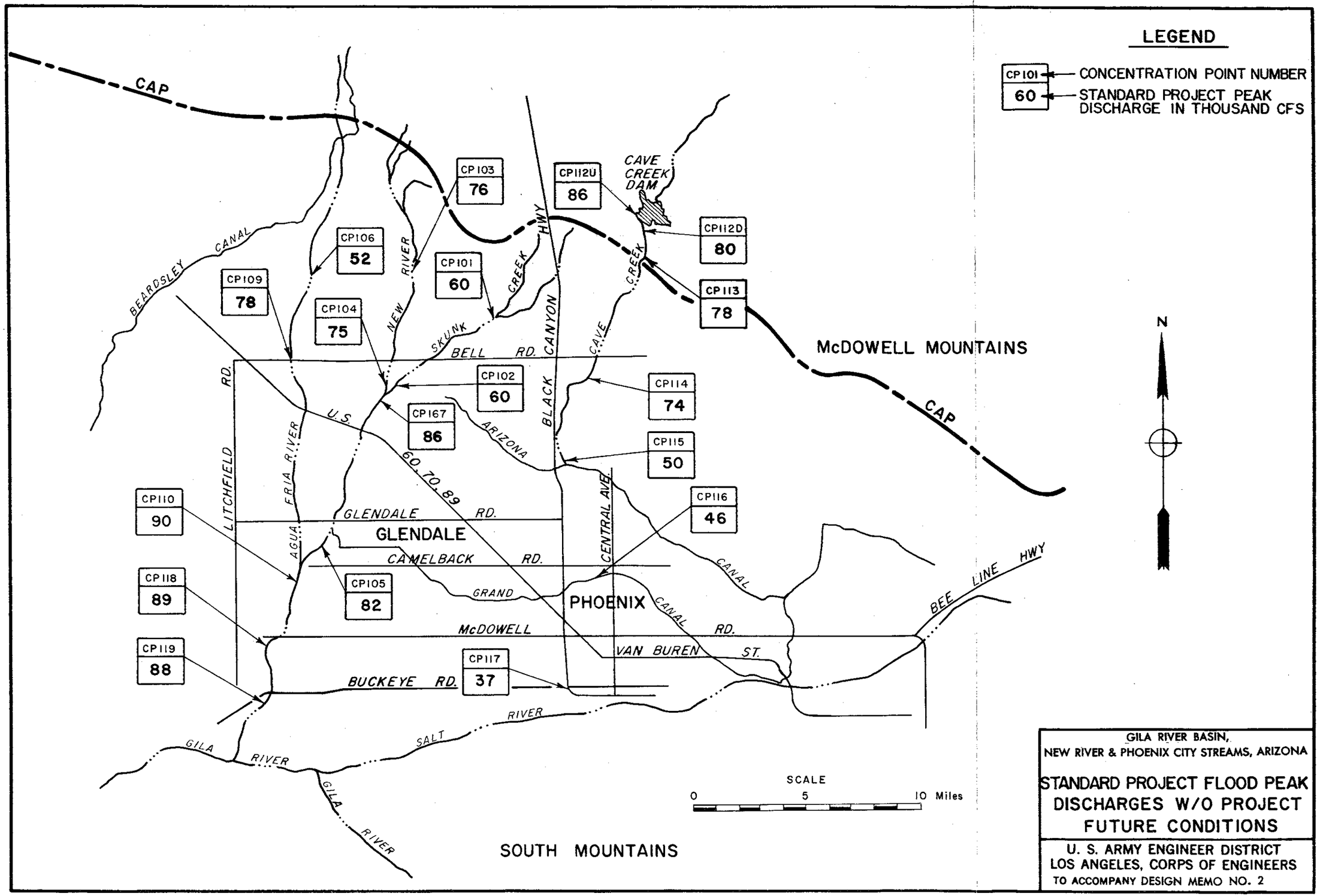
CP101 — CONCENTRATION POINT NUMBER

60 — STANDARD PROJECT PEAK DISCHARGE IN THOUSAND CFS

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

STANDARD PROJECT FLOOD PEAK DISCHARGES W/O PROJECT PRESENT CONDITIONS

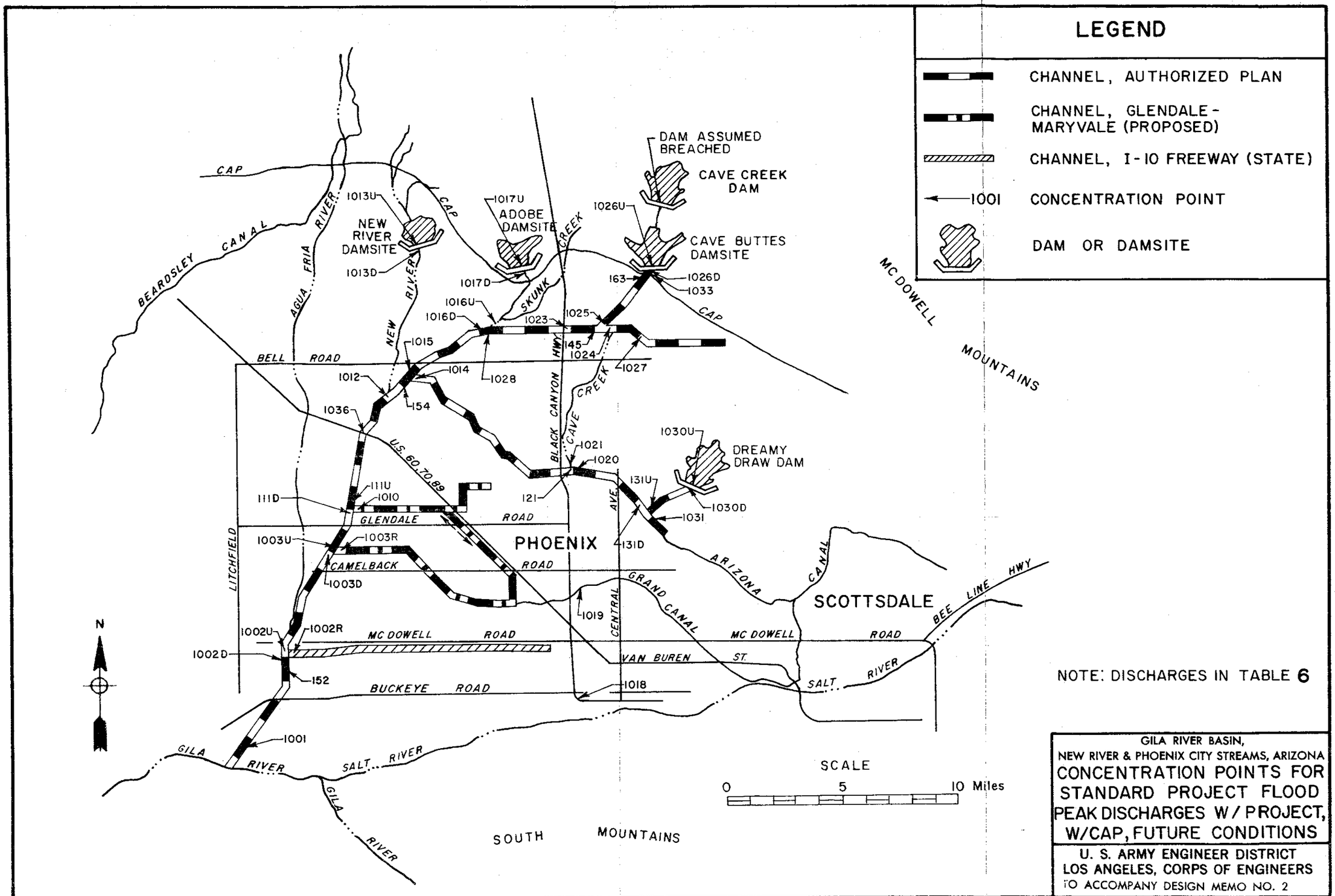
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

STANDARD PROJECT FLOOD PEAK DISCHARGES W/O PROJECT FUTURE CONDITIONS

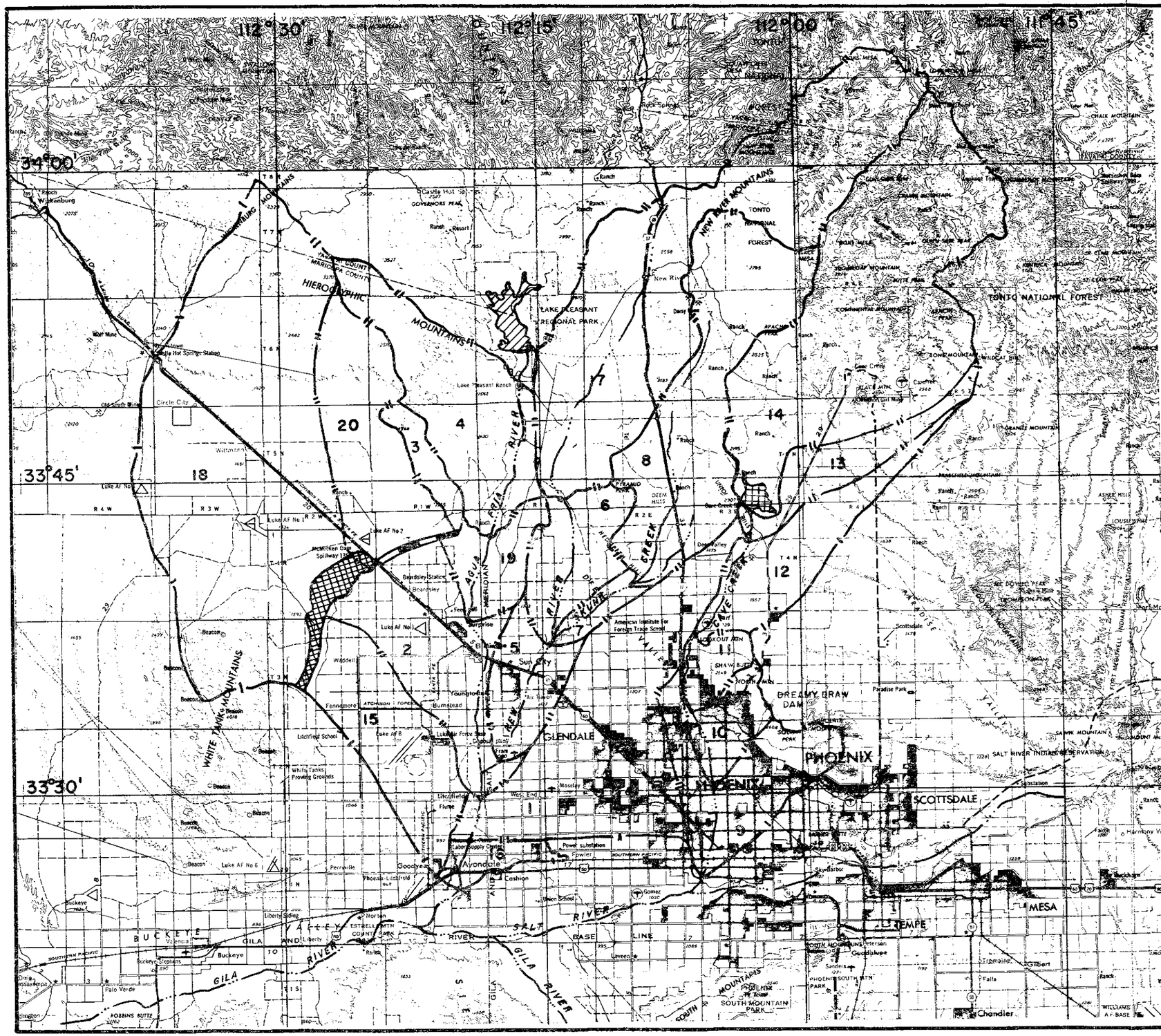
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



NOTE: DISCHARGES IN TABLE 6

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
CONCENTRATION POINTS FOR STANDARD PROJECT FLOOD PEAK DISCHARGES W/ PROJECT, W/CAP, FUTURE CONDITIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



LEGEND

—I—	BOUNDARY OF DRAINAGE AREA
—II—	BOUNDARY OF DRAINAGE SUBAREA
	EXISTING FLOOD CONTROL DAM
	EXISTING WATER CONSERVATION DAM
	EXISTING CHANNEL
18	SUBAREA NUMBER

SCALE

5 0 5 10

Miles

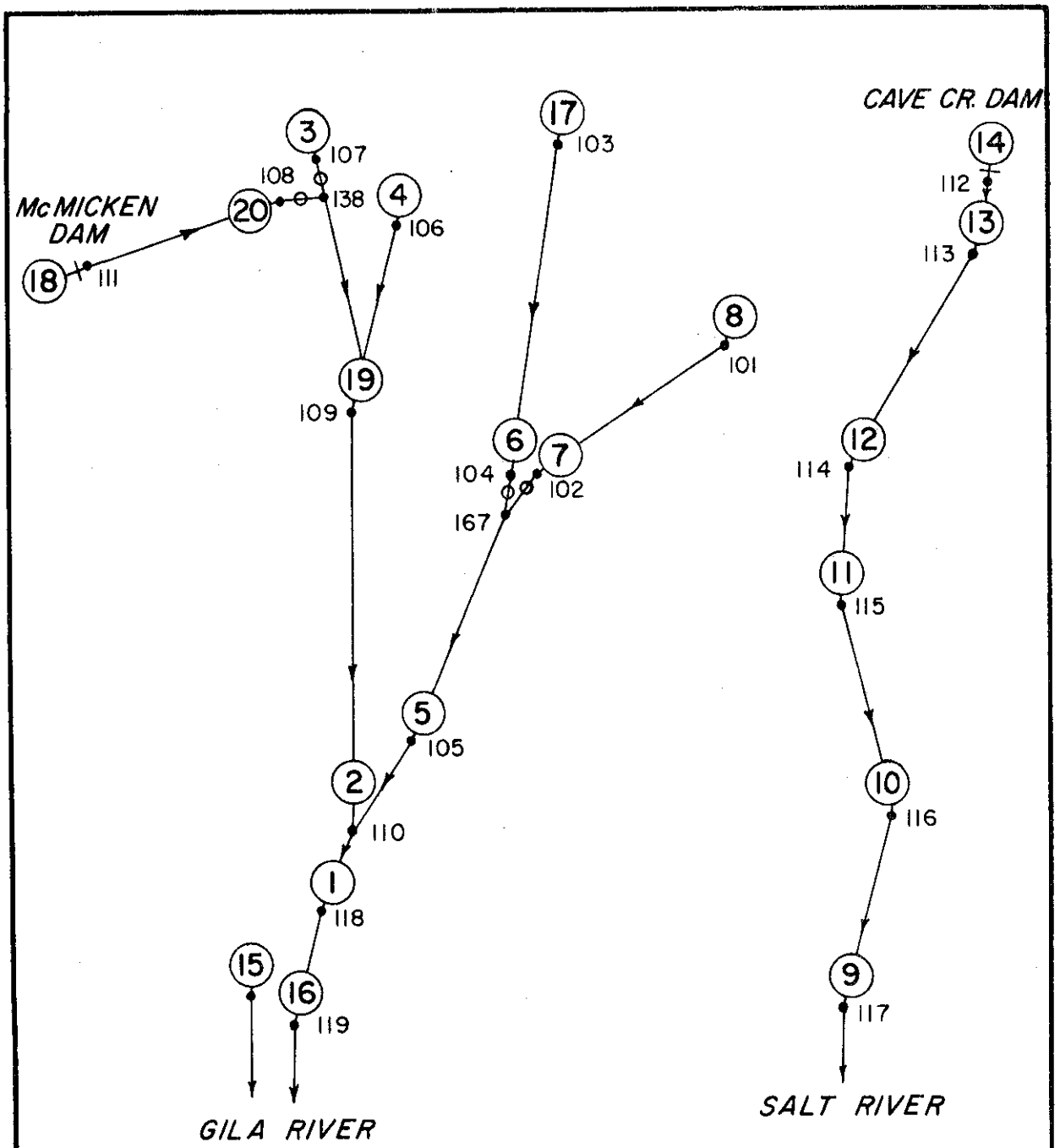
N

NOTE: SCHEMATIC FLOW DIAGRAM, SHOWN ON PLATE 35.

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

SUBAREA DELINEATION W/O PROJECT PRESENT CONDITIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



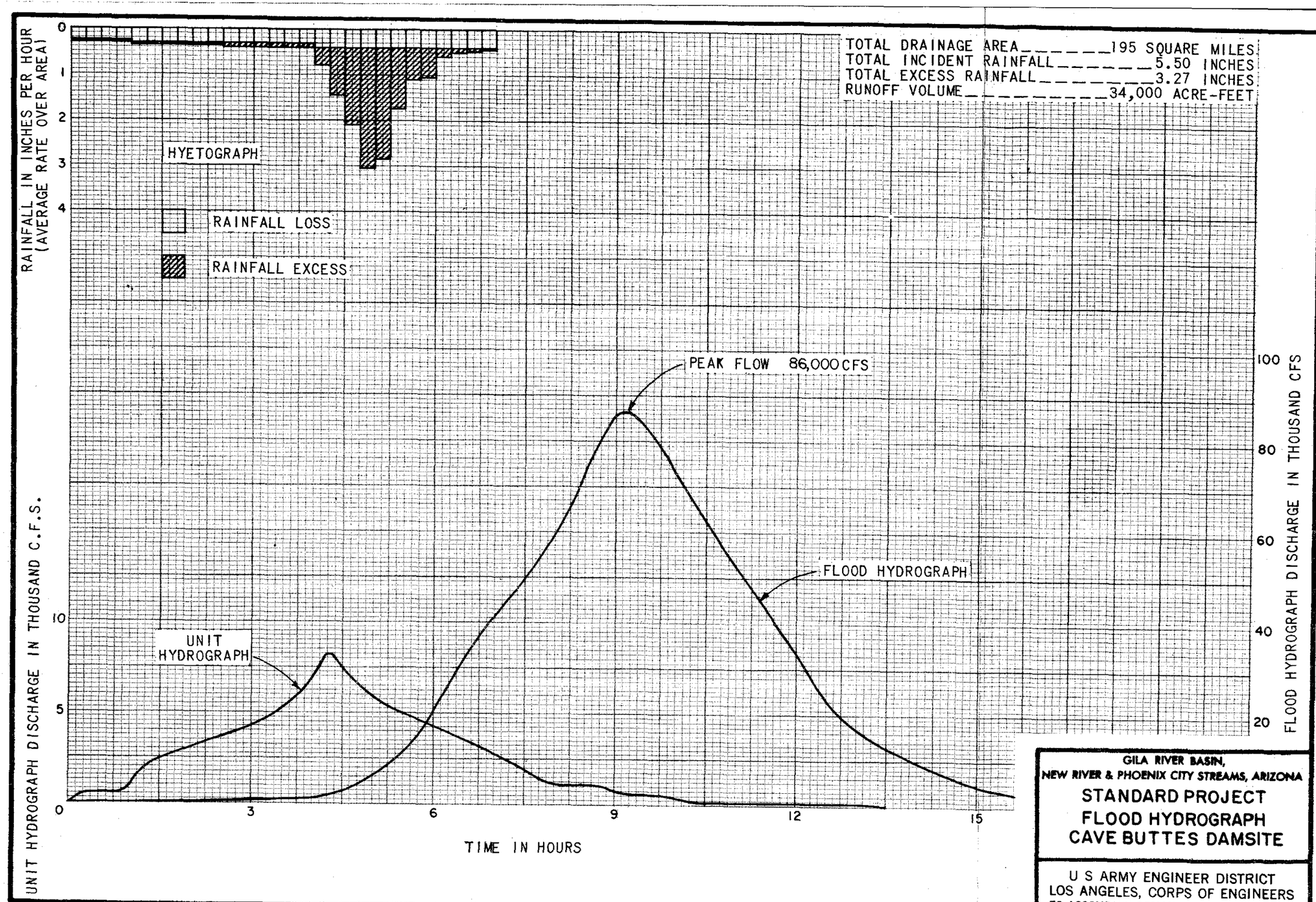
LEGEND

- (5) Subarea
- Reach of zero length
- Direction of flow
- Obstruction
- Concentration point

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

SCHEMATIC FLOW DIAGRAM W/O PROJECT PRESENT CONDITIONS

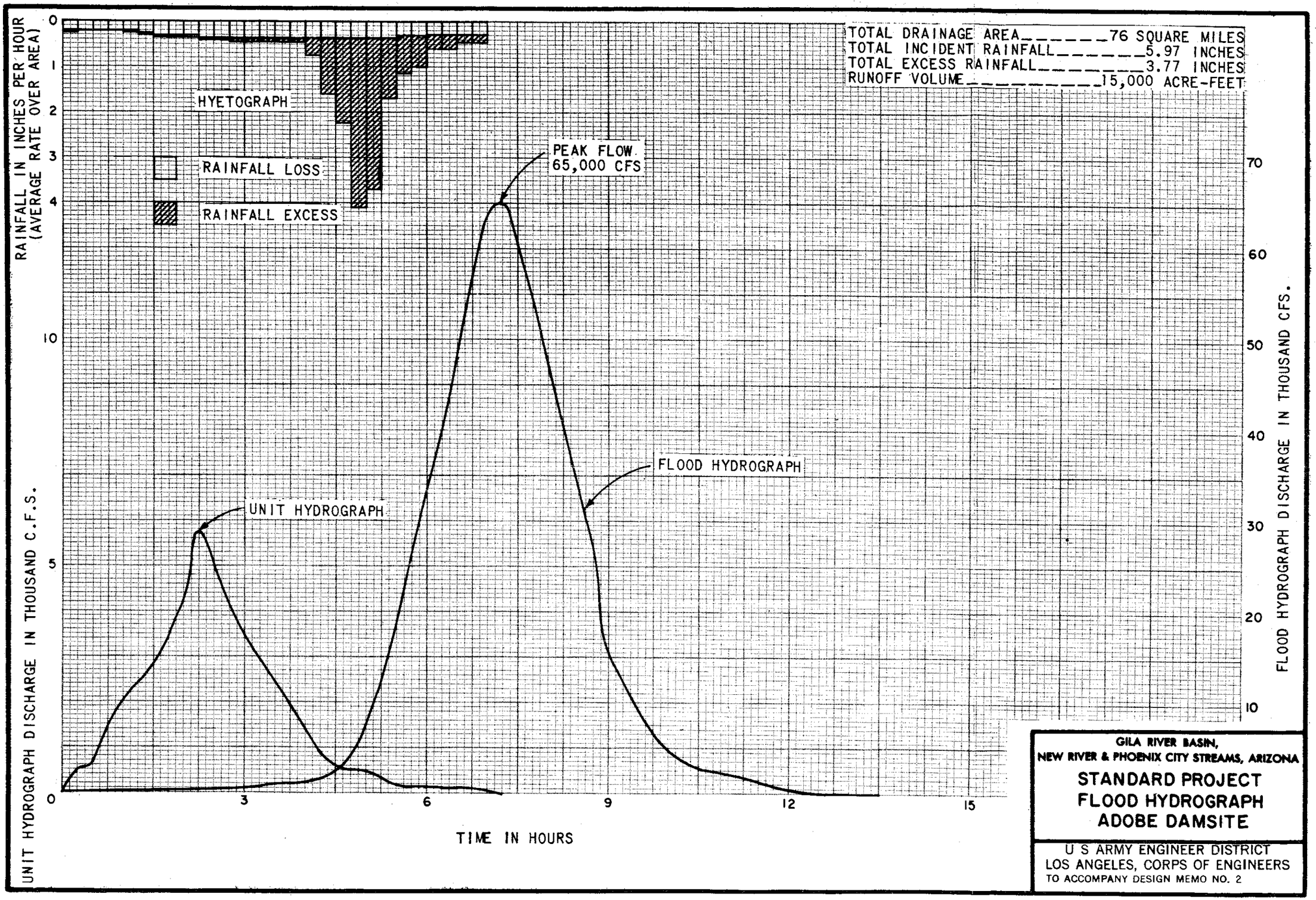
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

STANDARD PROJECT FLOOD HYDROGRAPH CAVE BUTTES DAMSITE

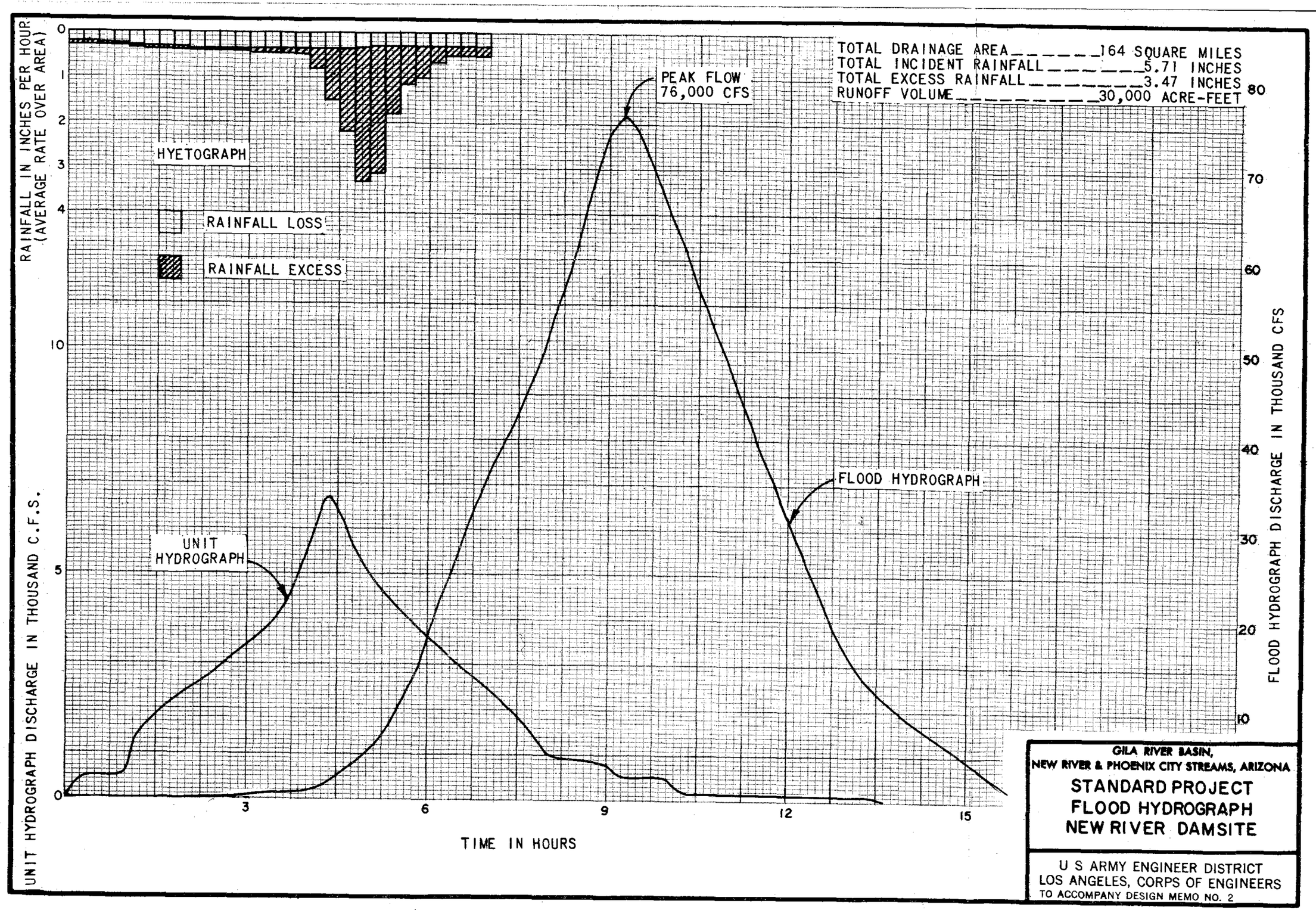
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

STANDARD PROJECT FLOOD HYDROGRAPH ADOBE DAMSITE

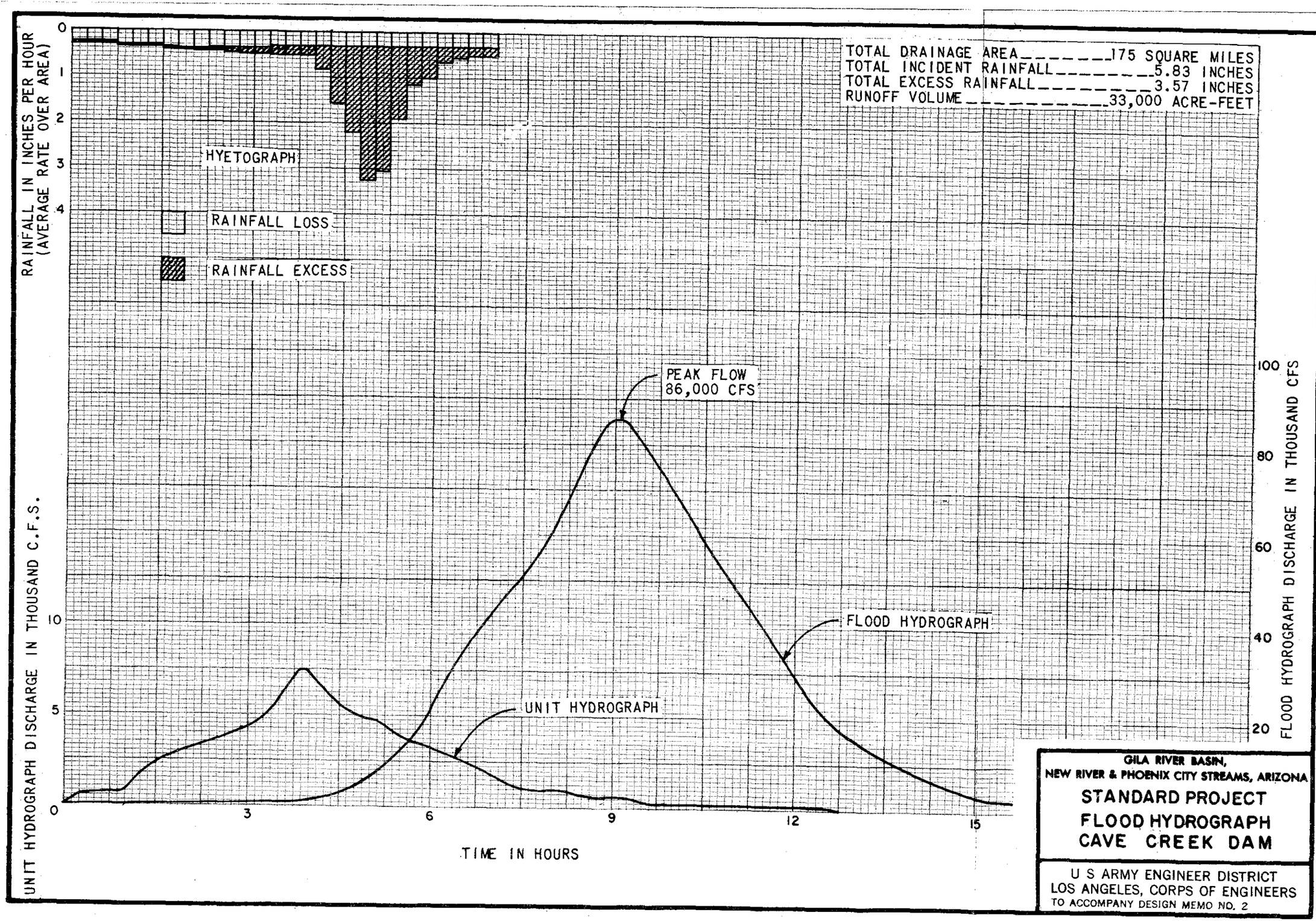
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

STANDARD PROJECT FLOOD HYDROGRAPH NEW RIVER DAMSITE

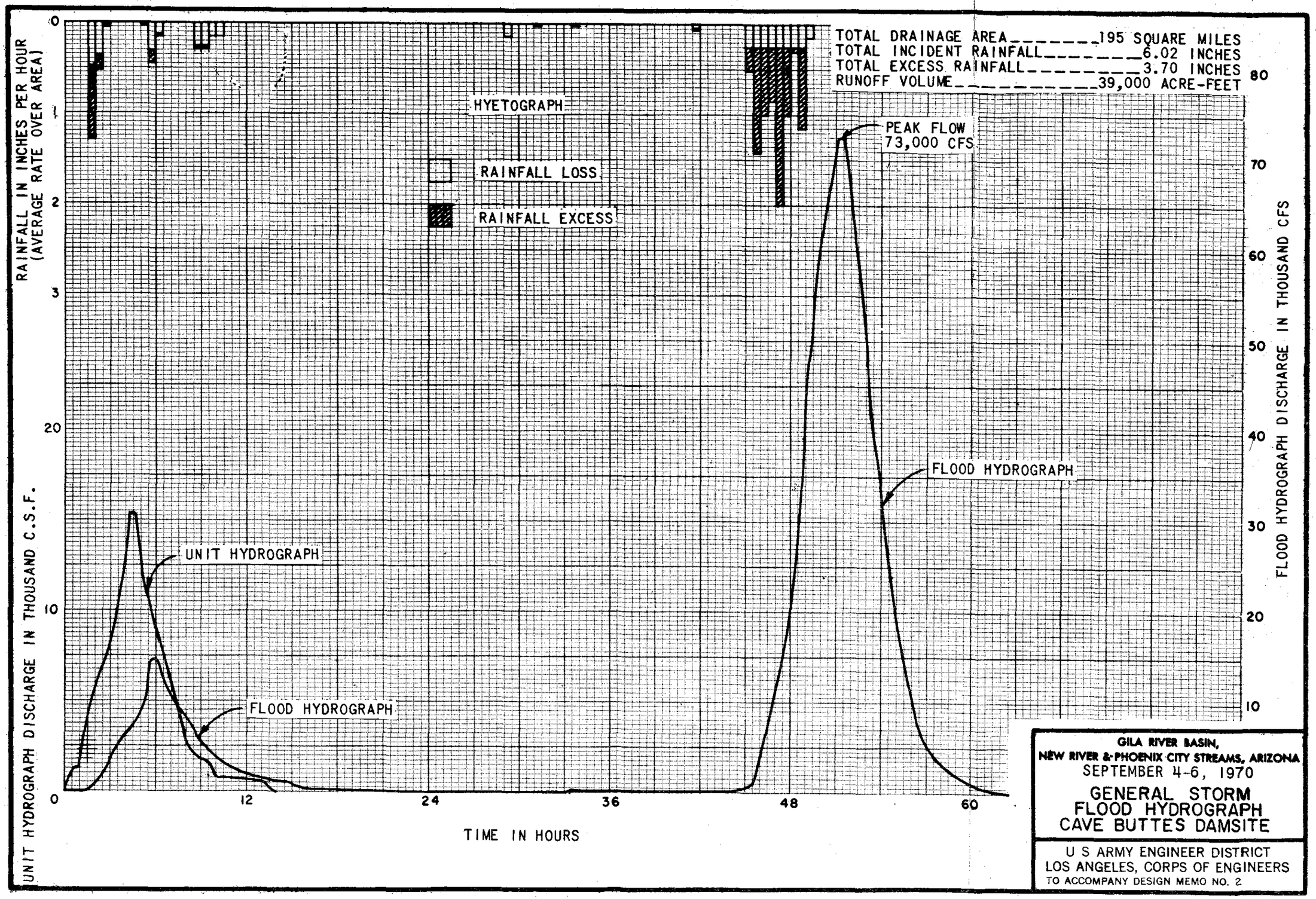
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

STANDARD PROJECT FLOOD HYDROGRAPH CAVE CREEK DAM

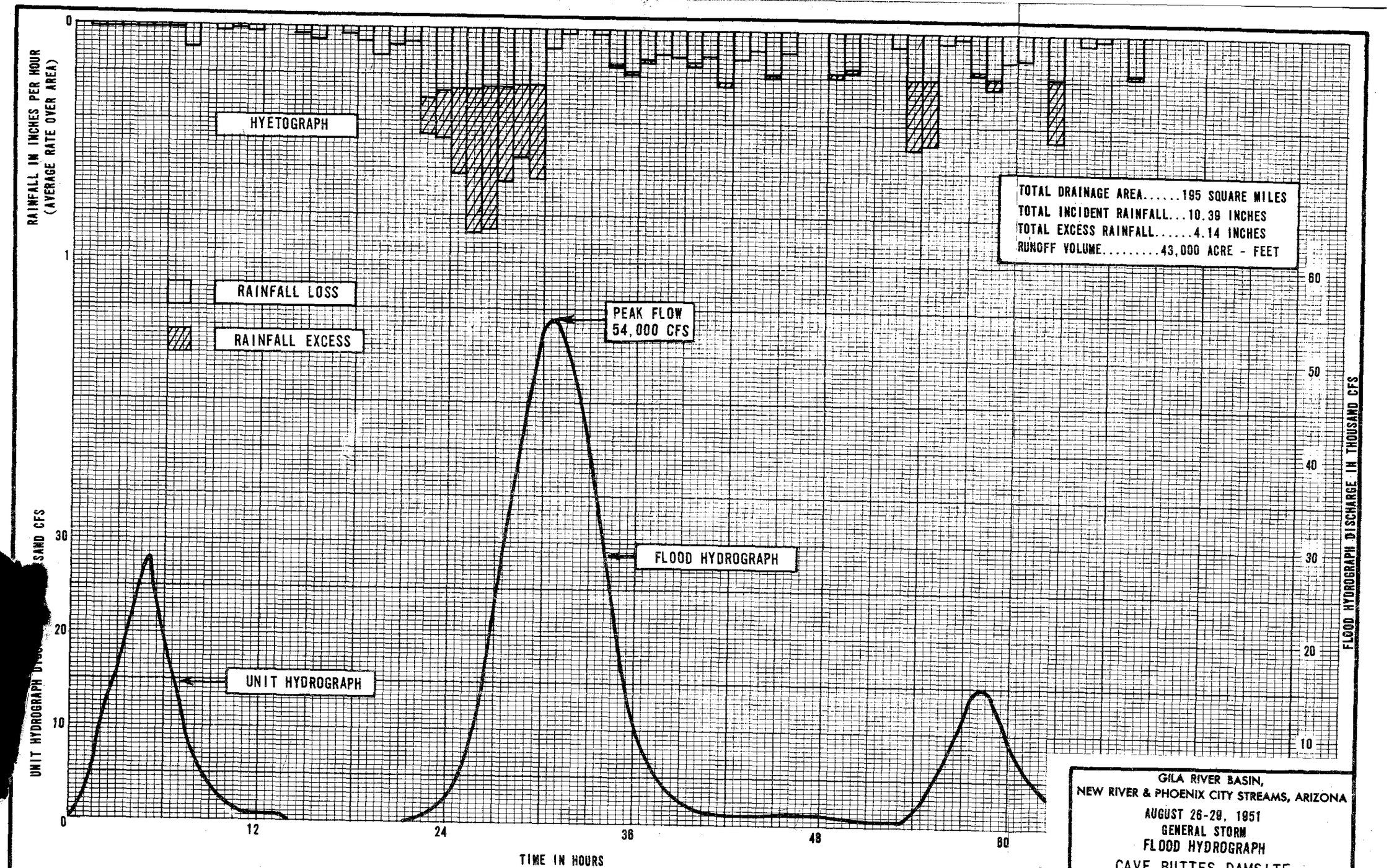
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
SEPTEMBER 4-6, 1970

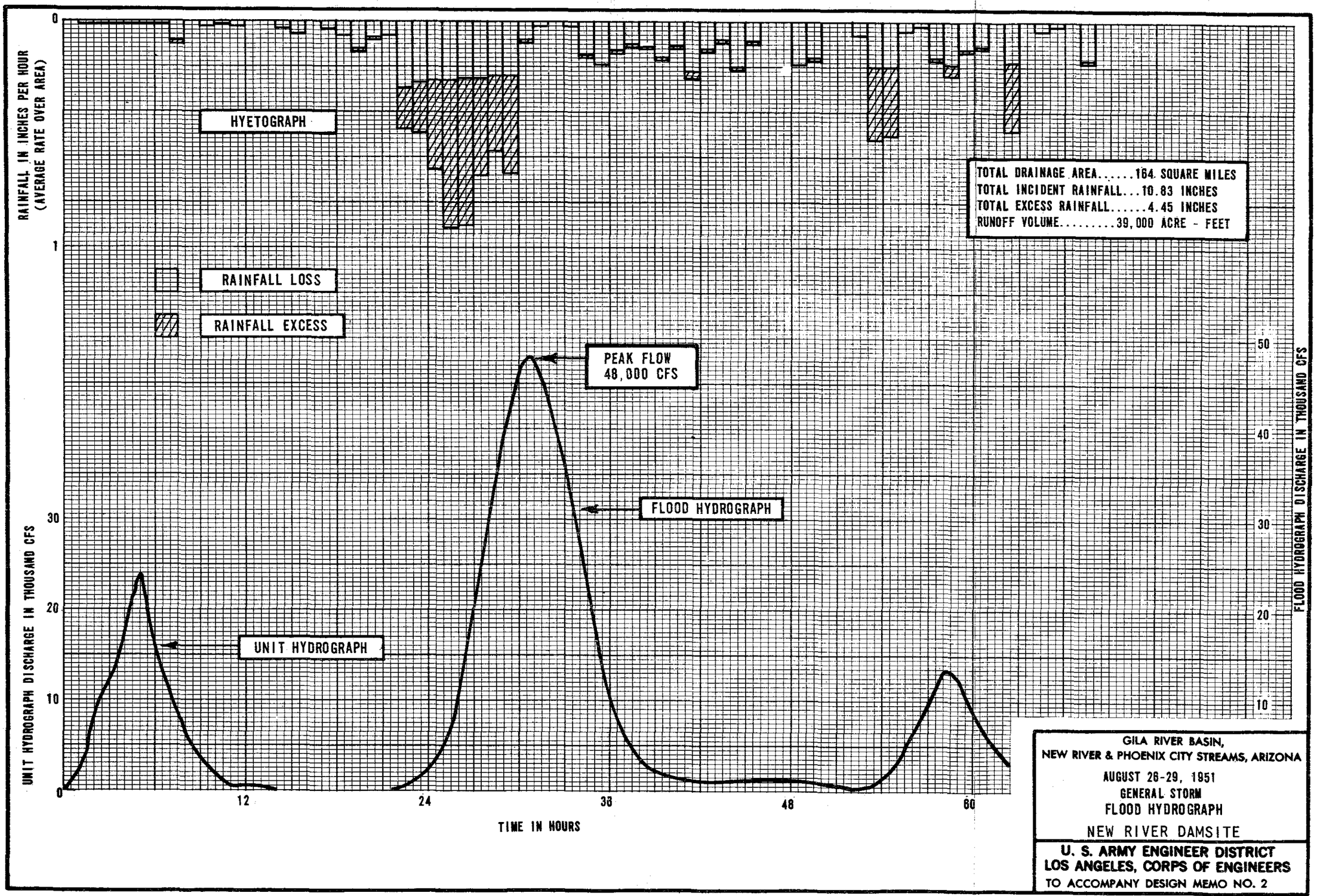
GENERAL STORM
FLOOD HYDROGRAPH
CAVE BUTTES DAMSITE

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
AUGUST 26-29, 1951
GENERAL STORM
FLOOD HYDROGRAPH
CAVE BUTTES DAMSITE

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

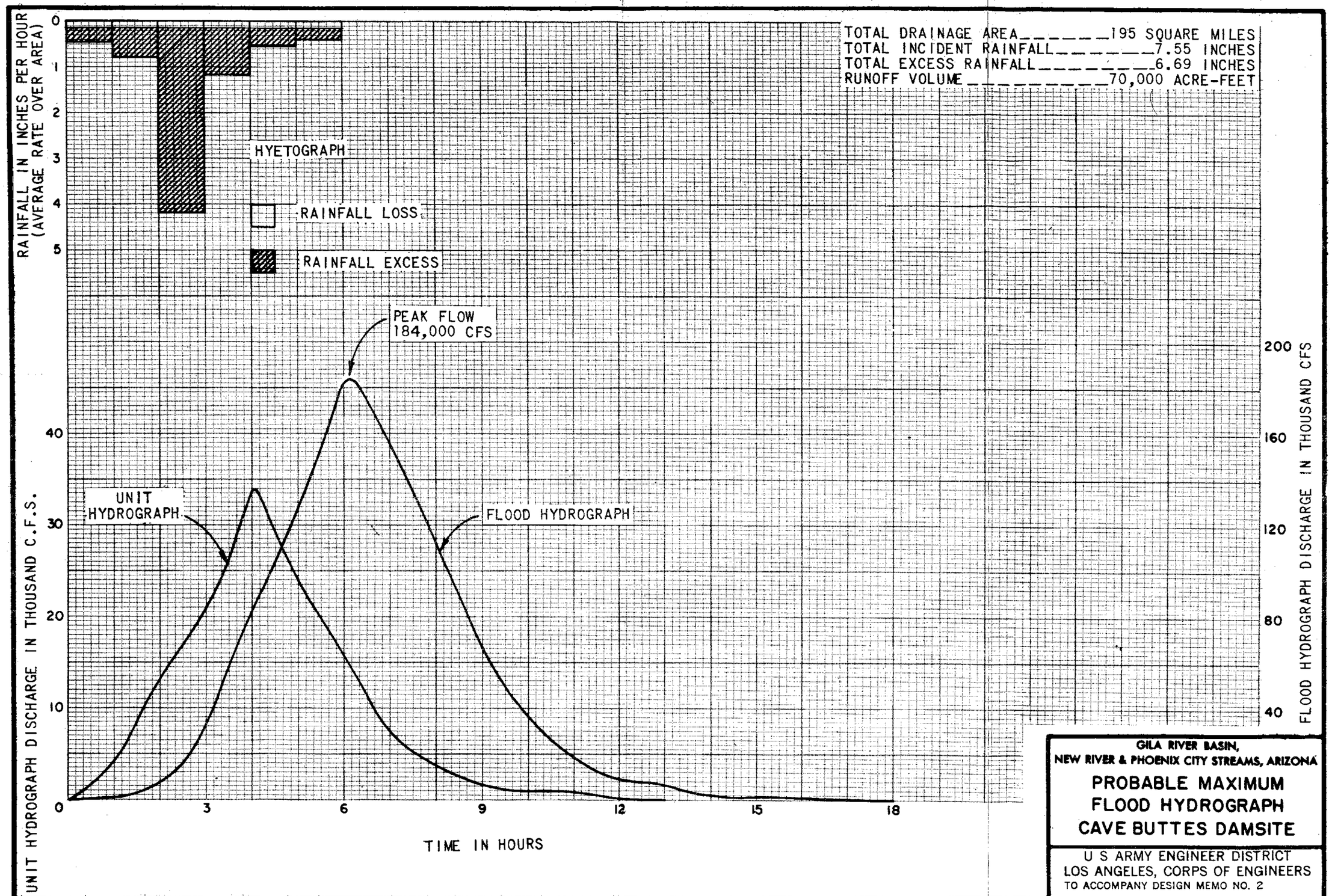


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

AUGUST 26-29, 1951
GENERAL STORM
FLOOD HYDROGRAPH

NEW RIVER DAMSITE

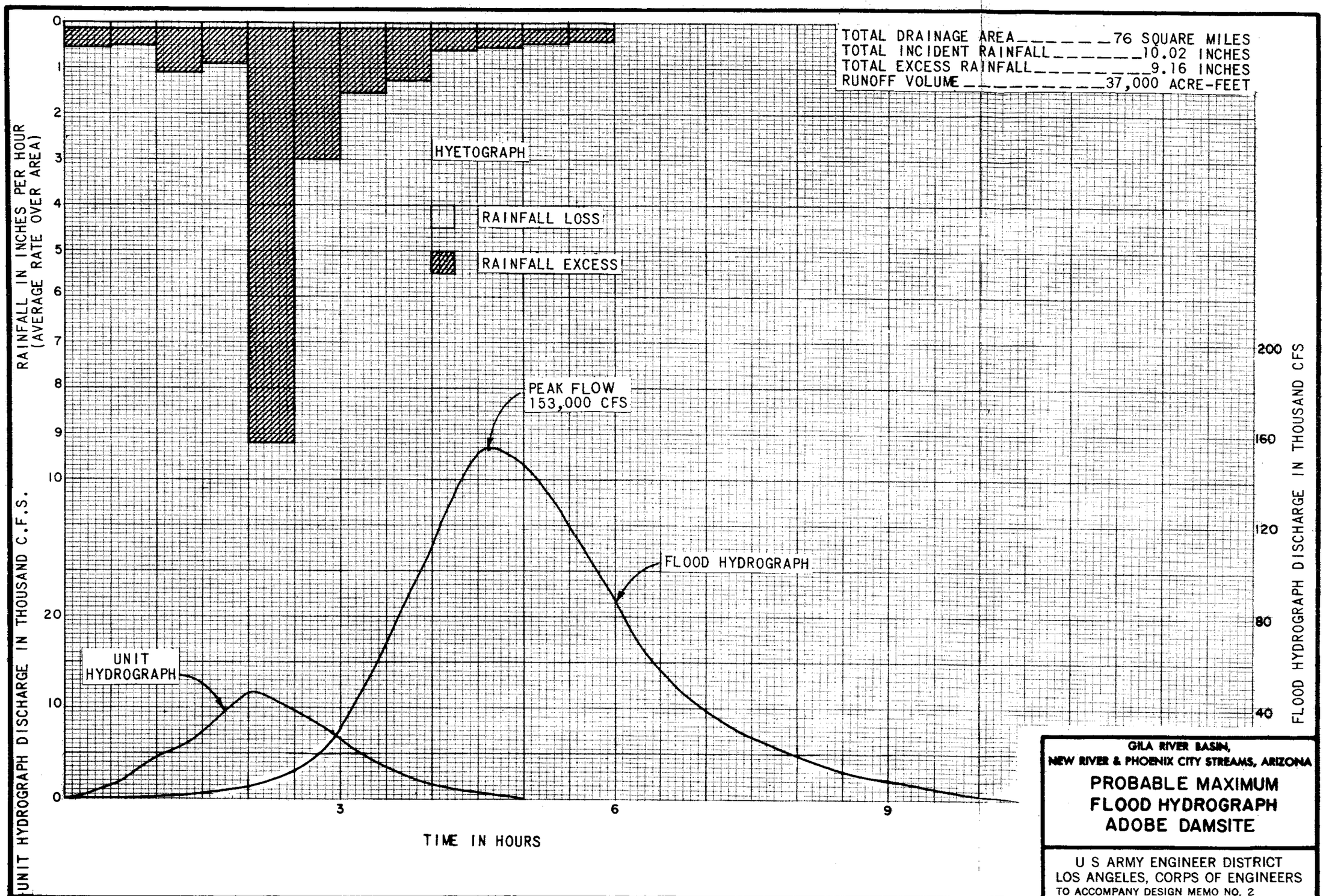
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

PROBABLE MAXIMUM FLOOD HYDROGRAPH CAVE BUTTES DAMSITE

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

PROBABLE MAXIMUM FLOOD HYDROGRAPH ADOBE DAMSITE

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

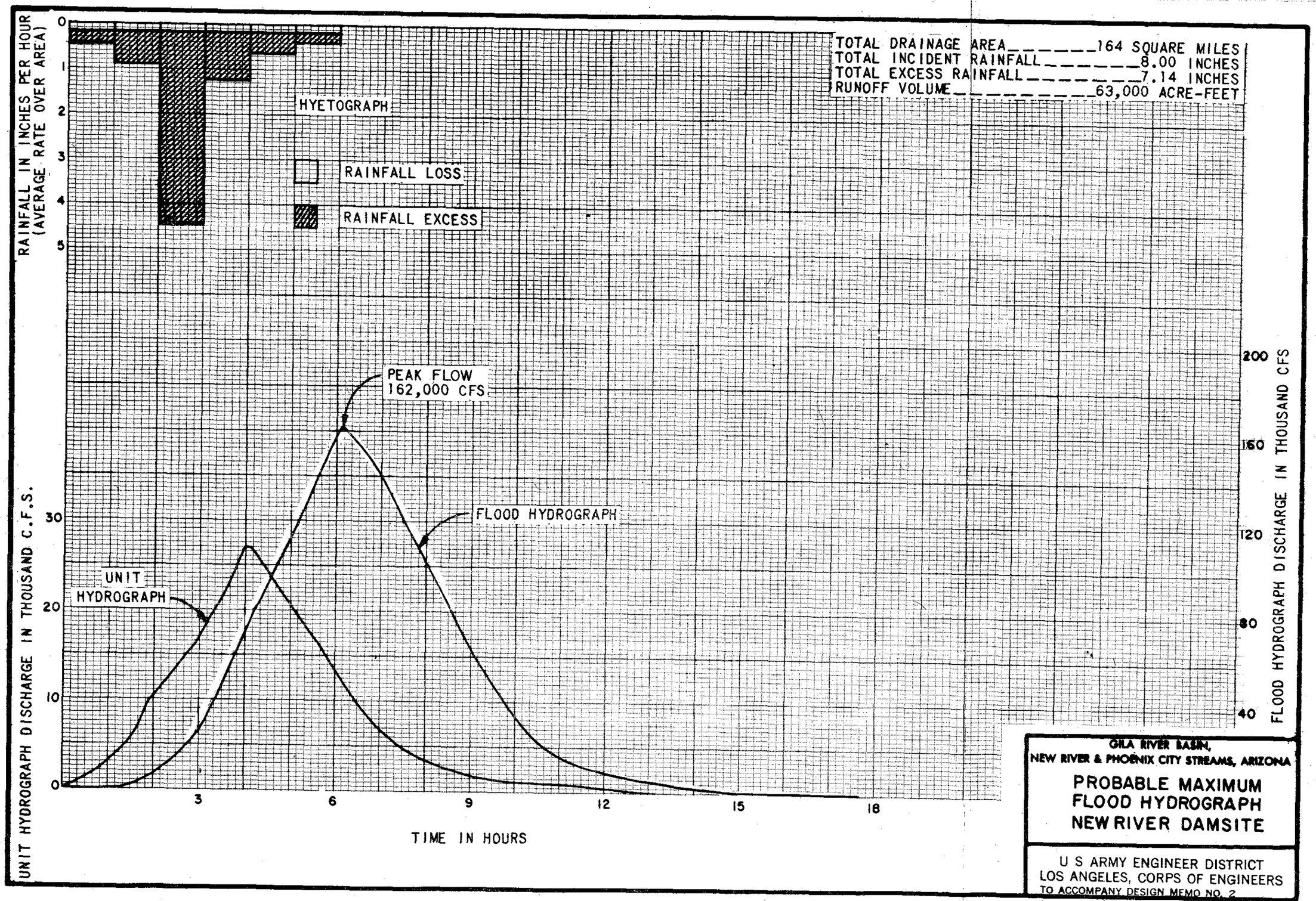
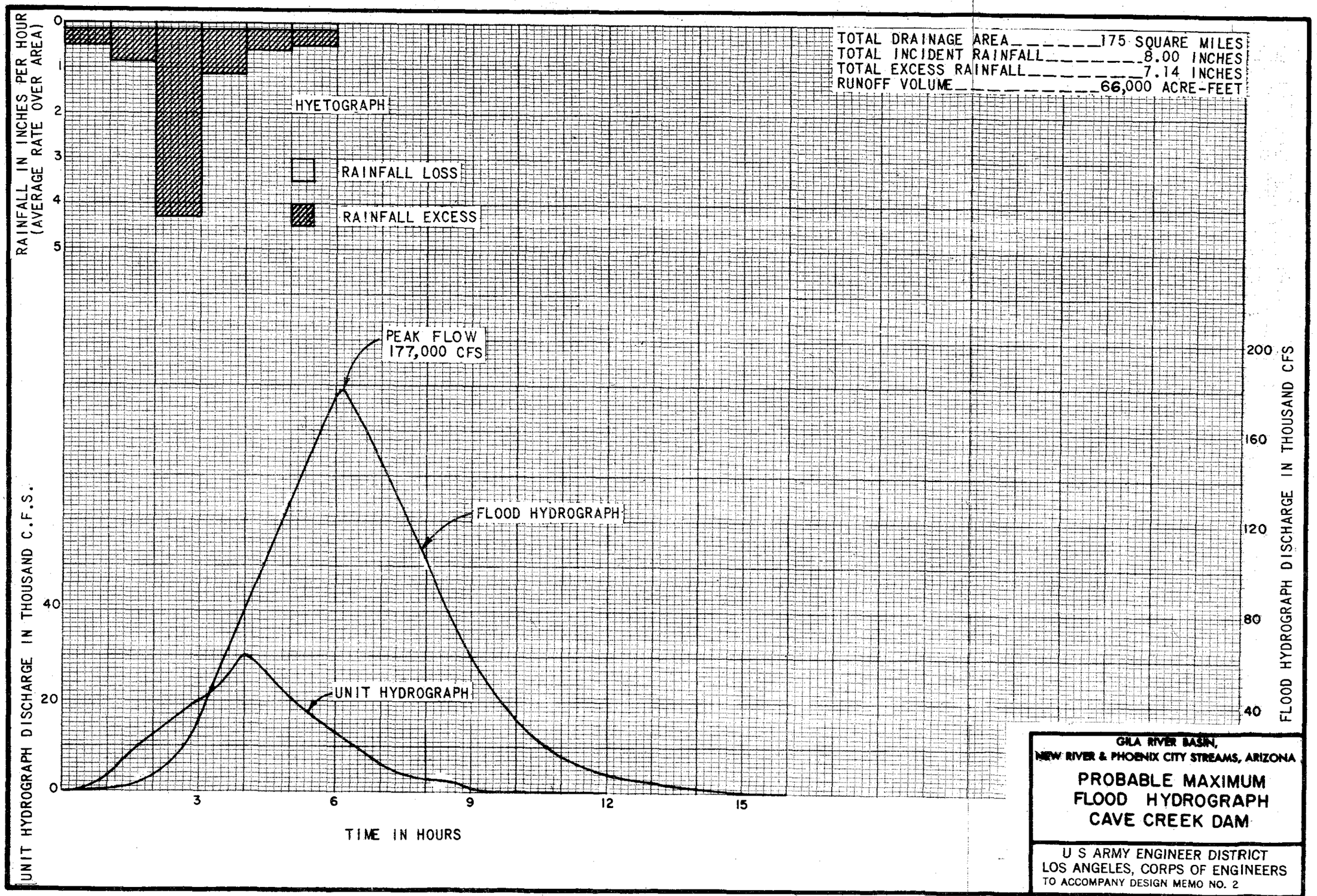

TOTAL DRAINAGE AREA 164 SQUARE MILES
TOTAL INCIDENT RAINFALL 8.00 INCHES
TOTAL EXCESS RAINFALL 7.14 INCHES
RUNOFF VOLUME 63,000 ACRE-FEET
RAINFALL IN INCHES PER HOUR (AVERAGE RATE OVER AREA)
0
1
2
3
4
5
HYETOGRAPH
RAINFALL LOSS
RAINFALL EXCESS
PEAK FLOW 162,000 CFS
FLOOD HYDROGRAPH
UNIT HYDROGRAPH
UNIT HYDROGRAPH DISCHARGE IN THOUSAND C.F.S.
0
10
20
30
FLOOD HYDROGRAPH DISCHARGE IN THOUSAND CFS
40
80
120
160
200
3
6
9
12
15
18
TIME IN HOURS
GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
PROBABLE MAXIMUM
FLOOD HYDROGRAPH
NEW RIVER DAMSITE
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 45

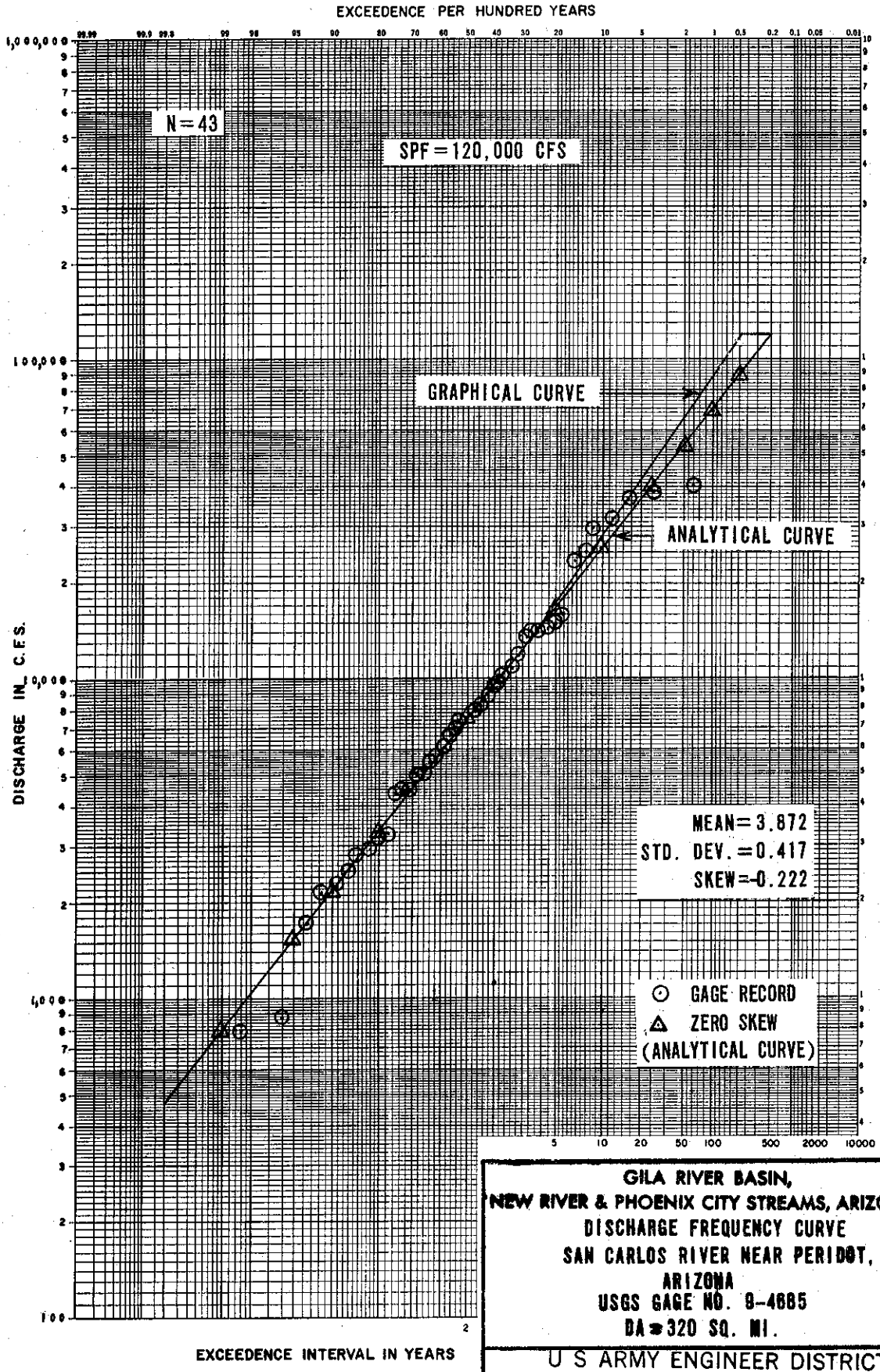


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

PROBABLE MAXIMUM
FLOOD HYDROGRAPH
CAVE CREEK DAM

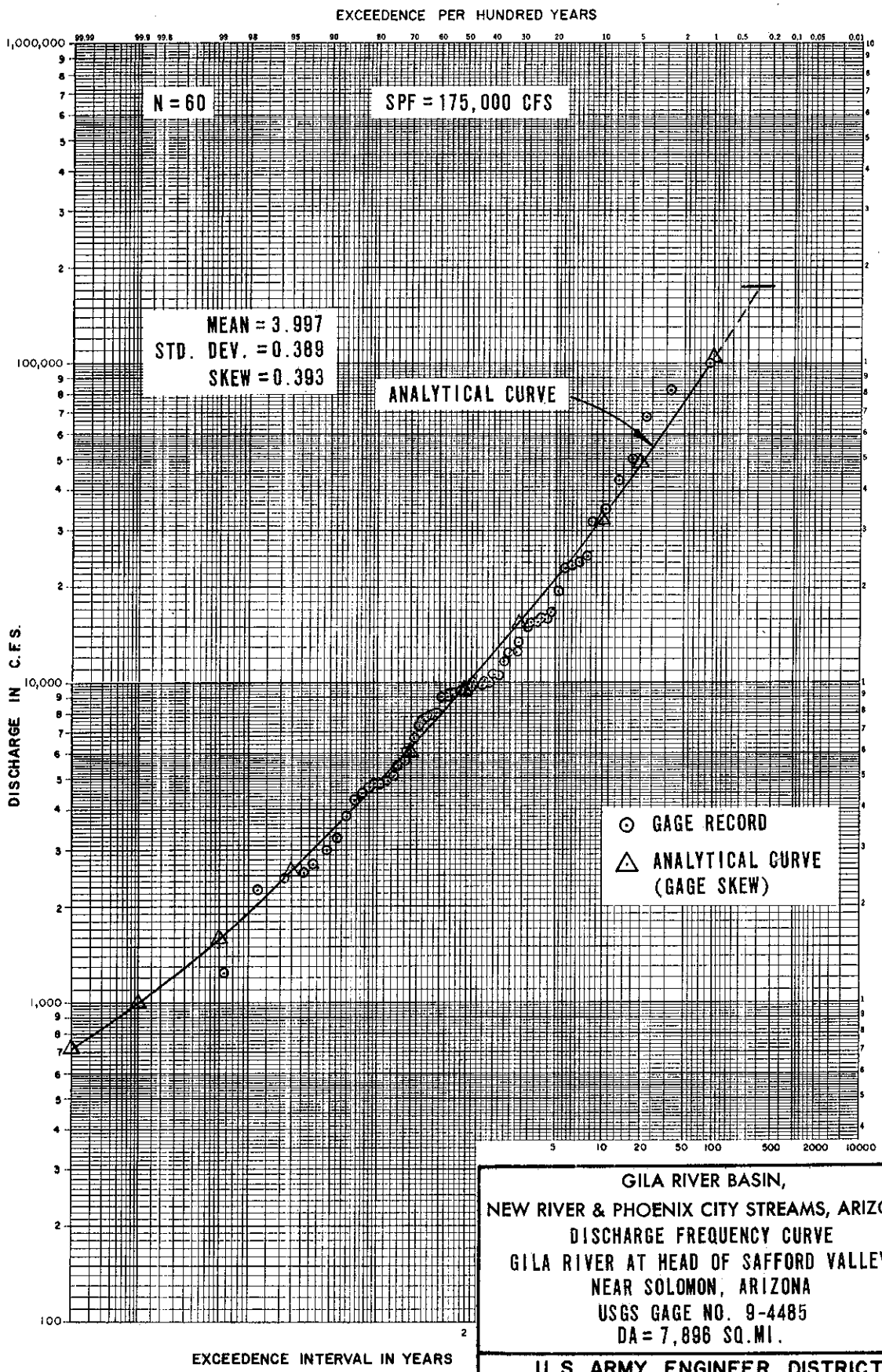
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 46



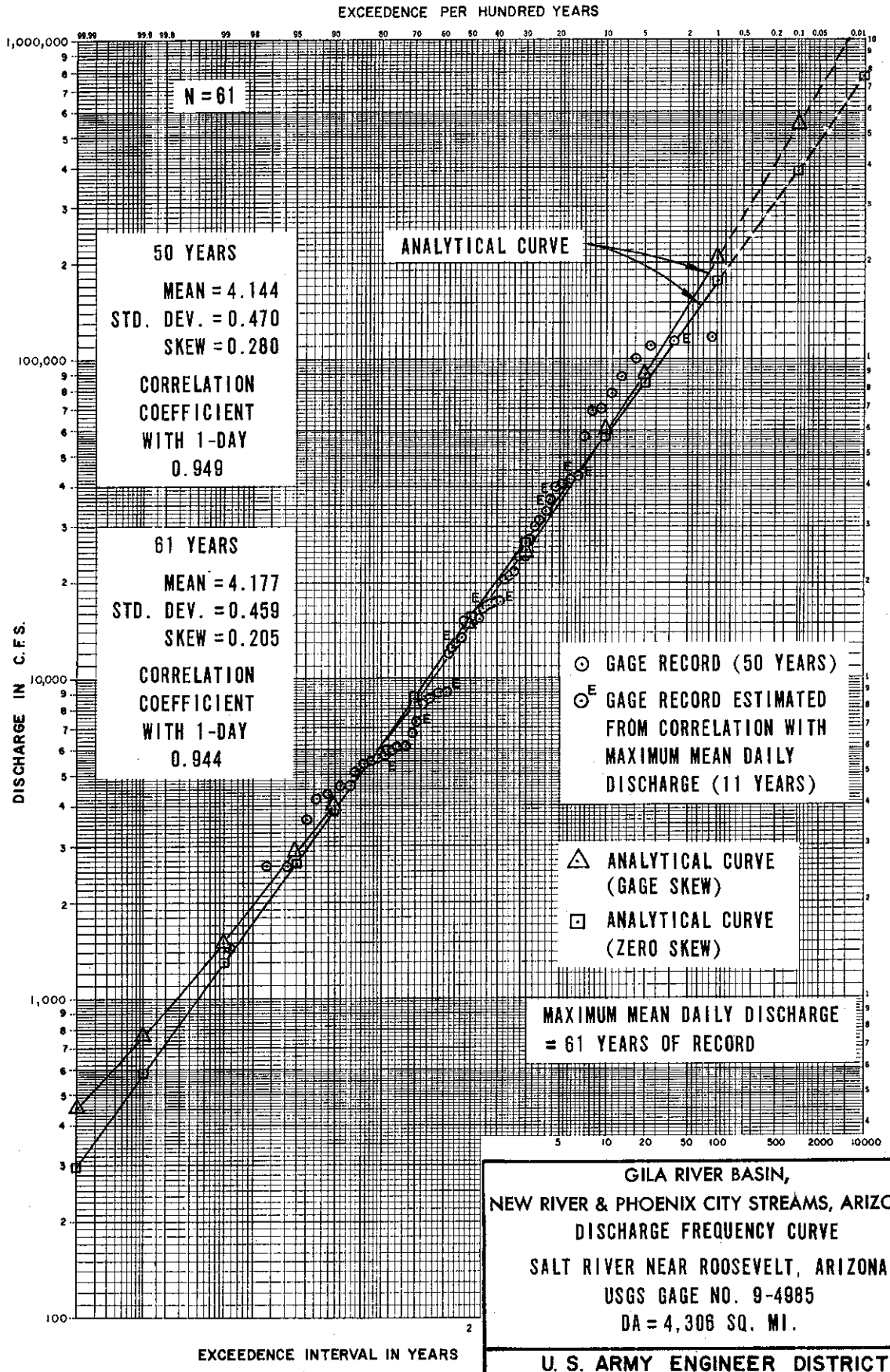
GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE FREQUENCY CURVE
SAN CARLOS RIVER NEAR PERIDOT, ARIZONA
USGS GAGE NO. 9-4665
DA=320 SQ. MI.

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE FREQUENCY CURVE
GILA RIVER AT HEAD OF SAFFORD VALLEY,
NEAR SOLOMON, ARIZONA
USGS GAGE NO. 9-4485
DA = 7,896 SQ.MI.

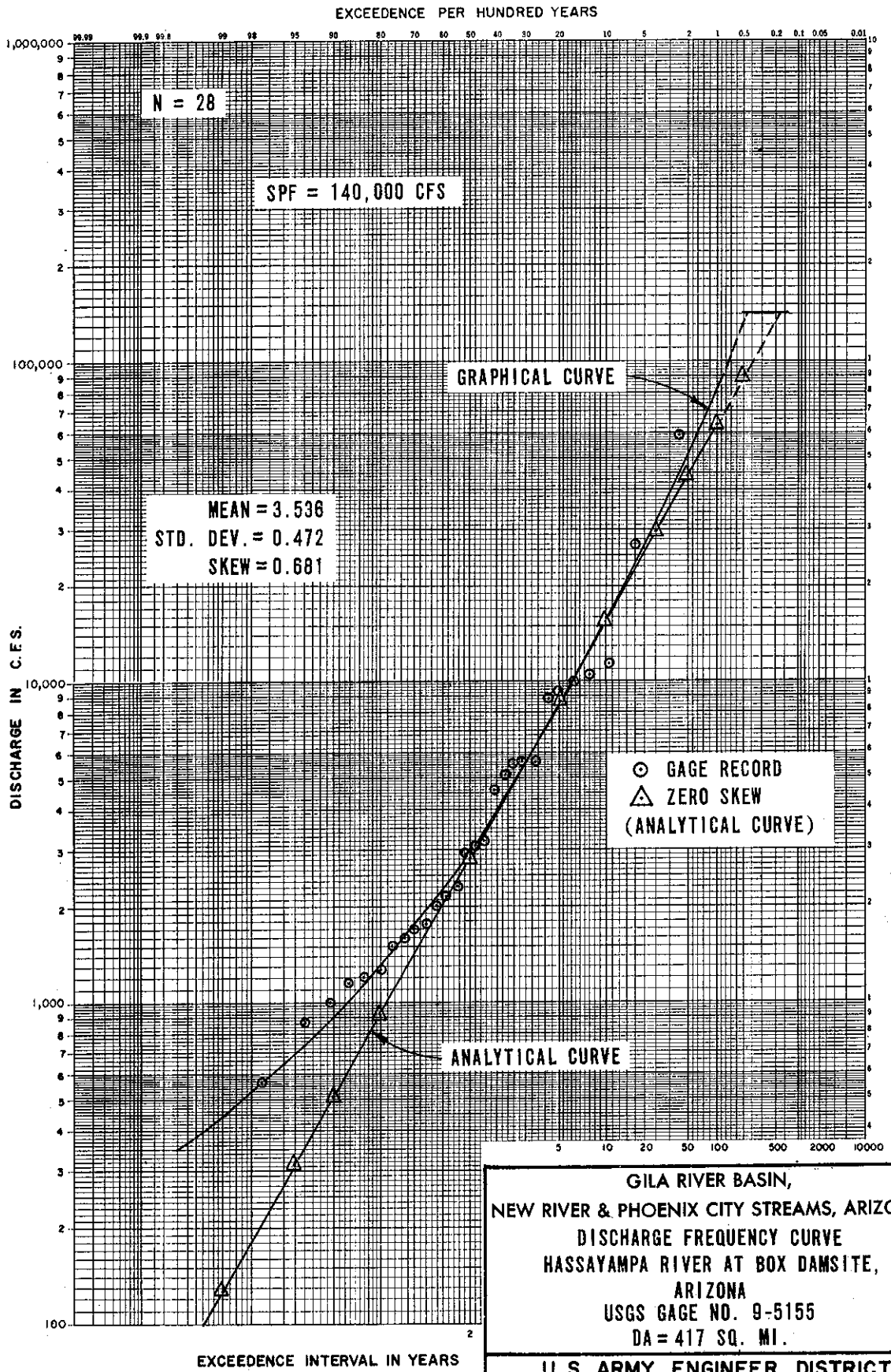
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE FREQUENCY CURVE

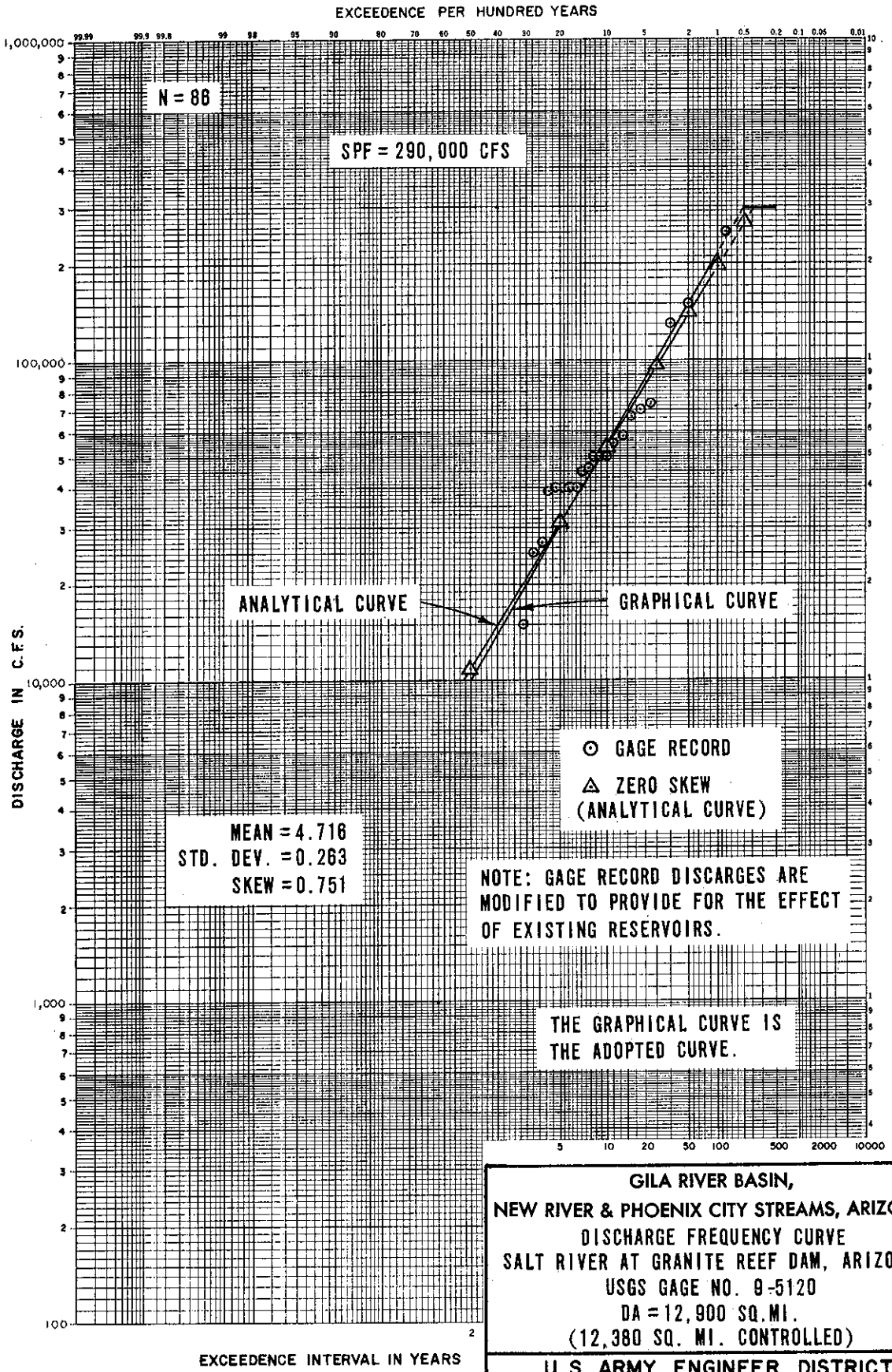
SALT RIVER NEAR ROOSEVELT, ARIZONA
USGS GAGE NO. 9-4985
DA = 4,306 SQ. MI.

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



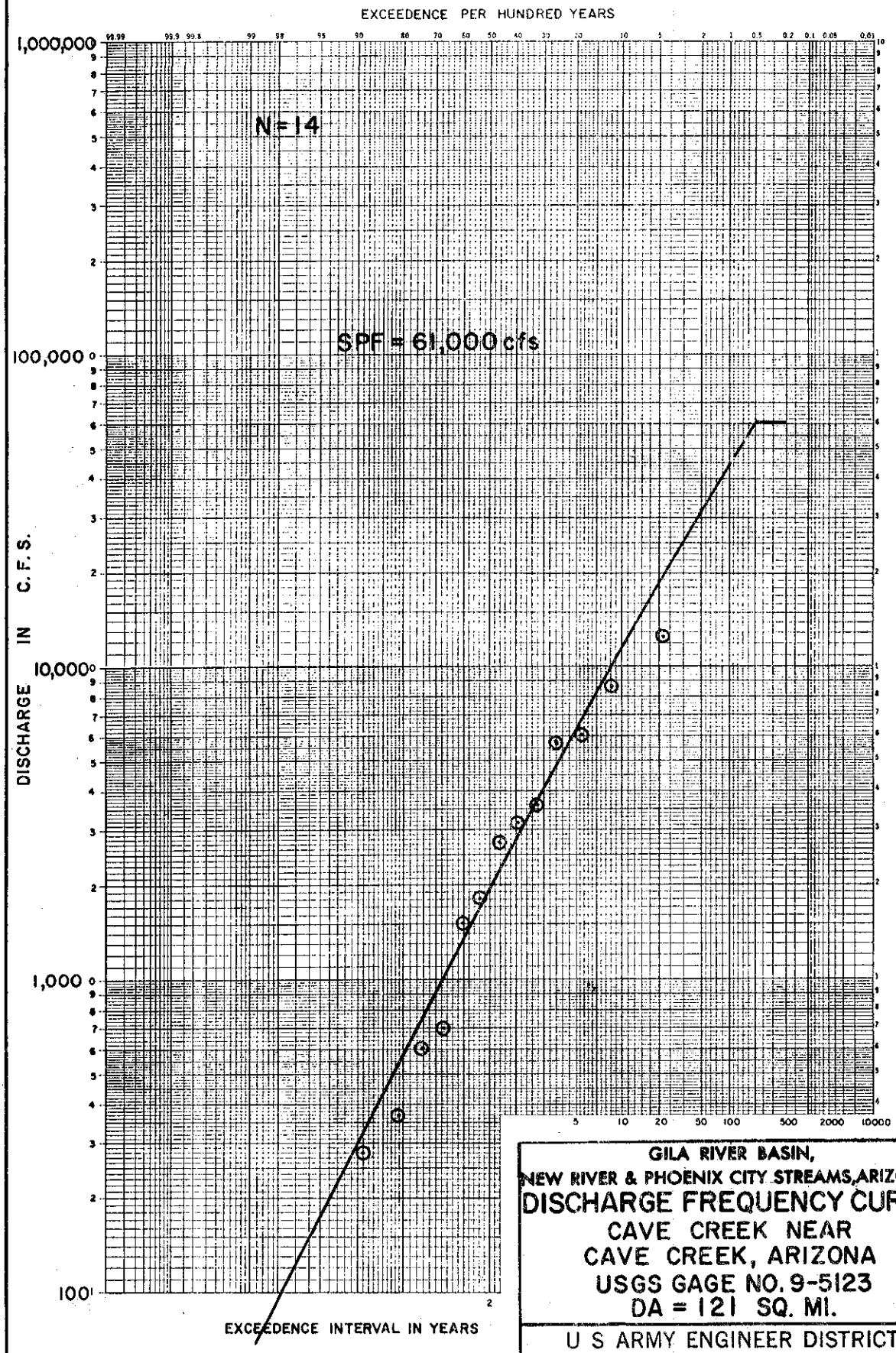
GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE FREQUENCY CURVE
HASSAYAMPA RIVER AT BOX DAMSITE,
ARIZONA
USGS GAGE NO. 9-5155
DA = 417 SQ. MI.

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE FREQUENCY CURVE
SALT RIVER AT GRANITE REEF DAM, ARIZONA
USGS GAGE NO. 9-5120
DA = 12,900 SQ.MI.
(12,380 SQ. MI. CONTROLLED)

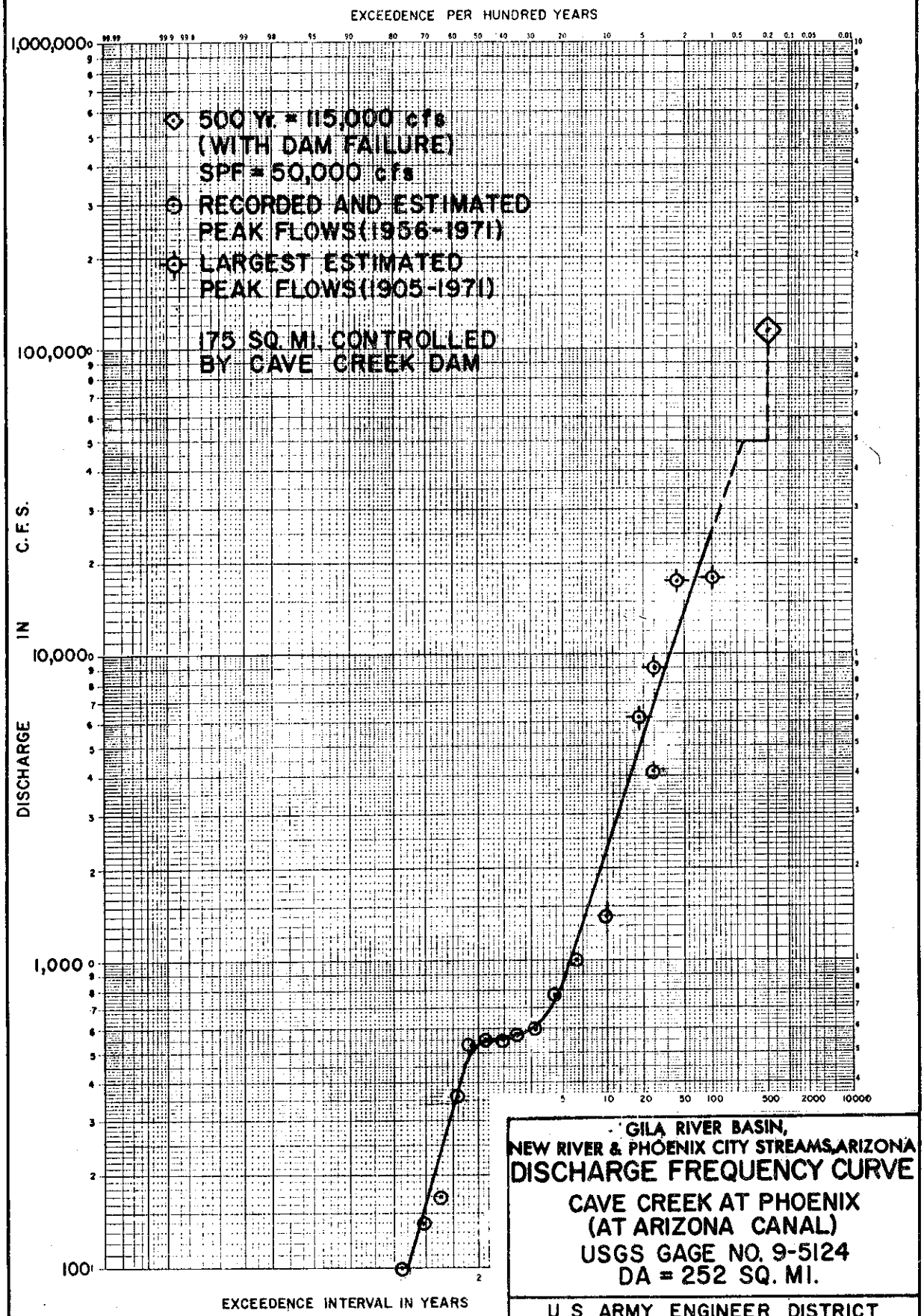
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE FREQUENCY CURVE
CAVE CREEK NEAR
CAVE CREEK, ARIZONA
USGS GAGE NO. 9-5123
DA = 121 SQ. MI.

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 50



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

DISCHARGE FREQUENCY CURVE

CAVE CREEK AT PHOENIX
(AT ARIZONA CANAL)
USGS GAGE NO. 9-5124
DA = 252 SQ. MI.

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

APPENDIX 2 - REFERENCES FOR STRUCTURES AFFECTING RUNOFF

1. Trilby Wash Dention Basin. Flood Control Lower Agua Fria and Vicinity, Maricopa County, Arizona Design Memorandum No. 2. Design analysis for Trilby Wash Detention Basin and Outlet Channel, 1 March 1954 (revised January 1955), Sacramento District, Corps of Engineers, Sacramento, California.

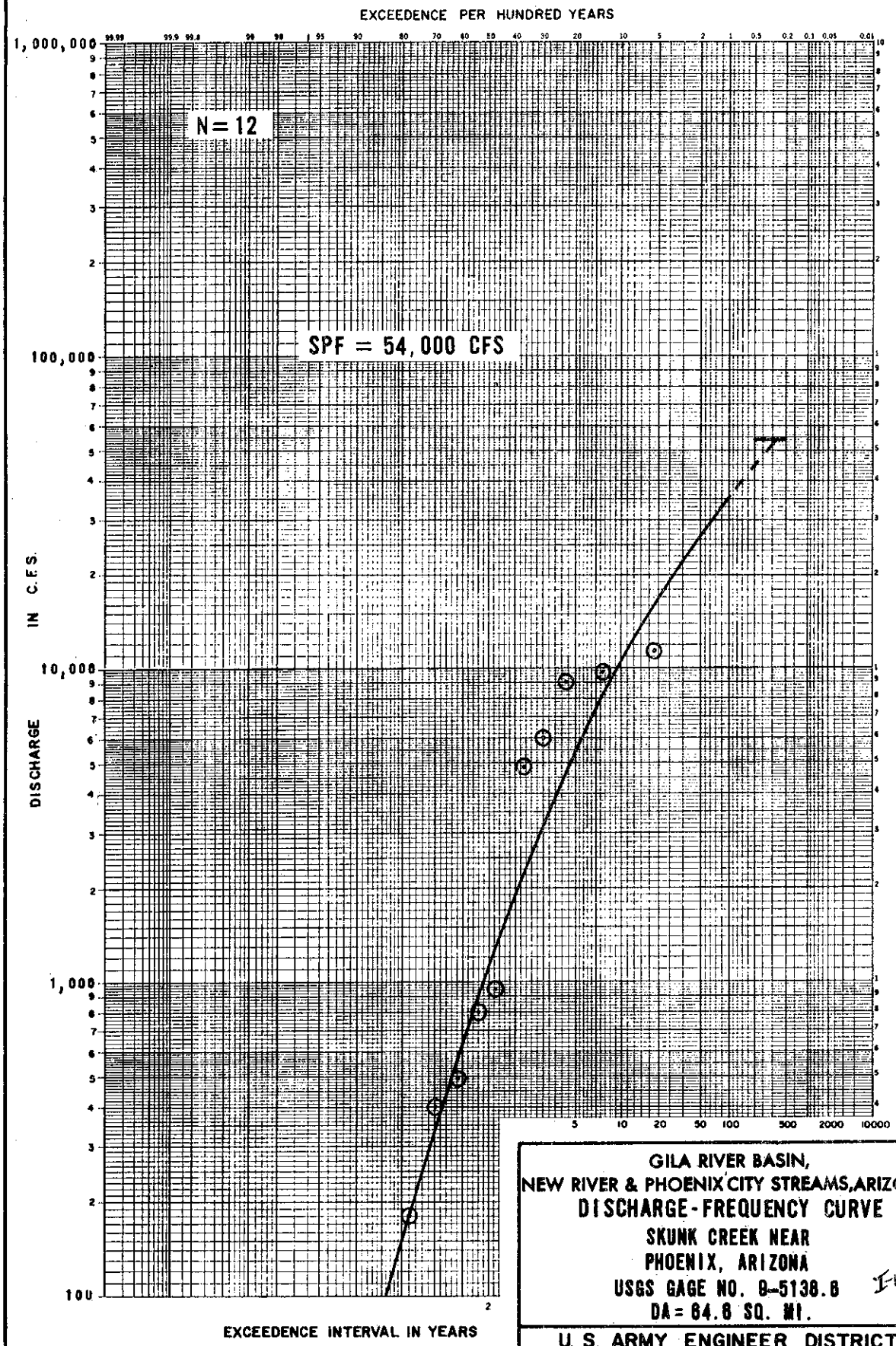
2. Central Arizona Project. Summary Report, Central Arizona Project with Federal Prepayment Power Arrangements, February 1967, Bureau of Reclamation.

3. Interstate 10 Freeway Channel. (Preliminary) Hydrology Reevaluation, Ehrenberg Phoenix Highway, Arizona Highway Department, November 1971.

4. Waddell Dam is operated by Maricopa County Municipal Water Conservation District No. 1.

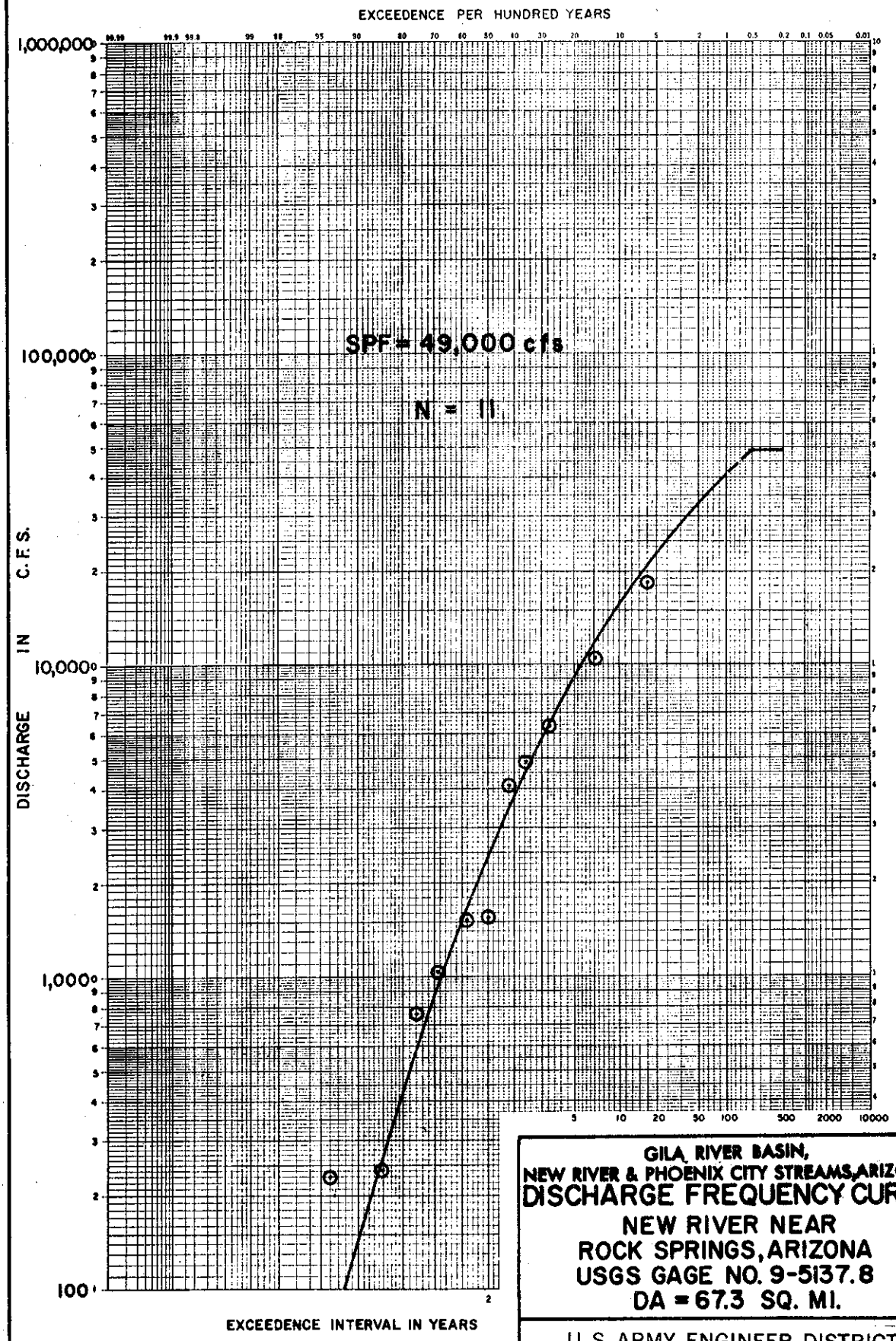
5. Proposed Glendale-Maryvale Channel. Interim Report of Survey for Flood Control, Phoenix, Arizona and Vicinity (including New River), Corps of Engineers, Los Angeles District, January 15, 1964.

6. Cave Creek Dam, completed in 1923, is under the jurisdiction of the Salt River Valley Water Users Association.



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE-FREQUENCY CURVE
SKUNK CREEK NEAR
PHOENIX, ARIZONA
USGS GAGE NO. 8-5138.8
DA = 64.8 SQ. MI.

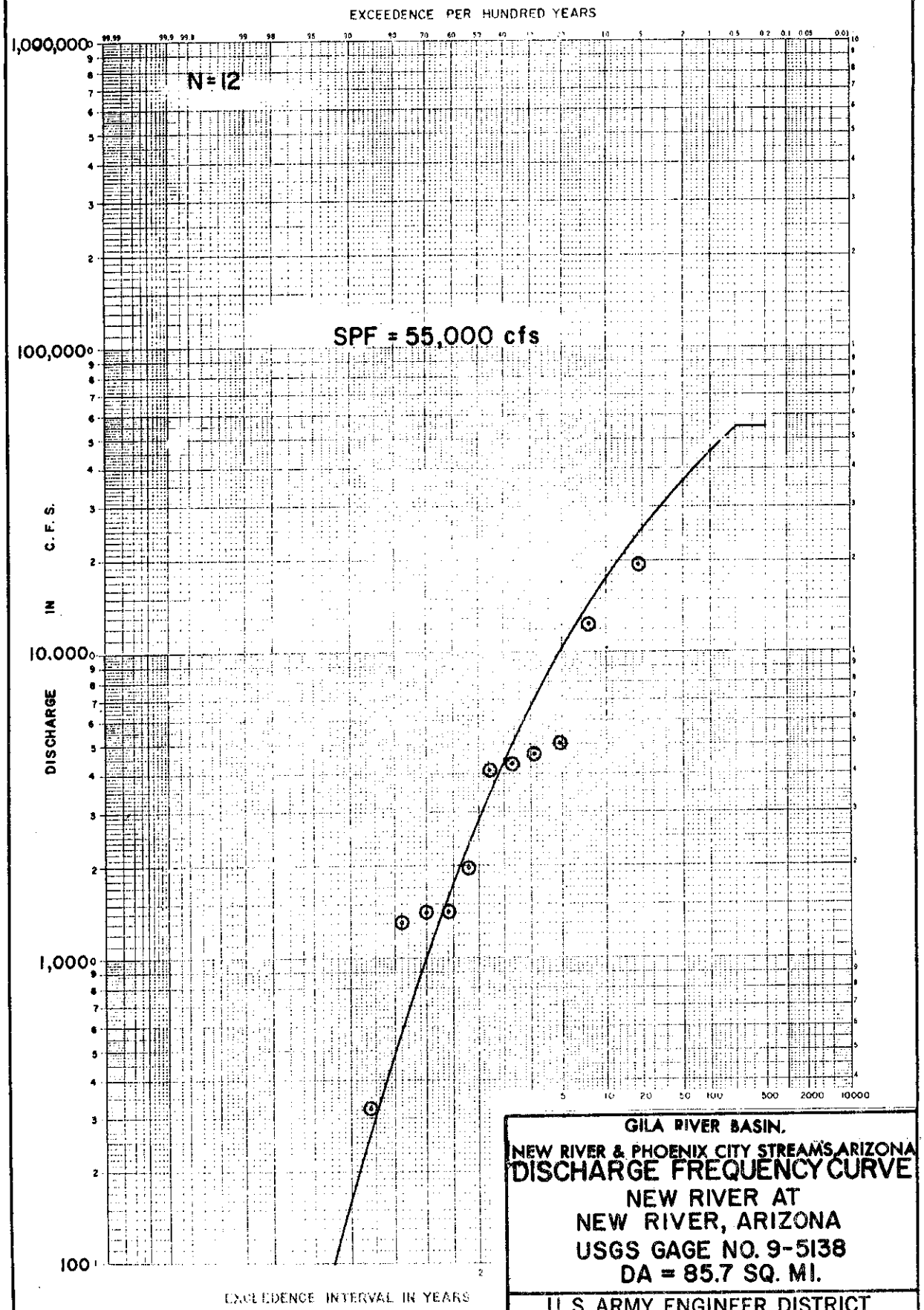
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE FREQUENCY CURVE
NEW RIVER NEAR
ROCK SPRINGS, ARIZONA
USGS GAGE NO. 9-5137.8
DA = 67.3 SQ. MI.

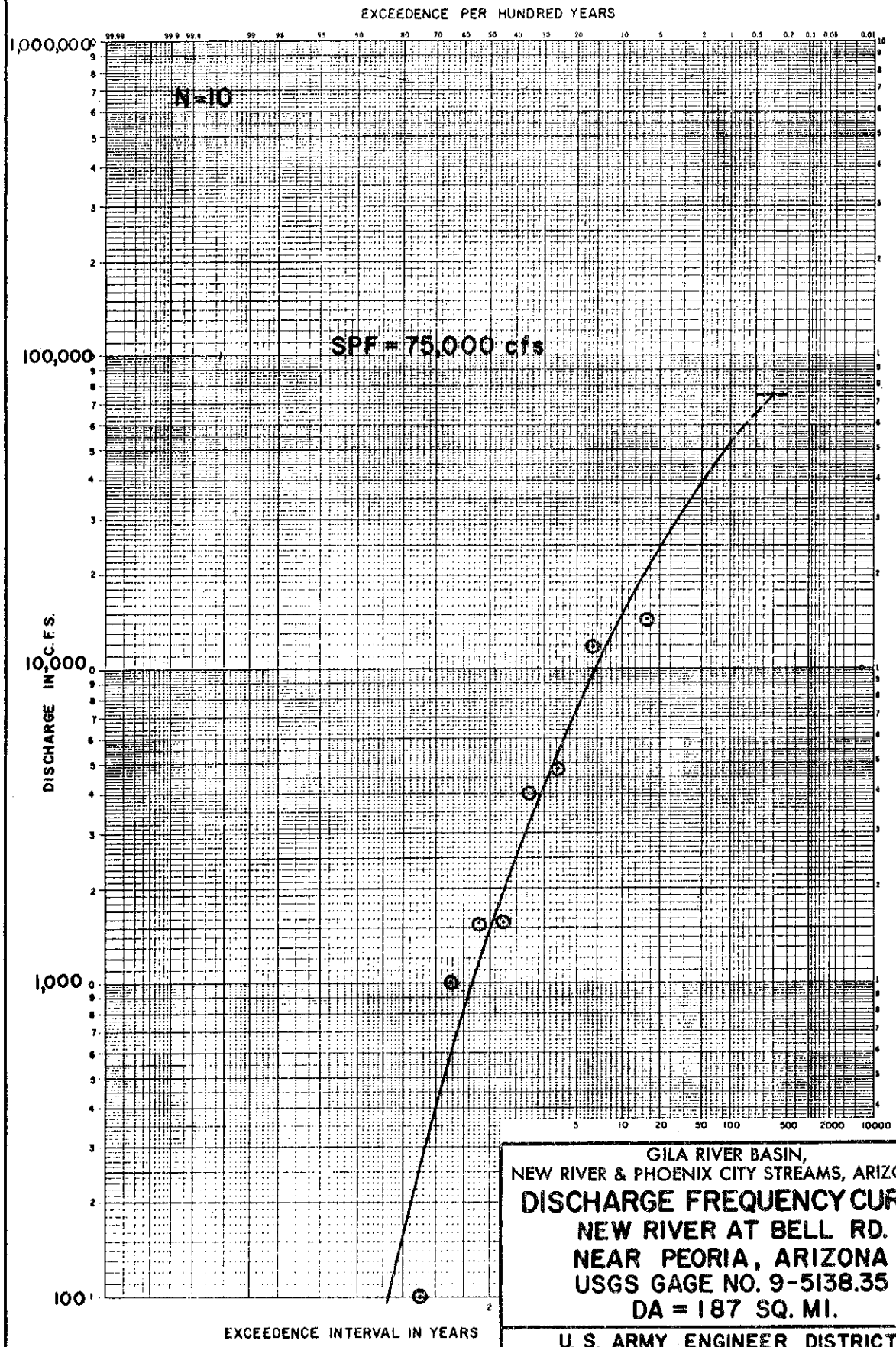
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 53



GILA RIVER BASIN.
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE FREQUENCY CURVE
NEW RIVER AT
NEW RIVER, ARIZONA
USGS GAGE NO. 9-5138
DA = 85.7 SQ. MI.

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

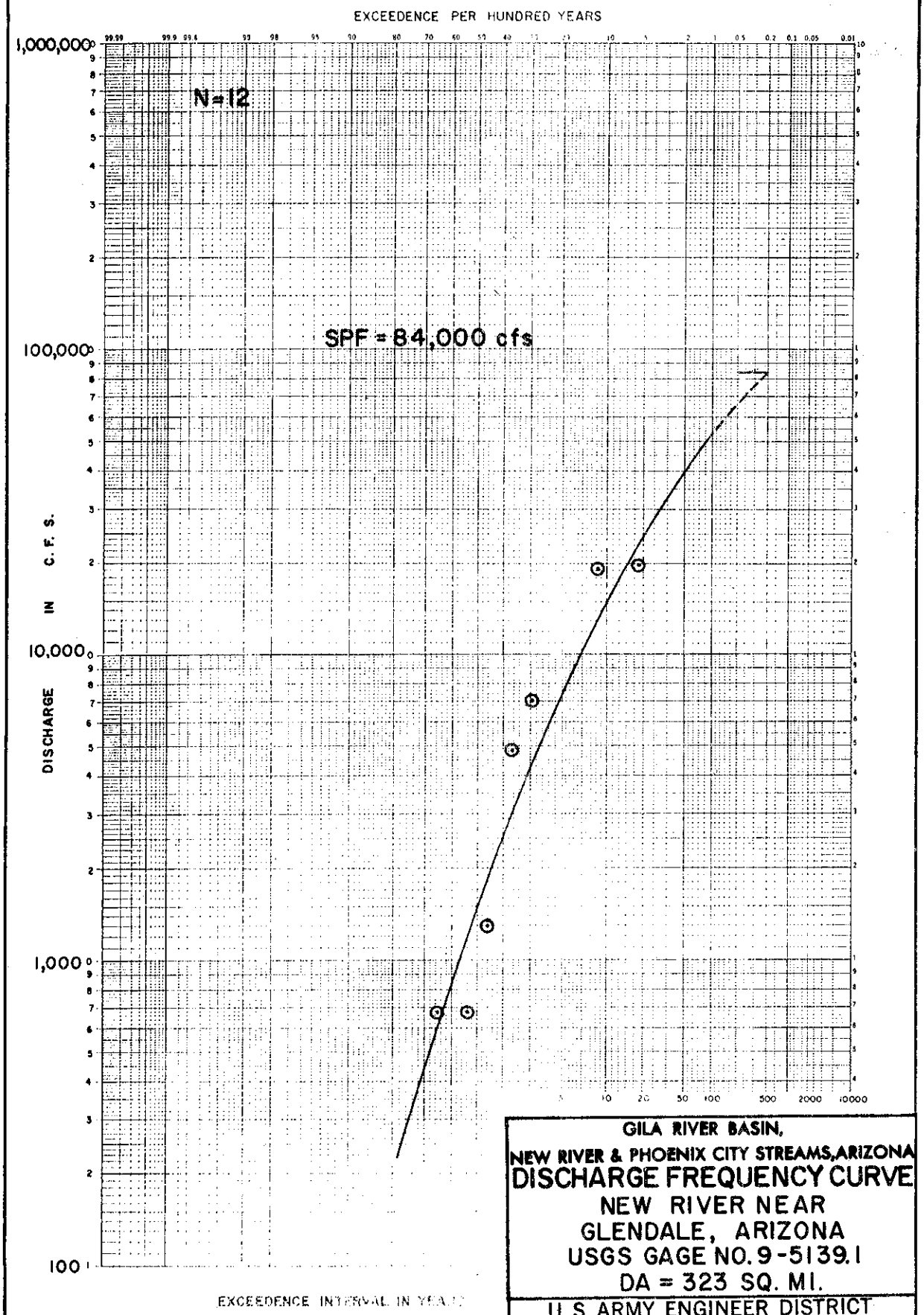


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

DISCHARGE FREQUENCY CURVE
NEW RIVER AT BELL RD.
NEAR PEORIA, ARIZONA
USGS GAGE NO. 9-5138.35
DA = 187 SQ. MI.

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

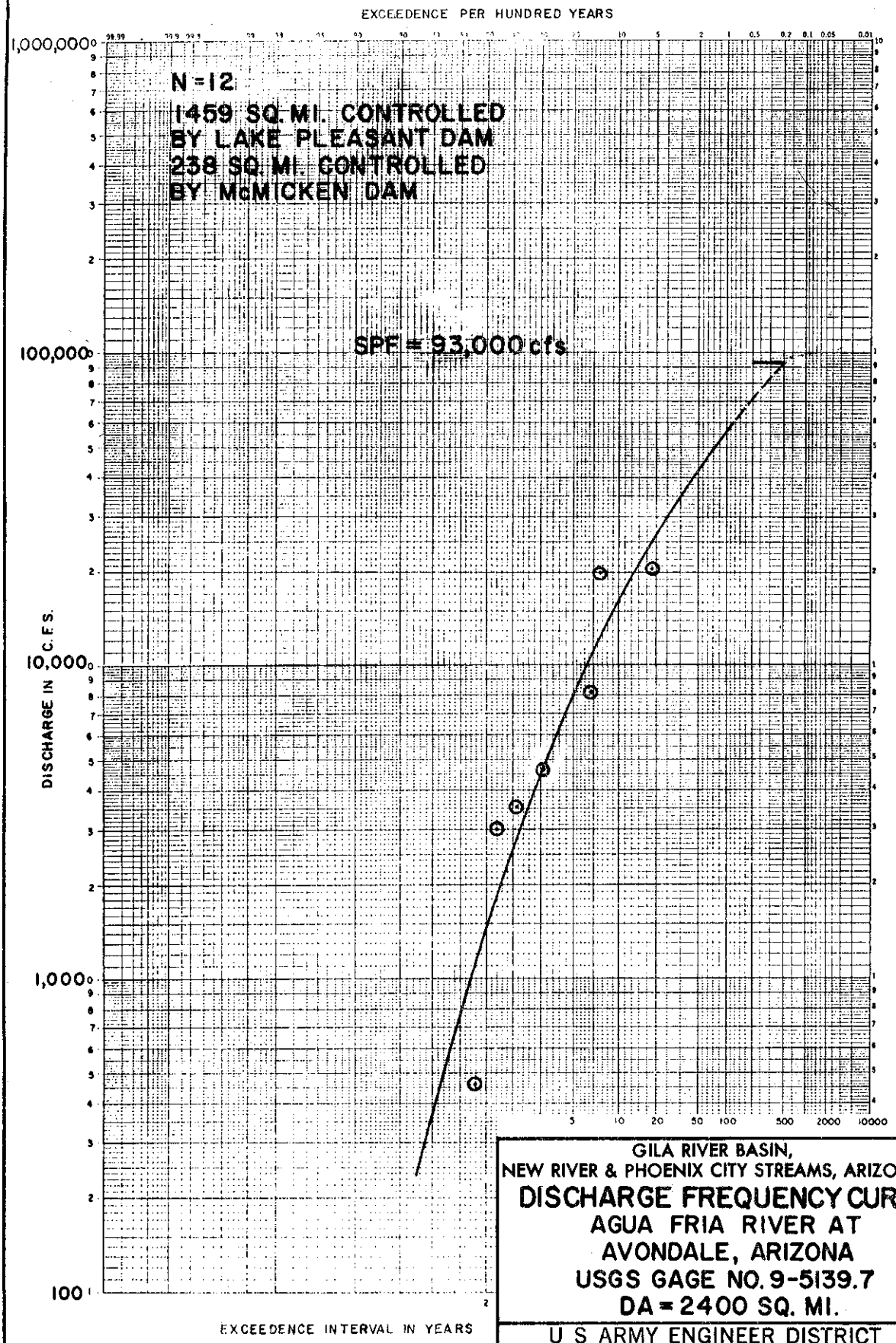
PLATE 55



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA
DISCHARGE FREQUENCY CURVE
NEW RIVER NEAR
GLENDALE, ARIZONA
USGS GAGE NO. 9-5139.1
DA = 323 SQ. MI.

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

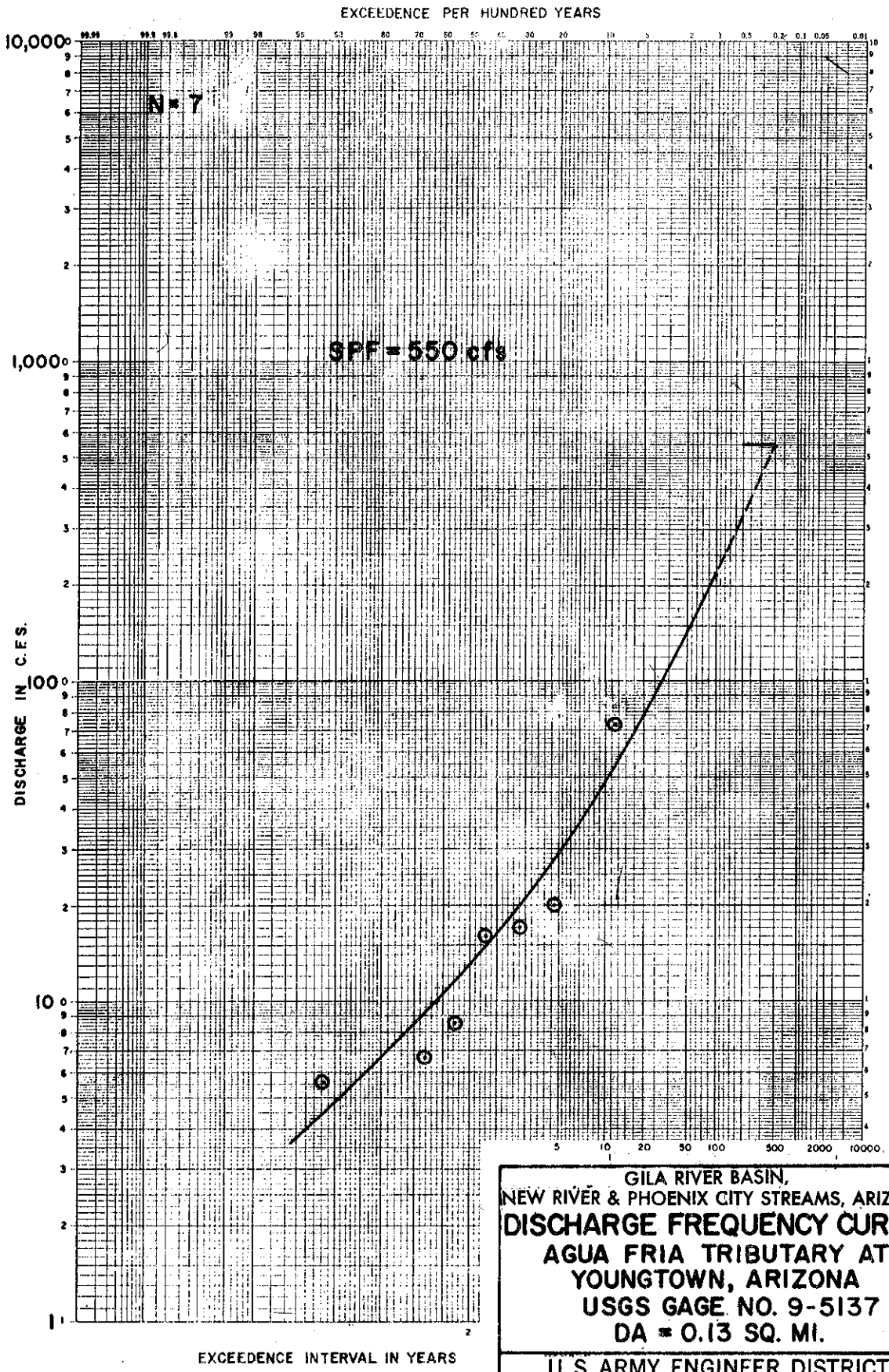
PLATE 56



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

DISCHARGE FREQUENCY CURVE
AGUA FRIA RIVER AT
AVONDALE, ARIZONA
USGS GAGE NO. 9-5139.7
DA = 2400 SQ. MI.

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

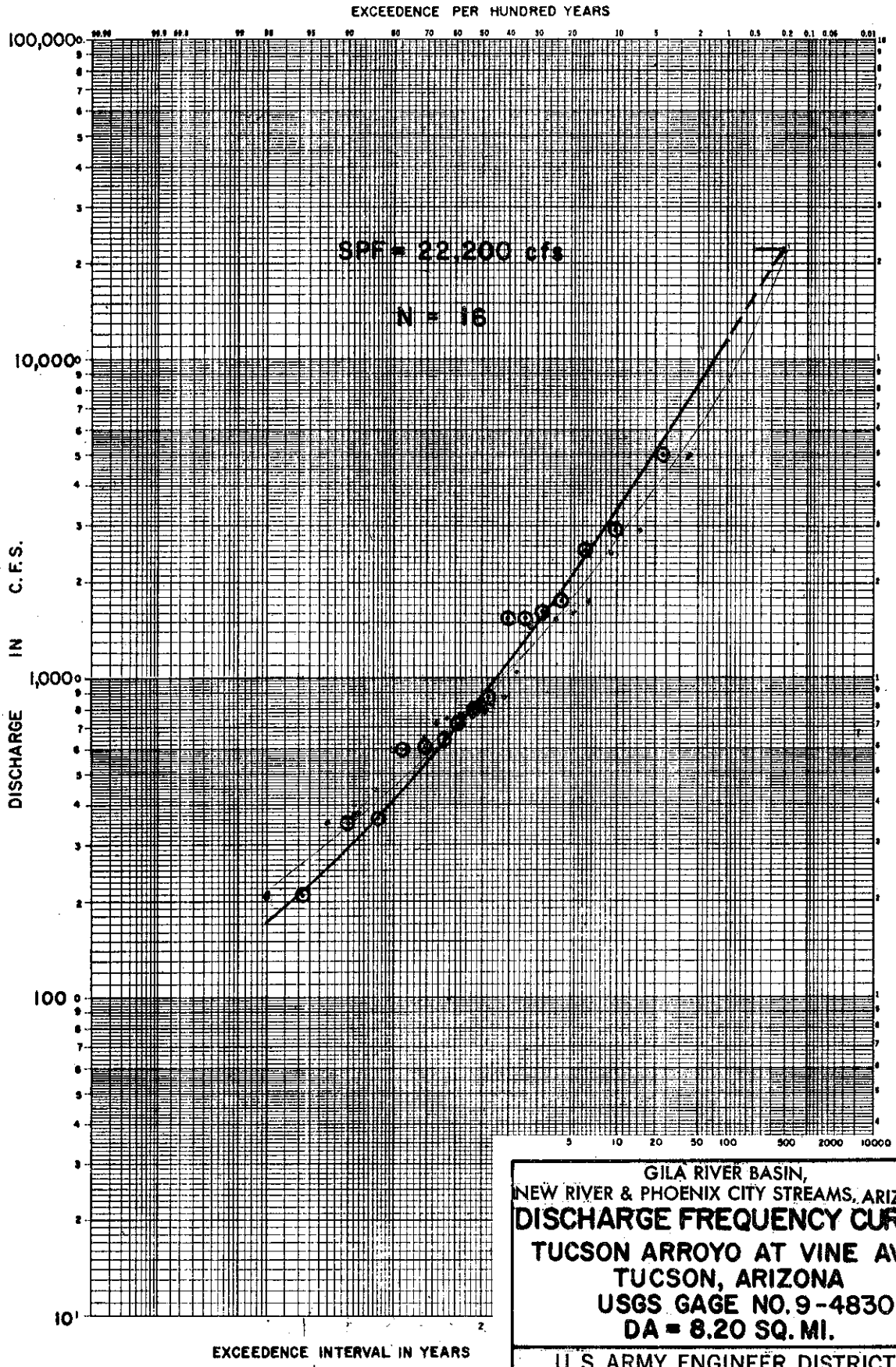


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

DISCHARGE FREQUENCY CURVE
AGUA FRIA TRIBUTARY AT YOUNGTOWN, ARIZONA
USGS GAGE NO. 9-5137
DA = 0.13 SQ. MI.

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 58



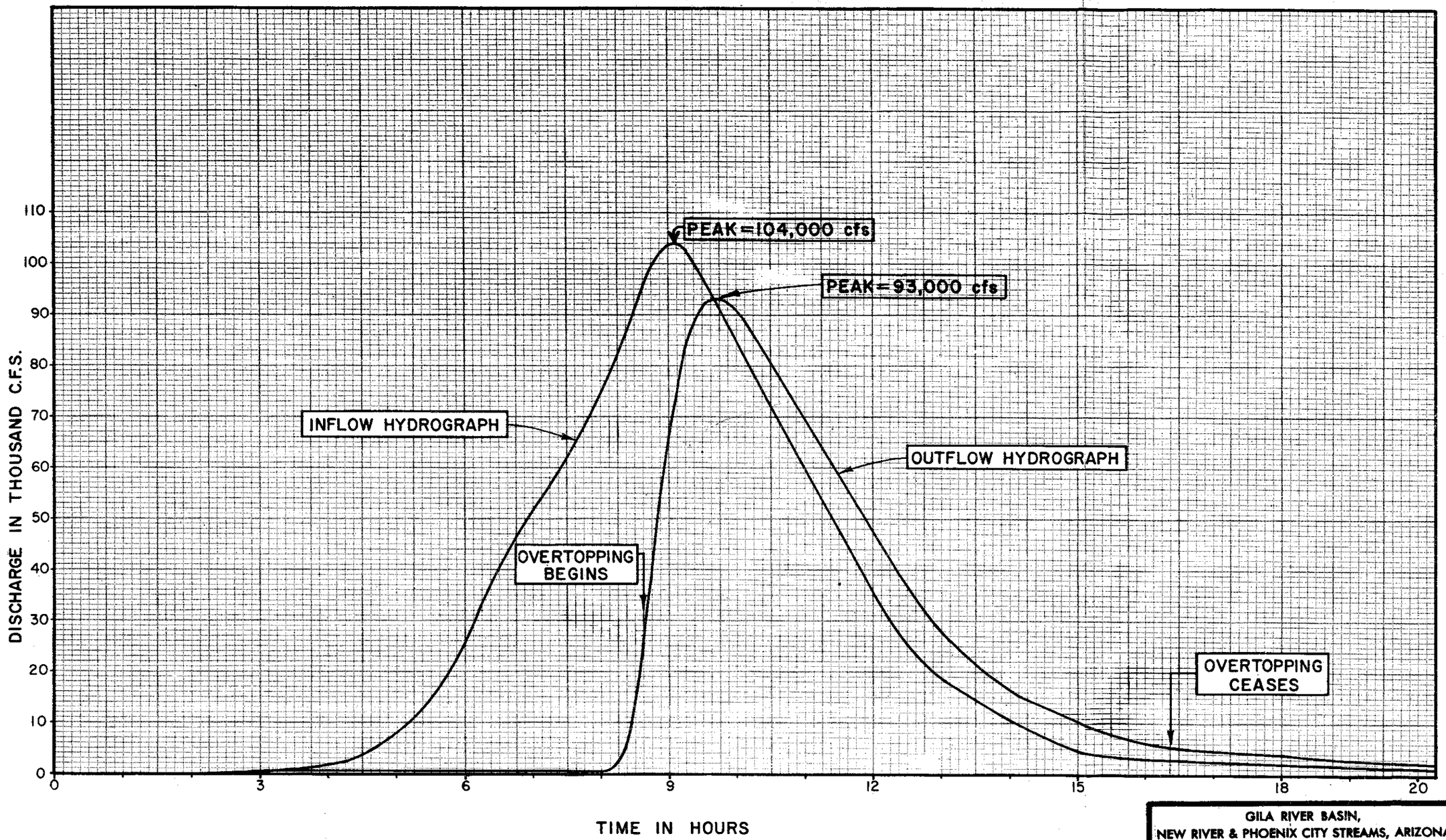
GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

DISCHARGE FREQUENCY CURVE

TUCSON ARROYO AT VINE AVE.
TUCSON, ARIZONA
USGS GAGE NO. 9-4830
DA = 8.20 SQ. MI.

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

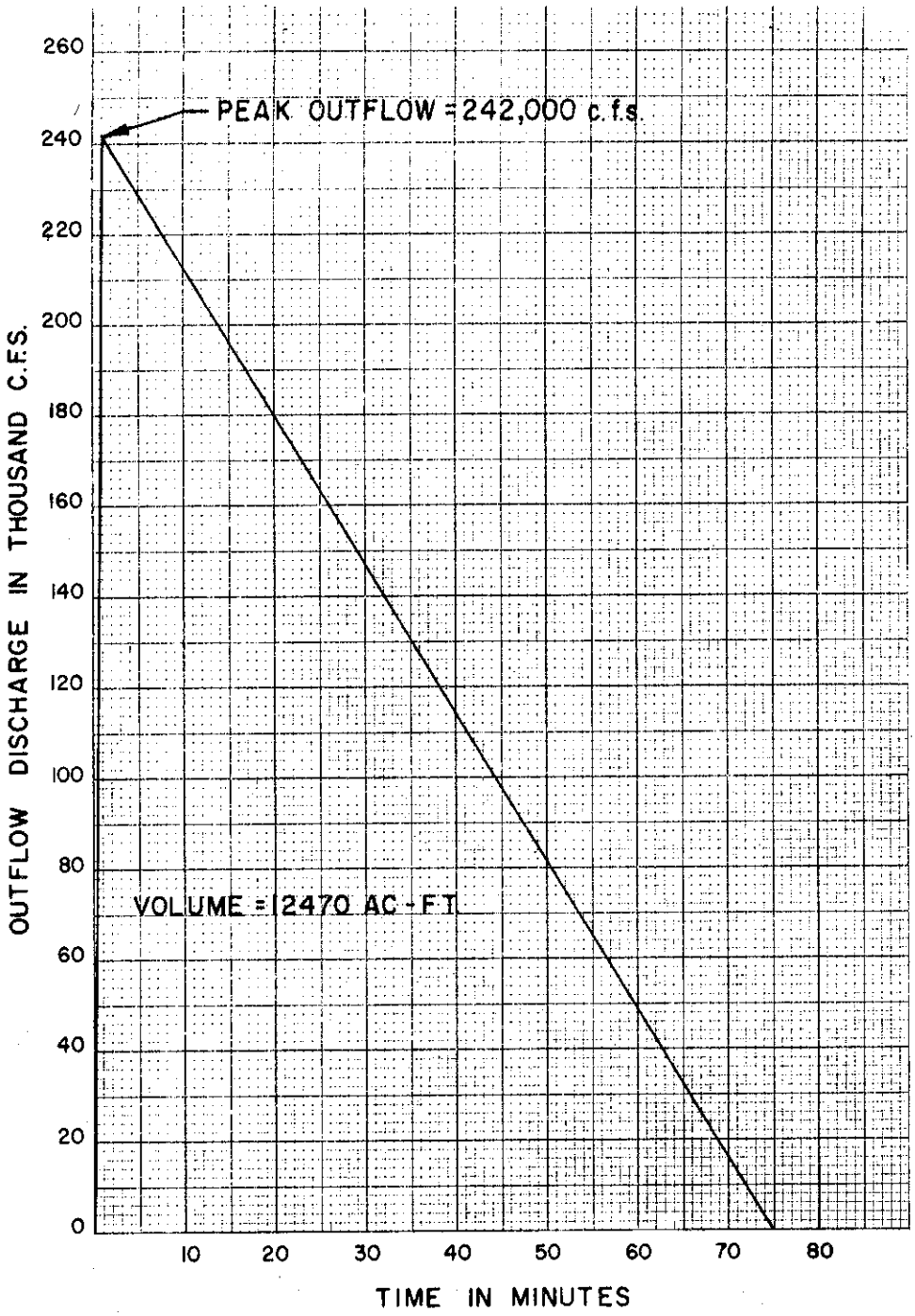
PLATE 59



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

500 YEAR FLOOD ROUTED THROUGH CAVE CREEK DAM

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

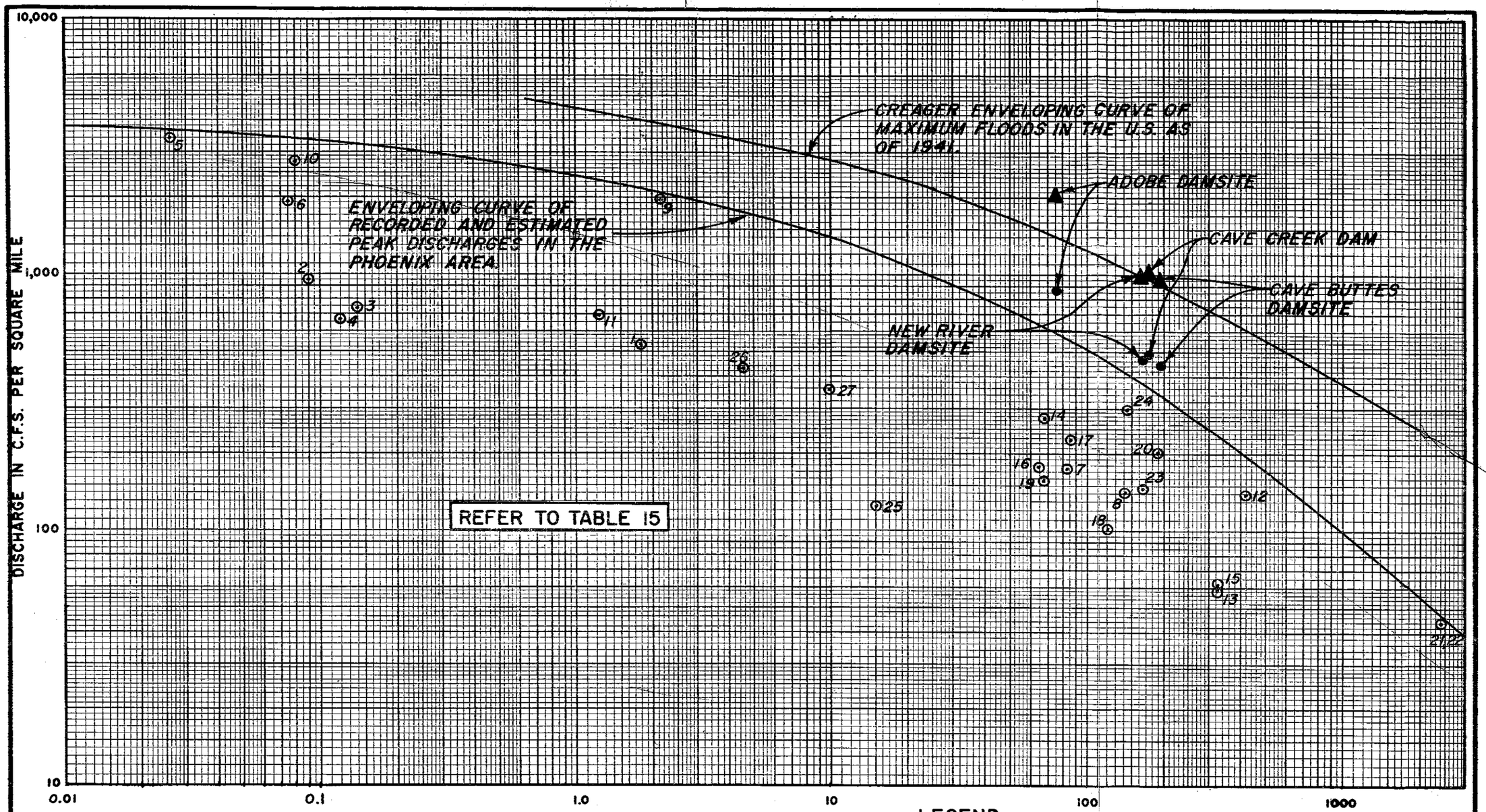


GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

CAVE CREEK DAM
FAILURE FLOOD HYDROGRAPH

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

PLATE 61



LEGEND

⊙1 PEAK DISCHARGE FOR MAJOR RECORDED FLOOD (SEE TABLE 15).

● STANDARD PROJECT PEAK DISCHARGE AT DAM OR DAMSITE.

▲ PROBABLE MAXIMUM PEAK DISCHARGE AT DAM OR DAMSITE.

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, ARIZONA

ENVELOPING CURVE OF PEAK DISCHARGES STREAMS IN THE PHOENIX AREA

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY DESIGN MEMO NO. 2

APPENDIX 1 - REPORTS RELATED TO WATER RESOURCES IN THE PHOENIX, ARIZONA AREA

1. Interim Report on Survey for Flood Control, Phoenix, Arizona and Vicinity (including New River), U.S. Army Engineer District, Los Angeles, Corps of Engineers, January 15, 1964.

2. Report on Survey for Flood Control, Indian Bend Wash, Arizona, U.S. Army Engineer District, Los Angeles, Corps of Engineers, April 15, 1962.

3. Interim Report on Survey for Flood Control Lower Agua Fria River and Vicinity, Gila River Basin, Arizona, U.S. Army Engineer District, Los Angeles, Corps of Engineers, December 10, 1952,

4. Gila River Basin, Phoenix, Arizona and Vicinity, phase B, Design Memorandum No. 1, Dreamy Draw Detention Basin, U.S. Army Engineer District, Los Angeles, Corps of Engineers, June 9, 1971.

5. Water Resources Report No. 44, Arizona State Land Department, Flood of September 1970 in Arizona, Utah, and Colorado, United States Geological Survey, April 1971.

6. Flood Damage Report, Storm and Flood of September 4-6, 1970, City of Phoenix, Arizona, February 1971.

7. Phoenix Storm Drainage Report, Yost and Gardner Engineers, Phoenix, Arizona, November 1, 1956.

8. Natural Disaster Survey Report, 70-2, United States Department of Commerce, National Oceanic and Atmospheric Administration, Arizona Floods of September 5 and 6, 1970, Rockville, Maryland, July 1971.

9. Storm Drainage Report from Maricopa Association of Governments, 1970, Yost and Gardner Engineers, Phoenix, Arizona.

10. Flood Damage Report on Storm and Flood of 16 and 17 August 1963. Glendale-Maryvale area near Phoenix, Arizona, U.S. Army Engineer District, Los Angeles, Corps of Engineers, June 1964.

11. Report on flood of 22 June 1972, Phoenix Metropolitan Area, Arizona, U.S. Army Engineer District, Los Angeles, Corps of Engineers, October 1972.

12. Flood Plain Information Studies for Maricopa County, Arizona.

U.S. Army Engineer District, Los Angeles Corps of Engineers.
Volume I - Indian Bend Wash Report, June 1964
Volume II - Cave Creek Report, November 1964
Volume III - Skunk Creek Report, March 1965
Volume IV - Wickenburg Report, December 1965
Volume V - New River Report, April 1967
Agua Fria River, March 1968
Hassayampa River, April 1972

13. Flood Hazard Information - Cave Creek from Arizona Canal to 19th Avenue, Phoenix, Arizona, U.S. Army Engineer District, Los Angeles, Corps of Engineers, October 1971.