3

GILA FLOODWAY HYDROLOGY
CORPS OF ENGRS.

GILA FLOODWAY
MARICOPA AND PINAL COUNTIES, ARIZONA

SURVEY REPORT

HYDROLOGY

Part I

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

January 1976

HYDROLOGY FOR SURVEY REPORT
GILA FLOODWAY
MARICOPA AND PINAL COUNTIES, ARIZONA

TABLE OF CONTENTS

TABLES

PLATES

PLATES (Continued)

APPENDICES

HYDROLOGY FOR SURVEY REPORT
GILA FLOODWAY
MARICOPA AND PINAL COUNTIES, ARIZONA

I. INTRODUCTION

1-01. PURPOSE AND SCOPE. This report presents Part 1 hydrology in support of survey studies of Gila Floodway, Maricopa and Pinal Counties, Arizona. The general location of the study region is shown on plate 1. Plates 3 through 8 show subbasin boundaries. Schematic flow diagrams are presented on plates 9 through 12. The hydrologic study began with four major objectives: (a) to define the basic meteorologic and hydrologic characteristics of the study region; (b) to develop methods and techniques with which to model the runoff process; (c) to compute standard project flood peak discharges and total storm volumes at selected locations for present and future basin development under pre-project conditions; and (d) to determine discharge frequency values at selected locations for present and future development under pre-project conditions. In the course of the study , it became apparent that man-made diversions and barriers to flow store runoff from portions of the basin and thereby reduce the effective drainage area tributary to the Gila Floodway. Hence, discharge values for floods more frequent than a 100-year flood were not determined for some locations shown in tables 1 and 2. Tables 1 and 2 give standard project flood and n-year flood peak discharge and total storm volume values. Throughout this report, the term "present conditions" refers to basin conditions existing in project year 1 (1985); likewise, "future conditions" pertains to project year 50 (2035).

1-02. PREVIOUS REPORTS. Recent Corps of Engineers reports containing hydrologic information pertinent to the study region are: (a) "Hydrology Report for Type 15 Flood Insurance Study, Mesa, Arizona," dated 30 April 1975 (reference 1) and subsequent revisions; and (b) "Gila River Basin, New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology, Part 1," dated October 1974 (reference 2). Appendix 1 lists these and other references with material of hydrologic importance for the study area.

II. GENERAL DESCRIPTION OF DRAINAGE AREA

2-01. PHYSIOGRAPHY AND TOPOGRAPHY.

a. The Gila Floodway drainage basin is located in southeast Maricopa County and northwest Pinal County, Arizona. All of the Pinal County portion of the study area is within the Gila River Indian Reservation. The drainage area, approximately 1,000 square miles in size, extends from the Usery, Goldfield, and Superstition Mountains on the east to the South Mountains on the west, and from the Salt River drainage boundary on the north to the Santan Mountains and the Gila River drainage boundary on the south. Approximately 20 percent of the basin is mountainous. The mountain areas are characterized by fairly rugged terrain and steep gradients. The remaining 80 percent of the area consists of very flat valley land with alluvial slopes at the base of the mountains. Gradients in the valley are typically 10-20 feet per mile.

b. There are no large streams in the study area with the exception of Queen Creek, which is controlled by Whitlow Ranch Dam. Flash floods from the small creeks and rivulets originating in the mountainous areas do, however, cause serious damage to the valuable agricultural land on the valley floor, but by about 1981, approximately 750 square miles of the 1,000 square miles total area is expected to be controlled by authorized Soil Conservation Service (SCS) projects. These projects are designed to control all runoff from the area above them up to and including a 100-year flood (see references 3, 4, 5, and 6).

2-02. AVAILABLE MAPPING. The best general contour coverage of the study area available for this study was the 1:24,000 scale series of topographic maps published by the U.S. Geological Survey. These maps have contour intervals of 10-20 feet. In the vicinity of the City of Mesa, however, more recent, detailed contour information, supplemented with extensive new field information, was assembled for a stormwater drainage study of Mesa which is described in reference 7. This new information was also utilized in this study. An additional survey was performed to better define the relative difference in elevation between certain man-made barriers and the surrounding ground. The extent of the survey is shown on plate 13. Other available physiographic mapping includes USGS 1:24,000 orthophoto quads of most of the study area published in 1971, aerial photographs of the City of Mesa and vicinity dated October 1973, and SCS soil survey maps (references 8 and 9).

2-03. GEOLOGY AND SOILS.

a. The geology of the study area ranges from recent alluvium of the valley floors to Precambrian igneous and metamorphic rocks of the mountains. The present topography reflects the results of extensive mountain building and erosion. No activity, either volcanic or seismic, has been recorded in the written history of the area.

b. Soils in the basin are strongly influenced by the parent material and the attitude of the land. The mountains and lower buttes form shallow soils that are mostly gravel and coarse sand with very little clay. In a few flat areas, a small subsoil horizon can form and will support vegetation. Extending from the mountain fronts to the lower valley, the old alluvial fans have deep soils consisting of clay loams and sandy clay loams, often 4-5 feet deep. Soil on the lower, newer alluvial fans and floodplains are the result of erosion of the older fans and the rocks of the mountains. They tend to be very deep, 5 feet or greater, and consist of loams, fine sandy loams, and clay loams. Almost all of the soils in the study area are moderately alkaline and strongly calcareous, tending to form large areas of impervious caliche below the surface.

2-04. VEGETATION. Natural vegetation is sparse at best. Cacti grow throughout the area along with other desert shrubs. Native trees such as juniper, paloverde, mesquite, ironwood, and scrub oak are scattered among the shrubs. The vegetation tends to be thicker along and adjacent to streams and irrigation canals. Perennial grasses form a very small portion of the natural vegetation, but annual grasses occur after winter rains. Cultivated crops include alfalfa, barley, cotton, sugar beets, potatoes, lettuce, sorghum, small feed grains, and citrus and deciduous fruits.

2-05. LAND USE.

a. Urbanization Projections. Much of the land in the study area is now devoted to agriculture or is still in its natural state. Urbanization is taking place rapidly, however, and by 2035, a large portion of the basin is expected to have been developed. Plate 14 shows the extent of projected urban development for present (1985) and future (2035) conditions in the study area. The projections were based on Office of Business Economics-Economics Research Service (OBERS) population projections; the Composite Land Use Plan for Maricopa County, Arizona, prepared by the Maricopa Association of Governments (MAG); and the City of Mesa 1990 General Land Use Plan. This information was supplemented by field surveys and analysis of aerial photographs and orthophoto quads. Projections were made in conformance with the National Flood Disaster Act of 1973. Projected city boundaries for the Cities of Mesa, Chandler, and Gilbert were obtained from the appropriate city agencies. It was assumed that no new extensive agricultural development would take place east of the proposed Central Arizona Project (CAP) canal.

b. Effective Impervious Cover.

(1) Effective impervious cover estimates used in this study are given in table 3. These percentages were based on the values given by Yost and Gardner Engineers for Mesa's storm drain study (reference 7). Since the values given for residential land use seemed low, an analysis of the percent impervious cover for two typical residential areas was performed, using 1973 aerial photographs having a scale of 1 inch equals 100 feet. The following items were determined:

(a) Area 1 - Tract bounded by Broadway Road, Gilbert Road, and Consolidated Canal.

1. Lot size is approximately 70 to 80 feet wide by 110 to 120 feet long equals 7,700 to 9,600 square feet.

2. There are about 5 units per acre.

3. Roof surface is approximately 2,000 square feet. (However, only a small portion is hydraulically connected to the street. Yost and Gardner, after field investigation, have estimated 20 percent effectiveness for roof surface contribution).

4. Driveways average about 500 square feet of pavement.

5. Streets average 10 to 15 feet to centerline by 70 to 80 feet long equals 700 to 1,200 square feet in front of each lot.

6. Total impervious area including streets (assuming 20 percent of roof surface effective) equals 1,600 to 2,100 square feet equals 19 percent impervious cover.

(b) Area 2 - Tract between Stapley Drive and Mesa Drive just north of Main Street.

1. Lot size is approximately 65 feet wide by 140 feet long equals 9,100 square feet.

2. There are approximately 3-1/2 units per acre.

3. Roof surface is approximately 1,600 square feet.

4. Driveways average about 500 square feet.

5. Total impervious area including streets (assuming 20 percent roof surface effective) equals 1,500 to 1,800 square feet or 15 to 18 percent impervious cover.

(2) An effort to confirm the measurements taken from the aerial photographs was made by contacting officials of the City of Mesa. The Assistant City Engineer* estimated that (a) the typical residential lot size is about 9,000 square feet; (b) the roof area of the average house is about 1,800 square feet; and (c) there are about 3 units per acre, although many lots are 1/2 acre in size.

(3) In view of the foregoing, the assumption of 25 percent effective impervious cover for typical residential areas seems reasonable. Higher density residential use areas such as apartment complexes and condominiums, a relatively small portion of the city, would have a higher degree of impervious cover as reflected in the land use table. However, the same concepts of hydraulic connectivity apply, tending to make the values lower than might be expected.

*Telephone conversation on 17 June 1975 with Assistant City Engineer Pete Peterson.

2-06. RUNOFF CHARACTERISTICS. None of the natural watercourses in the study area flow perennially. Generally runoff occurs only during and immediately following heavy precipitation because climatic and drainage area characteristics are not conducive to continuous flow. Significant runoff occurs mostly in the summer months (June through September) as a result of local storms and to a lesser degree general summer storms. Stream channels are fairly well defined in the mountain areas, but upon reaching the valley transition, lose definition, and flow proceeds downslope as sheetflow. The shallow depths encountered with sheetflow allow seemingly insignificant obstructions to radically alter the path of flow. Factors altering flow patterns are discussed in the following paragraphs.

2-07. FACTORS AFFECTING RUNOFF.

a. Whitlow Ranch Dam. The Whitlow Ranch Dam, an earthfill structure built by the Corps of Engineers in 1960, controls 143 square miles of Queen Creek. The reservoir was designed for SPF and has a gross capacity at spillway crest of about 36,000 acre-feet (reference 10).

b. SCS Flood Control Structures. Fourteen flood retarding structures, floodways, and diversions which have been built by the SCS or are authorized for construction are shown on plate 1. These structures are designed to control runoff from floods up to and including a 100-year flood (references 3, 4, 5, and 6). The design adequacy of the structures was found to be satisfactory, based on the computational procedures described in subsequent paragraphs. Thus, for floods with a return period of

Page 8?

100 years or less, the easternmost drainage boundary becomes the Roosevelt Water Conservation District (RWCD) Floodway, and the area potentially contributing runoff to the Gila Floodway is reduced from approximately 1,000 square miles to about 250 square miles. Routing SPF through the retarding structures showed that although emergency spillway flow would occur during a standard project flood, the structures would not be overtopped. The RWCD Floodway and the floodways connecting the retarding structures would be overtopped, however, but the system as a whole would serve to significantly lessen the downslope flood hazard.

c. Superstition Freeway. A significant factor affecting the drainage pattern in the Mesa area is the proposed Superstition Freeway, with its attendant drainage facilities. Traversing the study area from east to west, the freeway will divert flow from its normal north-east to south-west flow path, forcing flood waters to travel due west. The freeway drainage system, designed for a "50-year" flood according to Arizona Highway Department criteria (reference 11), consists of cross-drainage structures east of RWCD Floodway and a system of detention basins, to be constructed as part of the City of Mesa storm drain system, and connecting channel west of RWCD Floodway. Plate 15 ? shows the presently conceived drainage system west of RWCD Floodway.

d. Irrigation Canals.

(1) The need for water for agricultural use has given rise to numerous irrigation canals which criss-cross the study area, altering the normal drainage pattern. Two of the major canals west of the RWCD Floodway,

Eastern and Consolidated Canals, flow north to south through the City of Mesa and beyond on very shallow slopes cutting the drainage pattern with obstructions in the form of canal levees. The levees are normally 2-3 feet high, but in some places approach ground level. Runoff in the form of sheetflow is directed southward by the levees until the conveyance capacity is exceeded or another barrier such as a cross road is encountered. Roads crossing the canals are sometimes elevated above the surrounding ground for some distance, forming ponding areas at the intersection of the road and canal. Culverts through the roads are occasionally found, but they are usually small and plugging by debris and sediment is common. The canal levees are made from loose soil dredged from the canals at periodic intervals and are subject to failure when overtopped. When runoff volume exceeds the pond capacity, the levee breeches and flow proceeds downslope until the next barrier is encountered. Pond capacity is expected to be decreased in future years because of an FHA requirement that first floor level be above the 100-year flood level. Developers are complying by filling the area adjacent to the canals on the upstream side to approximately the elevation of the canal bank.* This was considered where future development was projected (see plate 14).

(2) Western Canal, between Canal Drive and the Chandler branch of the Southern Pacific Railroad, is another obstruction to flow attempting to reach the Gila Floodway. Except in the immediate vicinity of the railroad, the canal is 2-5 feet higher than the surrounding ground.

*Letter dated 15 December 1975 from Assistant City Engineer Pete Peterson.

Floodwaters north of Western Canal and west of the railroad would be diverted westward until meeting the Tempe Canal Channel. However, developers are filling the area between the railroad and Tempe Canal to approximately the elevation of the canal banks. When runoff volume is sufficient, flow will most likely overtop Western Canal near its junction with Tempe Canal, causing levee embankment failures. Floodwaters would then proceed in a southerly direction toward the Gila Floodway.

(3) Tempe Canal, just north of the Superstition Freeway would also obstruct the flow path of floodwaters. If flow rates exceed the capacity of the Tempe Canal Channel, ponding would take place. Sufficiently large volumes of runoff, as would occur during SPF, would overtop the canal, causing it to fail, and flow would proceed in the direction of the City of Tempe. ? west, south or both

e. Railroads. Three branches of the Southern Pacific Railroad run through the study area, effectively diverting or obstructing flood flows (see plate 1). The railroad bed is generally 3-6 feet higher than the surrounding ground. Such cross-drainage structures as exist are small and ofter plugged with dirt and debris. Where two of the branches converge near Mesa, the potential ponding capacity is approximately 1,000 acre-feet.

f. Streets and Highways. Numerous high crowned or elevated streets and highways provide artificial drainage boundaries. In addition to the Superstition Freeway, portions of Apache Trail (Main Street), Southern Avenue, and Interstate 10, to name a few, serve to concentrate runoff from its normal path. At intersections of roads with other barriers such as railroads or canals, small ponding pockets are effective in retarding flow and decreasing total runoff volume.

g. On-Site Storage Policy.

(1) Since September 1972, the City of Mesa has had a subdivision ordinance which gave the City Engineer authority to establish on-site storage requirements for storm runoff. The policy applies to all new development and is enforced through the building permit granting process. The policy states that all precipitation from a 50-year 24-hour storm (approximately 3 inches) which falls within a subdivision being developed must be retained within the subdivision boundaries. The method of retention is left to the developer. Two common methods are to depress individual lots to retain the rain falling on them, providing an additional storage area for street runoff, and to provide a large retention area designed to retain runoff from the entire subdivision.

(2) The City of Chandler adopted an on-site storage ordinance in late 1975 which applies to all new development and is enforced by inspection by city forces. The ordinance requires that the 50-year 24-hour precipitation be retained on site.

(3) In August or September 1975, the Town of Gilbert began enforcing an on-site storage requirement as part of its new subdivision ordinance. On-site retention of the 50-year 24-hour rainfall is required. Compliance will be enforced through the development plans approval process.

(4) In September 1975, the County of Maricopa adopted amendments to its subdivision regulations which require on-site storage for all unincorporated areas of the County. The amendments call for sufficient on-site

storage such that the peak discharge, computed from 100-year 2-hour rainfall, leaving the developed subdivision does not exceed the predevelopment peak discharge, determined from the same storm. The method of on-site storage is not specified in the ordinance, but, according to Maricopa County Flood Control District personnel, up to 50 percent of the future developed area may possibly use a retention type storage. The remaining development would use a detention type storage. The two types of storage and their effects on a hydrograph are discussed in paragraph 6-02.

h. Land Treatment. With much of the study area devoted to farming, land treatment becomes an important factor affecting runoff. Where crops are being grown, it is standard procedure in this region to grade the fields for optimal irrigation water use before planting. The fields are graded nearly flat, and tailwater berms are often built. The SCS has estimated that this type of treatment effectively stores an average of 2 inches of the rain falling on the field before runoff occurs. In addition, where citrus groves are located, berms inclose the groves to facilitate "flood" irrigation, permitting little or no runoff. Normally, shallow sheetflow is directed around these structures, altering an otherwise direct flow path.

2-08. CLIMATOLOGY.

a. The climate of the Gila Floodway basin ranges from warm and arid over the desert to relatively ? cool and moderately ? humid in the higher mountain portions of the basin. Mean maximum/minimum daily temperatures range from approximately 65/35 degrees Fahrenheit in January to about 105/75 degrees Fahrenheit in July over the lower valleys, and from around 50/25 in January to about 90/60 in July over the higher mountain peaks. At Mesa Experiment Farm, the National Weather Service normal temperatures range from a daily mean of 50.3 degrees in January to 89.2 degrees in July. The extreme temperatures experienced in the region range from 120 degrees in portions of the lower desert to near zero in some of the higher mountain canyons. Prevailing winds are generally rather light, but winds can become moderate during the winter and spring, especially during stormy periods. Summer thunderstorms often produce strong gusty winds over local areas.

b. Mean annual precipitation ranges from about 7 inches in the western portion of the basin to nearly 20 inches in the mountainous extreme eastern portion (see plate 2). The heaviest rainfall of the year normally occurs during the summer months of June through September; much of the remainder falls during the period December through March. Some snow falls in the higher elevations during the winter months. There is considerable month-to-month and year-to-year variability in precipitation in the basin.

c. Three basic types of storms affect the Gila Floodway basin, although some individual storms may consist of a combination of types: general winter storms, general summer storms, and thunderstorms. Reference 2 describes each type in detail, and gives several examples of each.

III. STORMS AND FLOODS OF RECORD

3-01. GENERAL. Historical accounts indicate that many large floods have occurred in the general Gila River basin. Apparently, there were important general floods in 1833, 1862, 1869, and 1880, although magnitudes of the flood events cannot be ascertained. Sizeable floods were produced by the general storms of February-March 1884, February 1891, January 1916, and

February-March 1938, but specific information for the Gila Floodway basin is not available. General winter storms can cause flooding in the study area, but the most severe floods generally occur during the summer months as a result of localized thunderstorms, often embedded in general summer storms. Severe local storms and floods occurred in and around the Gila Floodway basin in 1921, 1926, 1929, 1930, 1933, 1935, 1936, 1939, 1943, 1946, 1951, 1953, 1954, 1955, 1957, 1959, 1963, 1964, 1966, 1967, 1969, 1970, 1971, and 1972. Storm and flood information specific to the study area is sketchy, especially for the earlier events. It is known that the Town of Gilbert was hit by floods in 1926, 1930, and 1933, with others following in the 1930's and 1940's. More recent events for which information is available are briefly described in the following paragraphs and in table 4.

3-02. STORM AND FLOOD OF 19 AUGUST 1954.

a. The storm and flood of 19 August 1954 is the most severe occurrence of record within the Queen Creek drainage area, located in the southeast corner of the Gila Floodway basin. Precipitation in the Superstition and Pinal Mountain areas occurred between 0100 and about 1000 hours. Rainfall intensities were very high during portions of the storm, especially between 0500 and 0900 hours. Boyce Thompson Southwestern Arboretum, about 4 miles west of Superior, recorded the highest measured precipitation amount, 5.3 inches, most of which fell within 3 hours, although greater amounts are believed to have fallen in the mountains to the south. An estimated 140 square miles of area received over 5 inches of precipitation. Another storm, more local in character and with lower rainfall intensities, occurred in the vicinity of Apache Junction, just east of Mesa, between 0200 and 0500 hours on 20 August.

b. The extent of the flooded area can be seen on plate 16. Within the outline of the flooded area, there were places, such as Williams Air Force Base, which were not completely inundated. To the north and east of the flooded region shown on plate 16, flood waters traveled across the desert as sheetflow, with occasional islands of land not covered with water. To the south and east of the flooded area shown on the plate, severe damage occurred to the farmland along Queen Creek. Runoff from the Superstition Mountain area flooded agricultural land adjacent to and east of RWCD Canal, overtopped and breeched the canal in many places, and flooded farmland from the RWCD Canal all the way to Gilbert, inundating much of the town itself. For a more detailed description of the storm and flood of 19 August 1954, see references 2 and 12.

3-03. STORM AND FLOOD OF 31 OCTOBER 1957. Precipitation began early on the morning of 31 October. Rainfall continued with moderate to high intensities for a period of 2-3 hours. In the Queen Creek-Superstition Mountain area, recorded rainfall amounts varied from 0.95 inch at Williams Air Force Base to 1.71 inches at Superior, but greater amounts were indicated in the Superstition Mountains. Available peak discharge estimates are as follows: (1) Queen Creek at Whitlow Ranch Reservoir Site - 2,000 cfs; (2) Queen Creek 500 feet upstream from SPRR bridge - 1,000 cfs; (3) Queen Creek half mile west of Powers Road - 800 cfs; (4) Weekes Wash at U.S. Highway 60-70-80-89 - 1,100 cfs; (5) Siphon Draw at U.S. Highway 60-70-80-89 - 1,500 cfs; and (6) Williams Air Force Base channel at bridge at west end of base - 1,300 cfs. For more details, see reference 13.

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.

4-02. STANDARD PROJECT STORM. The 19 August 1954 thunderstorm that was centered generally in the Queen Creek drainage area was determined to be the most severe flood producing rainfall depth-area-duration relationship and isohyetal pattern that may reasonably be expected to occur over the study area. A detailed description of the storm, along with total rainfall amounts, intensity-duration relationships, and precipitation patterns, is given in reference 2.

4-03. RAINFALL-RUNOFF RELATIONSHIPS.

a. Runoff Models.

(1) Mountain areas. Reference 2 discusses rainfall-runoff relationships developed from reconstitutions of floods which have occurred in several mountain basins around Phoenix. These relationships were considered applicable to the mountainous portion of the study area.

(2) Valley areas.

(a) The valley watersheds in the Gila Floodway basin have very flat slopes. Urbanization together with agricultural development has obliterated most original watercourses, and runoff occurs basically as sheetflow. Flat valley areas are not conducive to good runoff measurement; hence, sufficient data with which to derive precipitation-runoff relationships from past runoff events does not exist. A theoretical procedure was needed which allowed computation of flood hydrographs using parameters that could be determined from topographic maps or had generally accepted values as opposed to empirically determined coefficients. A review of the literature revealed that several studies dealing with sheetflow have been conducted, and various models for determining hydrographs have been derived. For this study, some models were rejected due to the difficulty of accurately estimating rather sensitive input variables; purely graphical methods were not used because the large number of hydrograph computations necessitated computerization of the methodology.

(b) In this study, a linear storage system was used to determine the time distribution of runoff from an effective rainfall hyetograph. Although nonlinearity of the rainfall-runoff process has long been recognized, the sparsity of data makes the use of a simple procedure the most reasonable approach. The linear storage system is analogous to a reservoir in which storage is related to overflow by the equation

$$S = KO \tag{1}$$

in which K is a proportionality factor and is a constant value for a true linear storage system. Basin storage S at any time is equal to a summation of rainfall excess minus the volume of outflow up to that point.

When combined with the general storage equation

$$I - O = \frac{dS}{dt}, \tag{2}$$

equation (1) can be expressed as

$$O_2 = C_1 (I_1 + I_2) + C_2 O_1 \tag{3}$$

where

$$C_1 = \frac{\Delta t}{2K + \Delta t} \tag{4}$$

and

$$C_2 = \frac{2K - \Delta t}{2K + \Delta t} \tag{5}$$

The following paragraphs address two important questions: (1) How can the coefficient K be determined from the physical characteristics of a watershed and storm characteristics of the rainfall hyetograph? and (2) Does a linear system adequately model the sheetflow runoff process?

(c) In studies of urban watersheds, Willeke (reference 14) utilized the linear storage system concept and found that the coefficient K could be approximated by what he called "lag time" (time between centers of mass of effective precipitation and runoff). Further review of the literature did not reveal a relationship between lag time as defined by Willeke and measurable physical parameters; however, relationships between sheetflow time of concentration and measurable physical characteristics did exist. Time of concentration is defined as the time from commencement of rainfall excess until flow from the uppermost edge of the basin arrives at the overflow point (analogous to time to equilibrium). Henderson and Wooding (reference 15) have shown that for a constant effective rainfall intensity, time of concentration, tc, using kinematic wave theory, can be expressed as

$$tc = \left(\frac{Li^{1-m}}{\alpha'} \right)^{1/m} \tag{6}$$

where
- L = length
- α' = coefficient
- i = effective rainfall intensity
- m = exponent.

Ragan and Duru (reference 16) have demonstrated coefficient values for an altered form of equation (6) as follows:

$$tc = 0.93 \frac{(Ln)^{0.6}}{i^{0.4} s^{0.3}} \tag{7}$$

where
- tc = time of concentration in minutes
- L = length in feet
- i = effective rainfall intensity in inches per hour
- n = Manning's roughness coefficient
- s = slope in feet per foot.

The hydrograph resulting from a constant effective rainfall intensity i of a duration t = tc is shown on plate 17. If the time to peak is used to approximate the time to the center of mass of the hydrograph, then K = tc/2. Equations (4) and (5) then become respectively,

$$C_1 = \frac{\Delta t}{tc + \Delta t} \tag{8}$$

and

$$C_2 = \frac{tc - \Delta t}{tc + \Delta t}. \tag{9}$$

(d) The validity of any hydrologic model is best tested by the model's ability to reproduce observed flood hydrographs. The absence of runoff information in the study area required use of data from other sources. In the experimental program conducted by the Los Angeles District during the period 1948 to 1954 (reference 17), simulated rainfall produced runoff from surfaces of both concrete and simulated turf, set at various slopes. Different combinations of rainfall intensity, slope, length of plane, and surface roughness led to a variety of flow patterns. Plates 18 through 21 show some of the observed hydrographs together with reproduced hydrographs computed using equations (3), (8), and (9). Equation (7) was used to compute tc. For the reproduction on plate 21, the higher intensity was used in equation (7). Much of the discrepancy between the observed and reproduced hydrographs is probably due to the non-linearity of the rainfall-runoff process, although some is undoubtedly due to experimental error. For example, the observed steady-state discharge on plate 19 is about 2 percent greater than the theoretical steady-state discharge computed from rainfall. If K in the model was varied in an appropriate manner, less runoff would take place at the beginning of the hydrograph and more could be made to occur near the peak. However, the reproduced peak discharges correspond fairly well with the observed peaks.

(e) Reconstitutions of observed flood events in urbanized basins outside of the study area are shown on plates 22, 23, and 24. Although not technically sheetflow areas because runoff is concentrated by streets or, in the case of El Modena-Irvine, conveyed in a channel, hydrograph reproduction using the linear reservoir model are generally good. Measurement of lag time (time between centers of mass of rainfall excess and runoff) compares favorably with

(f) Further evidence of both the non-linearity of the runoff process and the appropriateness of the relationship for K in the model can be seen in the storage-outflow loops shown on plates 25 through 31. The coefficient K is equal to the ratio of the change in storage ΔS to the change in outflow ΔO. For a good portion of the various loops, $\Delta S/\Delta O$ approximates the computed K very closely. In other parts of the loops, the non-linearity of the storage-outflow relationship is obvious.

(g) A review and evaluation of the model was performed by the Hydrologic Engineering Center. The Center's comments, along with remarks by the SCS, are included as Appendix II.

b. Precipitation Loss Rates. In that the soil types in the Gila Floodway basin are fairly similar to soils in the Phoenix area, the loss rates used in this study were based on the loss function presented in reference 2, reproduced here as plate 32. Consideration of on-site storage, along with other necessary assumptions, suggested the use of an initial loss and an average constant loss rate. Using the "SPF-Local Storm" loss function on plate 32, the average loss rate during the fifth and sixth hours, the intense portion of the storm, were determined. This value, 0.35 inch per hour, was used as the constant loss rate for SPF calculations.

c. Depression Storage and Manning's N-Value.

(1) Summer storms in the study area often occur on relatively dry watersheds. Although the soil may have been wetted by antecedent rainfall, evaporation rates are high, and depression storage must be satisfied prior to runoff. The amount of this initial loss and the Manning's n-value for various types of surfaces used in this study are within the range recommended by Chow (reference 18). They are:

Surface Type	Depression Storage (inches)	Manning's N-Value
Impervious areas............	0.0625	0.015
Residential lawns...........	0.200	0.100
Natural terrain.............	0.500	0.050
Developed farmland..........	2.000	0.070
Citrus groves...............	Total Storm	No Contribution*
Pasture land................	2.000	0.200

*Citrus groves are considered non-contributing to runoff because of the dikes built around the groves to facilitate irrigation by flooding. These dikes are 6 inches or more in height.

Where a subarea contained more than one type of surface, some scheme had to be devised to compute average parameters for the composite catchment. In this study, depression storage was calculated as a weighted average based on the percent imperviousness of the catchment. For example, depression storage for a residential subarea with x fraction impervious was calculated by

$$\text{Average depression storage} = 0.0625(x) + 0.20(1-x) \tag{10}$$

In the case of Manning's n, a harmonic mean was used, following reference 20:

$$\frac{1}{n} = \frac{x}{0.015} + \frac{(1-x)}{0.10} \tag{11}$$

where 0.015 and 0.10 are the Manning's coefficients for impervious and residential lawn areas, respectively, taken from the above table.

(2) An extensive study of the sensitivity of the final results to various changes in Manning's n-value was not performed. Certain n-value changes made in the course of the study indicate, however, that the final solution is not extremely sensitive to changes in n. In one case, n was changed from 0.075

to 0.040, nearly 90 percent, with a resultant change in peak of about 25 percent and only a small change in timing of the peak. When n was increased from 0.030 to 0.050, about 70 percent change, the peak decreased about 15 percent. In this range of Manning's n, it appears that the ratio of change in n to change in the magnitude of the peak is about 4 or 5 to 1.

d. Baseflow and Snowmelt. Baseflow is considered negligible in the study area. Allowance for snowmelt is inappropriate in this region for storms occurring in the summer season.

4-04. FLOOD ROUTING.

a. Flood routing techniques in this study area 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 reasonably describe the attenuation of a flood wave when a large amount of channel storage or overland flow is encountered. For this reason, the Muskingum method was chosen for channel routing. Reservoir routing, where appropriate, was accomplished by the Modified Puls routing procedure.

b. Flood wave travel time in a reach is normally determined by dividing reach length by average peak flow velocity. Muskingum K value is approximated by the travel time. For overland flow over the flat topography typical of the study area, however, appropriate cross sections for velocity are difficult if not impossible to ascertain. For this study, a value of 1.5 feet per second was chosen as the flood wave velocity in overland flow areas.

This value compares favorably with slope-velocity curves such as those used by the SCS and the City of Los Angeles. An overland flow velocity of 1.5 feet per second was in fact used by SCS in their design of RWCD Floodway. A Muskingum X value of zero was used to approximate the rather level water surface profile of overland flow.

4-05. CANAL LEVEE FAILURE ASSUMPTIONS. Storage behind canal levees was computed using a variable depth, sloping wedge as the model for ponding areas. Volumes were calculated by the average end area method with maximum depths equal to the levee heights as illustrated on plate 33. Average height of levees and surface areas of ponding were determined by field inspection and from 1:24,000 scale topographic maps with superimposed 2 feet contour intervals taken from reference 7. Canal levee failure hydrographs were determined based on the following assumptions: (1) the levee will fail rapidly, but not instantaneously; (3) the maximum water surface elevation will be approximately the original levee height; (4) the failure section will erode to accommodate inflow plus dead storage (wedge) release; (5) dead storage outflow is maximum when inflow is maximum; (6) dead storage release varies linearly; and (7) outflow equals inflow plus dead storage release. Plate 34 shows the effect of canal levee failure for a hypothetical situation.

4-06. COMPUTATION OF STANDARD PROJECT FLOOD.

a. Stream System Analysis.

(1) A stream system analysis approach to computation of design floods involves division of the 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 are then routed and combined to determine the design flood at a desired location. Subdividing a watershed permits more accurate modeling of the runoff process, as variations in topography, urbanization, and rainfall, as well as consideration of on-site storage requirements and man-made barriers may be incorporated into the hydrologic description of the basin. The required number of subbasins are governed by the size of the basins used to verify the model.

(2) Standard project flood is computed by centering the standard project storm over the basin in the most critical flood producing manner. Application of the rainfall loss function described previously to standard project precipitation enables determination of the rainfall excess hyetograph. The appropriate runoff model, unit hydrograph for mountain areas and linear reservoir for valley areas, is then used to transfer the effective rainfall hyetograph into a runoff hydrograph for the subbasin, considering any on-site storage requirement. Routing and combining of all subarea hydrographs to the desired location completes the computation of SPF.

b. SPF peak discharges and total storm volumes, computed as described in the foregoing paragraphs, are presented in tables 1 and 2 for present without project and future without project conditions, respectively.

V. DISCHARGE FREQUENCY ANALYSIS

5-01. STREAMGAGE RECORDS.

a. Only two stream gages in the Gila Floodway basin have records over 10 years in length, both in relatively mountainous areas. The gage on Queen Creek at Whitlow damsite, near Superior, has a fragmented record

16 years long (1915-1920, 1948-1959). Data from this gage, along with others in the mountains around Phoenix (see reference 2), were used to estimate 100-year floods for the design adequacy analysis of the SCS structures in the mountain portions of the study area. The Queen Creek Tributary at Apache Junction gage has 14 years of record (1961-1974), but the data is affected by high channel percolation rates and is therefore somewhat unreliable.

b. An analysis of the records of stream gages in seven other basins in the general area with flat drainage slopes was attempted, but the results showed very little correlation on which to base runoff frequency. The gages used were Durham Wash, Silver Reef Wash, Agua Fria Tributary at Youngtown, Military Wash, Waterman Wash, Rainbow Wash, and Bender Wash.

5-02. DISCHARGE FREQUENCY DETERMINATION.

a. In the absence of runoff data from comparable basins, flood frequency in the Gila Floodway basin was based on rainfall frequency. The idea of determining flood frequency from rainfall frequency is not new. The intention is to estimate the flood of a selected frequency from rainfall of the same frequency. The actual relationship between frequency of rainfall and the derived flood is obscure as each part of the computational model introduces some joint probability. For this reason, frequency analysis of observed runoff data is the preferred procedure.

b. The basic premise adopted in this study was that if "average" values of other parameters such as Manning's n value and loss rate are used, the frequency of the derived flood should approximate the frequency of rainfall. The rainfall parameters chosen to preserve the consistency between rainfall and runoff frequency were the maximum 15 minute, 30 minute, and 1 hour precipitation amounts. Due to the nature of summer storms in the area, these parameters are good indicators of storm severity.

c. Summer storms generally last 12 hours or less, with most of the rain falling within 3 hours. The intense portion of the storm often lasts 1 hour or less. A comprehensive analysis of temporal patterns for summer storms in the area, described in reference 2, formed the basis for determining the average time distribution of rainfall employed in this study.

d. The maximum 15 minute, 30 minute, and 1 hour precipitation amounts were determined from n-year 6 hour and n-year 24 hour rainfall amounts and regression equations for finding n-year t-hour amounts TP-40 presented in reference 19. Depth-area relationships were based on the storm analysis presented in reference 2.

e. N-year flood peak discharge and total storm volumes were calculated in the same manner as SPF. Constant loss rates, estimated from the "Dry Watershed" loss function on plate 32, were adopted as follows:

Storm Frequency Years	Constant Loss Rate Inches per hour
100	0.40
50	0.45
25	0.50
10	0.55

Peak discharges and total storm volumes for selected locations are given in tables 1 and 2.

VI. ANALYSIS OF RESULTS

6-01. GENERAL. A quick glance at the discharge values presented in tables 1 and 2 may be disconcerting. It appears that the given discharges are low, in some cases extremely low. Plotting these values on an enveloping curve of peak discharges which have occurred in Arizona leads to the same conclusion. However, the following factors must be kept in mind: (1) the study area for the most part is very flat, and flood waters do not concentrate, traveling instead as sheetflow; (2) as will be shown, on-site storage can drastically alter the quantity and timing of runoff from a subbasin; (3) man-made barriers have changed drainage patterns, caused substantial opportunity for ponding, and, in certain cases, caused areas to be non-contributing to flow at a particular location; and (4) land in agricultural use has the potential to store about 2 inches of rain before runoff occurs.

6-02. EFFECT OF ON-SITE STORAGE.

a. On-site storage can drastically change the expected quantity and timing of runoff from an area. Maricopa County's on-site storage ordinance can be used to illustrate the effect. It will be recalled from previous discussion in paragraph 2-07g (4) that the County requires sufficient storage, retention, detention, or a combination of both, such that the peak discharge, generated by a 100-year 2 hour rainfall, leaving the developed subdivision, does not exceed the predevelopment peak, computed from the same storm. Detention storage is envisioned to react similar to a reservoir with an outlet, the maximum capacity of the outlet being equal to the predevelopment 100-year peak discharge.

Retention storage can be likened to a reservoir with an overflow spillway; the reservoir capacity would be equal to the accumulated volume of the developed condition 100-year hydrograph up to the discharge on the recession limb of the hydrograph equal to the predevelopment 100-year peak.

b. Plate 35 is a graphical description of the effect of each type of storage on a hydrograph. Note that the magnitude of the developed subbasin peak after taking on-site storage into account is significantly less than before the adjustment, but still greater than the predevelopment peak. This occurs in this case because SPF volume is greater than 100-year flood volume on which on-site storage is based. The volume of runoff from the developed subbasin has increased due to urbanization, but not as much as might be expected because of the assumption that half of the area would use <u>retention</u> type storage.

c. Plates 36 and 37 show the progression of floodwaters from subarea to subarea through a basin. Plate 36 represents the predeveloped condition; plate 37 shows the effect of on-site storage and routing on magnitude and timing of the peak. It can be seen that the magnitude of the peak, when on-site storage is considered, does not increase as much as might be expected with increased urbanization, and, indeed, may actually decrease.

d. The effect of on-site storage will vary both with the amount of rainfall (frequency of storm) and the land use of the basin before development. If the magnitude of the considered storm is small, runoff may not occur from a subbasin with retention type storage. Also, the peak discharge allowed from a subbasin which had been in agricultural use before development would be somewhat smaller than the allowable peak from, say, a basin in its natural state before urbanization because of the relative difference in predevelopment runoff potential. Since the allowable peak discharges would be different, the required storage volumes would also be different.

6-03. DISCUSSION OF RESULTS.

a. A few specific examples will best illustrate the concepts discussed in the foregoing paragraphs. Table 1 shows that most of the SPF peak discharges and volumes are very much greater than the corresponding 100-year flood peaks and volumes. The large difference at CP's 1220, 1304, 1310A, and 1310B is explained by the agricultural land use of the subbasins and the magnitude of the storm causing runoff. Point rainfall hyetographs for the standard project and 100-year storms are shown on plate 38. Note that the required storage for agricultural land use - 2 inches - is not satisfied until near the end of the 100-year storm. The intense portion of the 100-year storm can be seen to be ineffective in producing high runoff from agricultural areas. However, the maximum intensities of the standard project storm are not significantly affected.

b. Additional contributing drainage area for SPF is also responsible for some of the large differences between SPF and 100-year peaks and volumes. This situation occurs with CP 1751, 1753, 1223, and 1224. The additional area contributes runoff to these points because of levee breaches or overflow from RWCD Floodway during SPF.

c. The pattern of large SPF peaks and volumes compared with 100-year values is also apparent in table 2. In this case, the difference in peak is due to the on-site storage requirement for future development. The 100-year future condition peak is of the same order of magnitude as the 100-year present condition peak, which is the intention of the on-site storage requirement. The future condition volume is greater, however, but there is not as much difference as might be expected because of the assumption that half of the developed subareas would have retention type on-site storage.

6-04. COMPARISON WITH OTHER STUDIES.

a. Soil Conservation Service RWCD Floodway Design Study. Plate 39 shows a comparison of SCS and Corps 100-year flood peak discharge values at selected locations along the proposed RWCD Floodway. The SCS discharge values will be used in the design of the floodway. The Corps values were computed using the linear reservoir procedure described in this report. The results obtained from the two different methods are within approximately 25 percent of each other, which is considered acceptable in this case. The average difference is about 15 percent.

b. Superstition Freeway, Conceptual Study For Drainage. Only one location in this study, CP 1701, corresponds directly to concentration points determined for the Superstition Freeway conceptual drainage study (reference 11). The freeway drainage study, done by Yost and Gardner Engineers for the Arizona Department of Transportation, employed a modified Rational Method to compute 50-year flood peaks. Fifty-year flood peaks at CP 1701, computed by the two different methods, were almost identical. However, the 50-year 6-hour volume given in the freeway study is about 10 to 25 percent higher than the Corps volume (392 acre-feet vs. 360-300 acre-feet) depending on whether present or future conditions are being compared. The difference is mainly due to different assumptions regarding potential ponding behind canal levees. Since the main purpose of the freeway study was to provide protection for the freeway, conservative assumptions may be justified.

c. City of Mesa Flood Insurance Study. Discharges presented in the flood insurance study (FIS) for the City of Mesa are not directly comparable to those given in the Gila Floodway study, although the computation methods are the same. The FIS was done for 1975 conditions, while the Gila Floodway study considers present conditions to be conditions existing in 1985. In the next ten years, Superstition Freeway and RWCD Floodway are expected to be constructed; in addition, future urbanization must consider Mesa's on-site storage requirement. These factors will serve to reduce the flood potential in Mesa from what it is today.

TABLE 1

PEAK DISCHARGES AND TOTAL STORM VOLUMES

PRESENT CONDITIONS WITHOUT PROJECT

C.P.	Contributing Drainage Area SPF Sq. Mi.	Contributing Drainage Area Other Sq. Mi.	Standard Project Flood	100-Year Flood	50-Year Flood	25-Year Flood
			City of Mesa			
1701	96	33	6,700 cfs (3,350 AF)	3,500 cfs (570 AF)	2,300 cfs (360 AF)	1,400 cfs (210 AF)
1751	260	29	4,500 cfs (4,300 AF)	600 cfs (380 AF)	-- --	-- --
1220	6	6	3,400 cfs (1,080 AF)	550 cfs (210 AF)	300 cfs (100 AF)	100 cfs (50 AF)
			Vicinity of Gilbert			
1753	240	20	4,000 cfs (2,520 AF)	600 cfs (320 AF)	100 cfs* (10 AF)	50 cfs* (10 AF)
			Vicinity of Chandler			
1223	14	5	5,000 cfs (2,330 AF)	1,000 cfs (190 AF)	500 cfs (90 AF)	350 cfs (50 AF)
1224	17	3	4,800 cfs (2,680 AF)	500 cfs (100 AF)	200 cfs (60 AF)	50 cfs (10 AF)

TABLE 1 Con't

PEAK DISCHARGES AND TOTAL STORM VOLUMES

PRESENT CONDITIONS WITHOUT PROJECT

C.P.	Contributing Drainage Area SPF Sq. Mi.	Other Sq. Mi.	Standard Project Flood	100-Year Flood	50-Year Flood	25-Year Flood
			Gila Floodway			
1304	18	18	4,300 cfs (2,750 AF)	400 cfs (330 AF)	-- --	-- --
1310A	37	37	9,700 cfs (5,680 AF)	1,100 cfs (800 AF)		
1310B	70	70	19,000 cfs (11,600 AF)	2,500 cfs (1,770 AF)	1,500 cfs (800 AF)	700 cfs (330 AF)
1362	95	95	18,000 cfs (13,100 AF)	3,100 cfs (2,400 AF)	-- --	-- --
1372	117	117	17,000 cfs (15,700 AF)	4,000 cfs (3,130 AF)	-- --	-- --
1382	136	136	16,000 cfs (18,000 AF)	4,100 cfs (3,900 AF)	2,900 cfs (2,500 AF)	2,000 cfs (1,520 AF)

* Consolidated Canal in subarea 653 does not breach for 50-year and 25-year Floods.

TABLE 2

PEAK DISCHARGES AND TOTAL STORM VOLUMES

FUTURE CONDITIONS WITHOUT PROJECT

C.P.	Contributing Drainage Area SPF Sq. Mi.	Contributing Drainage Area Other Sq. Mi.	Standard Project Flood	100-Year Flood	50-Year Flood	25-Year Flood
			City of Mesa			
1701	96	33	6,700 cfs (3,630 AF)	3,300 cfs (490 AF)	2,000 cfs (300 AF)	1,100 cfs (160 AF)
1751	260	29	4,600 cfs (4,280 AF)	800 cfs (690 AF)	-- --	-- --
1220	6	6	3,000 cfs (1,160 AF)	700 cfs (380 AF)	300 cfs (280 AF)	300 cfs (240 AF)
			Vicinity of Gilbert			
1753	240	20	4,000 cfs (2,540 AF)	650 cfs (350 AF)	150 cfs* (20 AF)	100 cfs* (20 AF)
			Vicinity of Chandler			
1223	14	5	5,200 cfs (2,450 AF)	800 cfs (250 AF)	600 cfs (200 AF)	550 cfs (170 AF)
1224	17	3	5,400 cfs (2,900 AF)	500 cfs (180 AF)	200 cfs (150 AF)	200 cfs (120 AF)

TABLE 2 Con't

PEAK DISCHARGES AND TOTAL STORM VOLUMES

FUTURE CONDITIONS WITHOUT PROJECT *

C.P.	Contributing Drainage Area SPF Sq. Mi.	Other Sq. Mi.	Standard Project Flood	100-Year Flood	50-Year Flood	25-Year Flood
			Gila Floodway			
1304	18	18	4,900 cfs (3,080 AF)	700 cfs (840 AF)	-- --	-- --
1310A	37	37	10,000 cfs (6,060 AF)	2,200 cfs (1,810 AF)	-- --	-- --
1310B	70	70	19,000 cfs (10,300 AF)	4,000 cfs (2,480 AF)	2,900 cfs (1,810 AF)	2,300 cfs (1,340 AF)
1362	95	95	18,000 cfs (13,200 AF)	4,300 cfs (3,540 AF)	-- --	-- --
1372	117	117	17,000 cfs (15,700 AF)	4,300 cfs (4,300 AF)	-- --	-- --
1382	136	136	16,000 cfs (18,000 AF)	4,300 cfs (5,060 AF)	3,000 cfs (3,620 AF)	2,300 cfs (2,620 AF)

* Consolidated Canal in subarea 653 does not breach for 50-year and 25-year floods.

TABLE 3

Effective Impervious Cover Estimates

Land Use	Units Per Acre	Percent Effective Impervious Cover
Agricultural		0
Low Density Residential	5	25
Medium Density Residential	5-10	35
High Density Residential	Over 10	40
Commercial		90
Industrial		70

TABLE 4

History of Storms and Floods
Gila Floodway Basin and Vicinity

Year	Description	Source of Data*
1953	Rain began about 10:00 p.m. on 16 July. Storm was pretty general over the area north and east of Williams Air Force Base and extending into Superstition Mountains. Precipitation averaged about 0.75 inch in a 2 hour period. Peak discharge estimated at Queen Creek Road (now called Santan Road) bridge crossing the waterway just east of RWCD canal was 1,330.	1
1953	At about 4:00 p.m. on 29 July, precipitation began in the area. The most damaging storm was located in the Chandler Heights area, extending south into the Santan Mountains and east about 8 miles. During the storm, which lasted until about 10:00 a.m. on 30 July, 1.20 inches of rain is said to have fallen in 20 minutes. Highwater marks on Sonqui Creek at Power Road indicates a peak discharge of 425 cfs.	2
1954	19 August; See paragraph 3-01a.	
1957	31 October; See paragraph 3-01b.	
1959	Storm occurred on 17 August. West of Florence Junction, Queen Creek came to within 2 feet of floor of bridge. In the town of Queen Creek, creek overflowed, running into homes, damaging crops and irrigation ditches.	3
1963	On evening of 16 August, unstable masses of moist air moved into eastern Maricopa County from northeast and from southeast. Storm was multicellular event with most intense cell centered over Glendale. Another cell was centered just north of US Highway 60-70-80-89 west of Apache Junction and east of Mesa. The average duration of the storm was about 4 hours. A maximum storm total of about 3.5 inches was estimated. Just over 1 inch fell in the City of Mesa. Many businesses along the Highway were flooded by runoff from the hills. A few houses had water on the first floor. Many residences were trailers, and were built high. Residential yards were subjected to both scouring and sedimentation. Several	4

*See footnotes at end of table.

TABLE 4 (Continued)

History of Storms and Floods
Gila Floodway Basin and Vicinity

Year	Description	Source of Data*
1963	county roads were extensively damaged, and RWCD Canal was breeched in 16 places, mostly between Broadway Road and Southern Avenue. Most farmland damage was in conjunction with canal breaks and ponded water trying to get into and over canal. Damage consisted of inundation of crops, sedimentation, and some scouring.	
1969	On 14 September, the Tempe-Mesa-Chandler area was hit by an intense thunderstorm, causing some severe flash flooding. The Weather Service raingage near Baseline Road and 56th Street recorded 3.52 inches between 6:00 p.m. and 7:00 p.m. Guadalupe reported 2.73 inches in the same period. Chandler recorded 0.61 inch for the hour. The areal coverage of the storm was fairly small. A Weather Service employee living in South Phoenix estimated the diameter of the cloud shaft to be 3 miles, with the shaft extending to 50,000 feet. Runoff flooded streets and irrigation ditches, and canals in the area filled to overflowing.	5,6
1971	Just after midnight on 16 August, the area around Apache Junction began to experience flooding. Heavy precipitation must have occurred in the Goldfield Mountains. The Weather Service reported 3.40 inches at Horse Mesa Dam and the Forest Service recorded 3 inches at Tortilla Flat. A large crew spent all day clearing sediment from Apache Trail.	7
1972	General precipitation began in the Gila Floodway Basin at about 6:00 p.m. on 18 October. By 8:00 p.m. on the 19th the storm was pretty well over. Twenty-six hour rainfall amounts ranged from about 1.5 inches in the Mesa-Chandler-Gilbert area to about 4.5 inches in the Queen Creek-Magma basins. In the Queen Creek area, the cotton crop yield was reduced 10-60 percent by flood damage, depending on the water depth. The depth of flooding in the fields varied from 0.5 feet to 2.6 feet. Floodwaters ponded behind and overtopped the railroad tracks at several locations. The flood retarding structures in the basins north of Queen Creek controlled the runoff eminating from the mountains above them, thus significantly reducing damage in the lower portions of the watershed.	8

*See footnotes at end of table.

TABLE 4 (Continued)

History of Storms and Floods
Gila Floodway Basin and Vicinity

*Source of Data.

1. SCS-Phoenix Files, Office Memorandum dated 17 July 1953.

2. SCS-Phoenix Files, Office Memorandum dated 31 July 1953.

3. SCS-Phoenix Files, Office Memorandum dated 18 August 1959.

4. SCS-Phoenix Files, Office Memorandum dated 28 October 1963.

5. Letter from Maricopa County Flood Control District to Los Angeles District Corps of Engineers dated 8 October 1969.

6. Newspaper article, "The Arizona Republic" Phoenix, Monday, September 15, 1969.

7. SCS-Phoenix Files, handwritten note.

8. SCS-Phoenix Files, Administrator's General Memorandum dated 23 April 1973.

APPENDIX 1

REFERENCES

1. Hydrology Report for Type 15 Flood Insurance Study, Mesa, Arizona (and subsequent revisions), U.S. Army Engineer District, Los Angeles, Corps of Engineers, 30 April 1975.

2. 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, October 1974.

3. USDA Environmental Impact Statement (Draft), Buckhorn-Mesa Watershed, Maricopa and Pinal Counties, Arizona, USDA Soil Conservation Service, February 1975.

4. Watershed Work Plan, Buckhorn-Mesa Watershed, Maricopa and Pinal Counties, Arizona, USDA Soil Conservation Service, January 1963.

5. Watershed Work Plan, Apache Junction-Gilbert Watershed, Maricopa and Pinal Counties, Arizona, USDA Soil Conservation Service, January 1963.

6. Watershed Work Plan, Williams-Chandler Watershed, Maricopa and Pinal Counties, Arizona, USDA Soil Conservation Service, January 1963.

7. Stormwater Drainage for the City of Mesa, Yost and Gardner Engineers, Phoenix, Arizona, Project No. 71-6, November 1973.

8. Soil Survey, Eastern Maricopa and Northern Pinal Counties Area, Arizona, USDA Soil Conservation Service, November 1974.

9. General Soil Map, Maricopa County, Arizona, USDA Soil Conservation Service, 1973.

10. Gila River Basin, Arizona and New Mexico, Design Memorandum No. 1, Hydrology for Whitlow Ranch Reservoir, Queen Creek, Arizona, U.S. Army Engineer District, Los Angeles, Corps of Engineers, July 1957.

11. Superstition Freeway, Conceptual Study for Drainage (Tempe Canal to RWCD Canal), Project No. F-028-1-402 PE, Yost and Gardner Engineers, February 1975.

12. Report on Flood of 19 August 1954, Queen Creek and Vicinity, Arizona, U.S. Army Corps of Engineers, Los Angeles District, 24 September 1954.

APPENDIX 1

REFERENCES (Continued)

13. Report on Floods of 31 October 1957, Santa Rosa Wash, Queen Creek and Vicinity, Arizona, U.S. Army Engineer District, Los Angeles, Corps of Engineers, 28 February 1958.

14. Time in Urban Hydrology, G.E. Willeke, Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, HY1, January 1966.

15. Overland Flow and Groundwater Flow from a Steady Rainfall of Finite Duration, F.M. Henderson and R.A. Wooding, Journal of Geophysical Research, Vol. 69, No. 8, April 15, 1964.

16. Kinematic Wave Nomograph for Times of Concentration, R.M. Ragan and J.O. Durn, Journal of Hydraulics Division, Proceedings of the American Society of Civil Engineers, HY 10, October 1972.

17. Runoff from Impervious Surfaces, Y.S. Yu and J.S. McNown, U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi, Contract Report No. 2-66, February 1963.

18. Open-Channel Hydraulics, Ven Te Chow, McGraw-Hill Book Company, 1959.

19. Precipitation-Frequency Atlas of the Western United States, Vol. VIII, Arizona, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, 1973.

20. Storm Water Management Model, Volume I-Final Report, by Metcalf and Eddy, Inc., University of Florida, and Water Resources Engineers, Inc. for the Environmental Protection Agency, July 1971.

APPENDIX 2

Review and Evaluation of Rainfall-Runoff Model
by the Hydrologic Engineering Center

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
THE HYDROLOGIC ENGINEERING CENTER
609 2D STREET, DAVIS, CALIFORNIA 95616

SPKHE 29 August 1975

SUBJECT: Request for Review and Evaluation of Rainfall-Runoff Model for Use in Sheet Flow Areas

District Engineer
US Army Engineer District, Los Angeles
ATTN: SPLED-HE
P.O. Box 2711
Los Angeles, California 90053

1. Reference your letter SPLED-HE, dated 25 June 1975, subject as above.

2. Review comments are contained in the inclosed Special Projects Memo No. 441.

3. We appreciate the opportunity to conduct the review. If you have any questions regarding review comments, please do not hesitate to contact us.

FOR THE DISTRICT ENGINEER:

Incl
as

Bill S. Eichert
BILL S. EICHERT, Director
The Hydrologic Engineering Center

SPKHE 29 August 1975

SPECIAL PROJECTS MEMO NO. 441

SUBJECT: Review of Rainfall-Runoff Model for Use in Sheet Flow Areas

1. References:

a. SPLED-HE 25 June 1975, Subject: Request for Review and Evaluation of Rainfall-Runoff Model for Use in Sheet Flow Areas.

b. Letter SPKHE 14 July 1975 to Chief, Engineering Division, Los Angeles District.

2. The subject review was requested by the Los Angeles District in reference a. Reference b suggested arrangements for the review, including the holding of a meeting at HEC to discuss the technical bases for the subject model. Messrs. John Pederson and Daniel Norling of the Los Angeles District met with Messrs. Dale Burnett, Arthur Pabst and John Peters at HEC on 19 August 1975. The following paragraphs contain the HEC's review comments based on that meeting and on subsequent review of written material pertaining to the subject model.

3. The rainfall-runoff problem that required modeling for the Mesa, Arizona Flood Insurance Study and Gila Floodway Study is indeed complex. Runoff is expected to occur as sheet flow; that is, runoff will spread over broad planar surfaces rather than concentrate in well-defined channels. There are a number of obstructions to the flow in the form of low, uncompacted levees on the upstream sides of irrigation canals which traverse the area. These levees are certain to fail when overtopped, but yet the temporary impoundment of runoff caused by these levees and intersecting roadway embankments cannot be ignored. Portions of the drainage area have undergone substantial urban development.

A basic element of the subject rainfall-runoff model is application of the concept of the single linear reservoir for transforming rainfall excess to runoff. Studies at Purdue University and elsewhere have demonstrated that this concept is appropriate and useful for small drainage areas (e.g. less than 5 sq. miles) and we concur with its application.

5. Application of the linear reservoir procedure requires evaluation of a parameter K which represents the slope of a linear relationship between outflow and storage. The parameter K can be shown to be equal to the time interval between the centers of mass of rainfall excess and direct runoff. In the case where the runoff process is linear, as assumed in unit hydrograph theory, this time interval is independent of both intensity

and duration of rainfall excess. In the proposed method, K is assumed to equal 50% of the time of concentration, where time of concentration is defined by an equation by Ragan and Duru as a function of length, slope, roughness and intensity of effective rainfall.

6. The derivation of the relationship between K and t_c (i.e. $K = 0.5\ t_c$) is subject to question. The steps leading to this relationship are valid only under certain conditions. The time of concentration, t_c is always less than the time to equilibrium, t_e (or 97% of the time to equilibrium, t_{97}). The larger the amount of storage in a watershed (or runoff plane), the greater will be the difference between t_c and t_{97}. The t_c may be taken to be analogous to t_{97} only where surface storage (detention storage in the flow profile) is quickly satisfied. The conclusion that the time to peak may be used to approximate the time to the center of mass as implied in Plate 17 is valid only when the hydrograph is symmetrical about the peak. Thus the derived relationship may not be valid in general. The relationship bears similarity to another relationship used in Soil Conservation Service procedures - $t_p = 0.6\ t_c$. Because K is typically greater than t_p, it seems that K could be expected to be a larger proportion of t_c than 50%. However, successful reproduction of observed events affirms the useability of the relationship. It would be desirable to determine the sensitivity of the final results to various changes in K. This would provide a basis for judging whether a need exists for a more refined relationship.

7. Application of the equation for t_c by Ragan and Duru requires that rainfall intensity be specified. The equation is based on a condition of constant rainfall intensity. To apply the equation to storm rainfall of varying intensity, an assumption must be made as to what rainfall intensity to use. In the subject procedure, the maximum 15-minute rainfall intensity was adopted.

8. The equation for t_c by Ragan and Duru also requires that a roughness coefficient, n, be specified. A problem exists in selecting n values, and in establishing composite n-values for non-homogeneous areas. For example, n values could be weighted in accordance with the size of area to which they apply, or criteria such as the inverse weighting relationship recommended for use in the EPA Storm Water Management Model could be used. The latter criteria was adopted for calibration purposes.

9. Because of the assumptions discussed in the preceding paragraphs, the desirability of verifying the adopted procedures against field data is apparent. The verifications on data for the El Modena-Irvine channel and Aqua Fria tributary are certainly encouraging, but it is hoped that additional verification can be made in the future.

10. Once runoff from subareas has been computed, assumptions are required with respect to ponding behind, and failure of, levees. The procedure for

estimating depth-volume characteristics of ponding areas appears to be adequate and appropriate. Assumptions implicit in modeling levee breaks are that the levees will not fail until they are overtopped, the water surface behind the levee will not exceed the top of levee elevation, and discharge from dead storage behind the breeched levee will be in proportion to the 'inflow' hydrograph to the ponded area. The latter assumption is somewhat tenuous but should be adequate provided that release from dead storage is not the major portion of flow through the levee.

11. Muskingum routing criteria adopted for the study assumes a constant velocity of flood wave travel of 1.5 feet per second and an X of 0. Because of the changing ground slope through the study area, it might be more appropriate to vary the velocity with the square root of slope. However, because of the uncertainties regarding the paths that the flow will follow, the adopted criteria are reasonable.

12. In summary, the subject model is logical, reasonable and appropriate for the problem being modeled. The complexity of the problem requires numerous assumptions, some of which are commented on above. No runoff data are available in the study area. The results of application of the model would appear to yield a reasonable assessment of the hydrologic behavior of the area modeled. As additional data become available it would be desirable to reevaluate model results and further validate the procedures used.

John C. Peters

Copy furnished:
Chief, Engineering Divison
Los Angeles District

JOHN C. PETERS, Chief
Training and Methods Branch
The Hydrologic Engineering Center

Bill S. Eichert

BILL S. EICHERT, Director
The Hydrologic Engineering Center

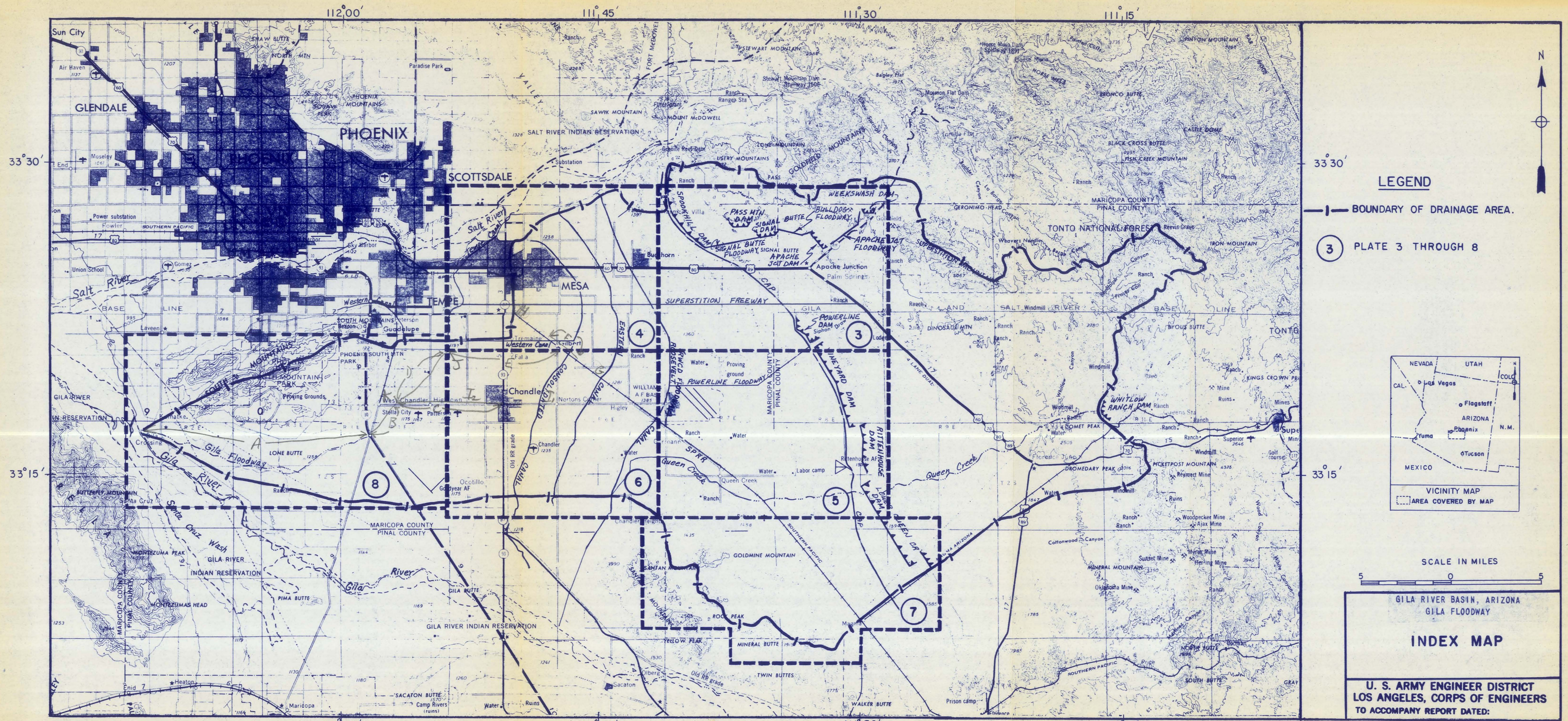
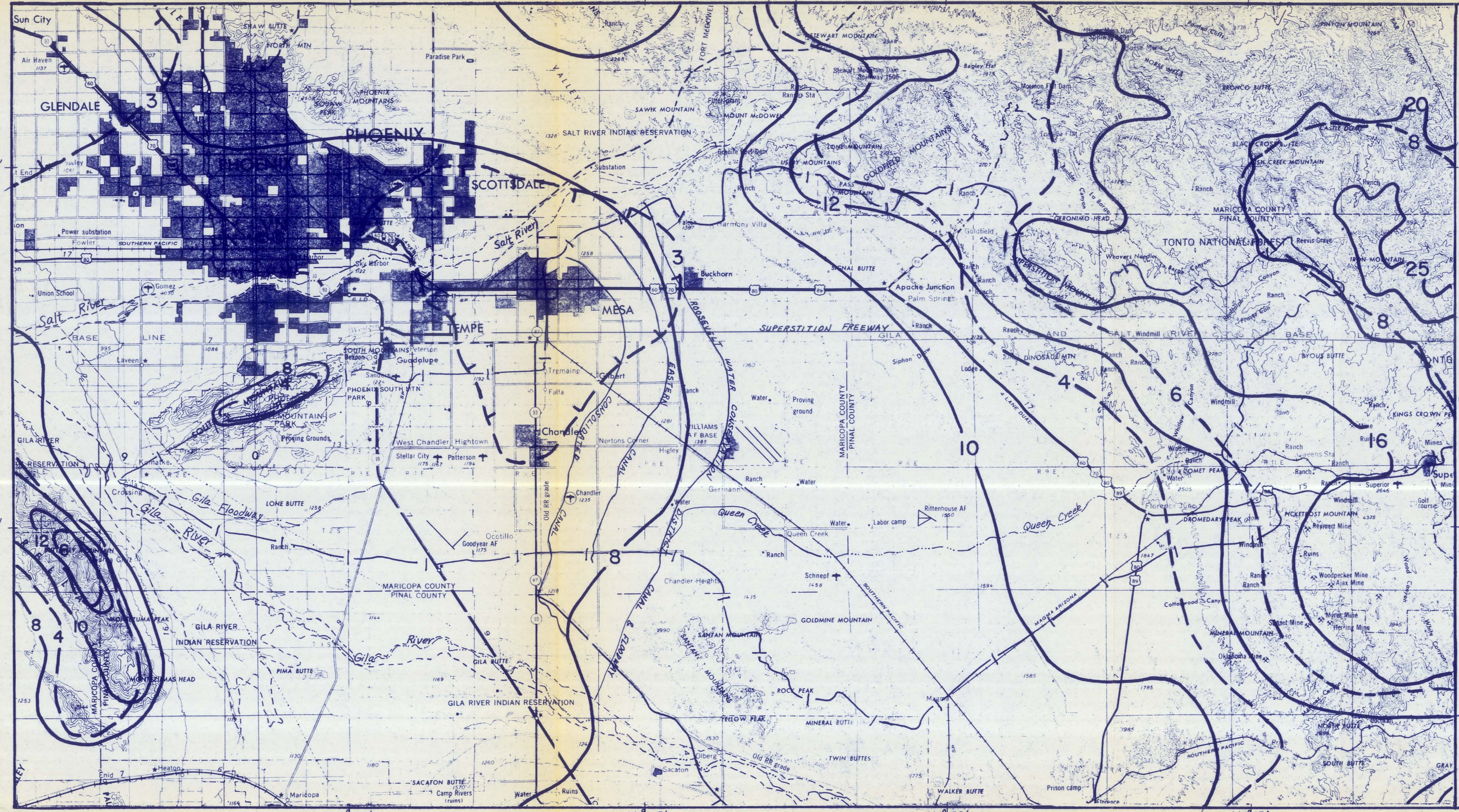

GILA RIVER BASIN, ARIZONA
GILA FLOODWAY
INDEX MAP
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:
LEGEND
BOUNDARY OF DRAINAGE AREA.
3 PLATE 3 THROUGH 8
SCALE IN MILES
5 0 5
VICINITY MAP
AREA COVERED BY MAP
NEVADA
UTAH
COLO
CAL.
Las Vegas
Flagstaff
ARIZONA
Phoenix
N.M.
Yuma
Tucson
MEXICO
112°00'
111°45'
111°30'
111°15'
33°30'
33°15'
N
Sun City
GLENDALE
PHOENIX
SCOTTSDALE
TEMPE
MESA
Chandler
Guadalupe
Salt River
Gila River
Gila Floodway
SUPERSTITION FREEWAY
POWERLINE FLOODWAY
POWERLINE DAM
RITTENHOUSE DAM
VINEYARD DAM
WEEKSWASH DAM
BULLDOG FLOODWAY
PASS MTN. DAM
SIGNAL BUTTE DAM
SIGNAL BUTTE FLOODWAY
APACHE JCT DAM
APACHE JCT FLOODWAY
WHITLOW RANCH DAM
SPOOKHILL DAM
RWCD FLOODWAY
Queen Creek
TONTO NATIONAL FOREST
MARICOPA COUNTY
PINAL COUNTY
SALT RIVER INDIAN RESERVATION
GILA RIVER INDIAN RESERVATION
SOUTH MOUNTAINS
SUPERSTITION MTNS
GOLDFIELD MOUNTAINS
USERY MOUNTAINS
GOLDMINE MOUNTAIN
SANTAN MOUNTAIN
Santa Cruz Wash
1
2
3
4
5
6
7
8

PLATE 1



ISOLINES OF NORMAL ANNUAL PRECIPITATION IN INCHES.

ISOLINES OF NORMAL MAY-SEPTEMBER PRECIPITATION IN INCHES.

BOUNDARY OF DRAINAGE AREA.

SCALE IN MILES

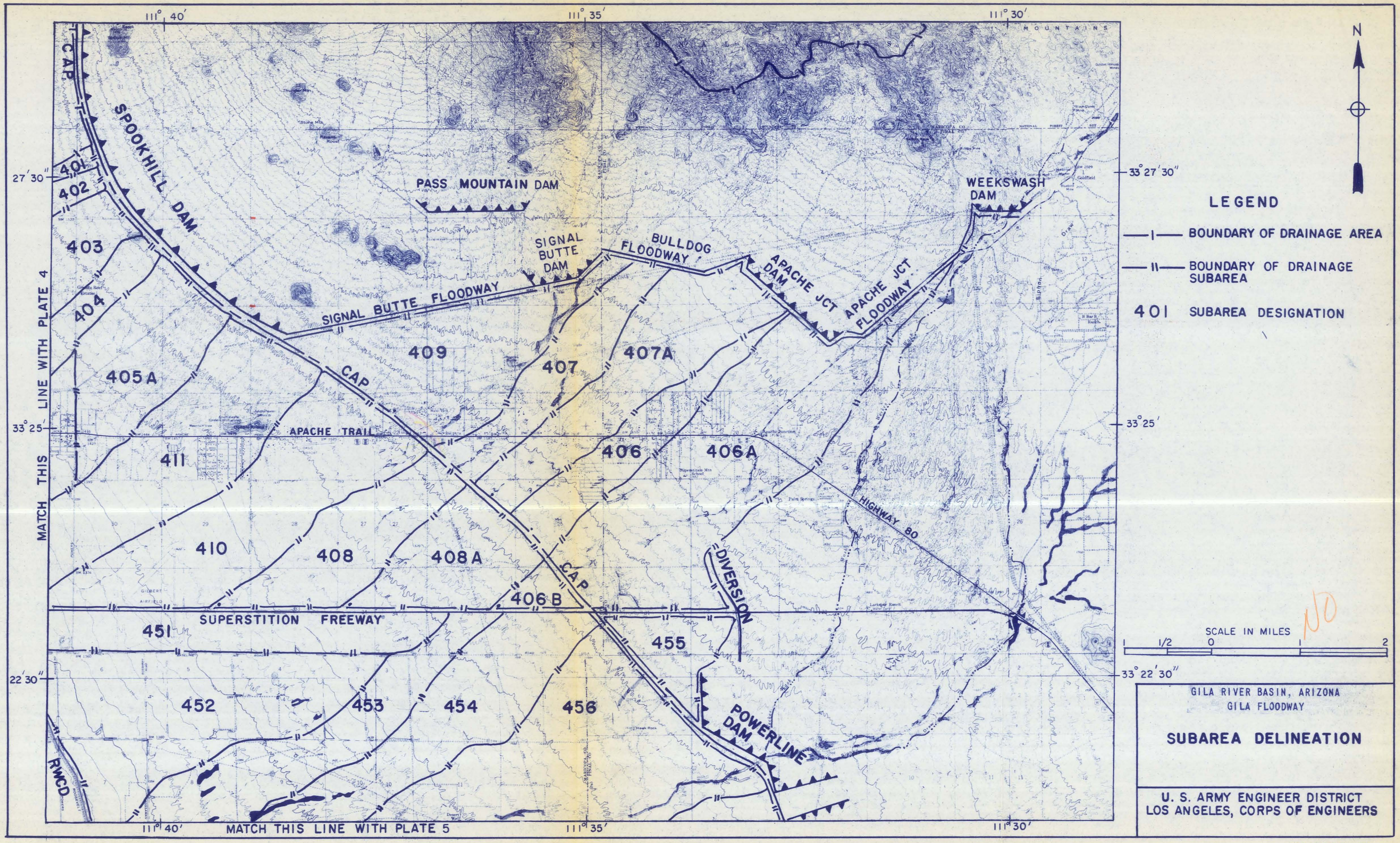
5 0 5

GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

HYDROLOGIC MAP

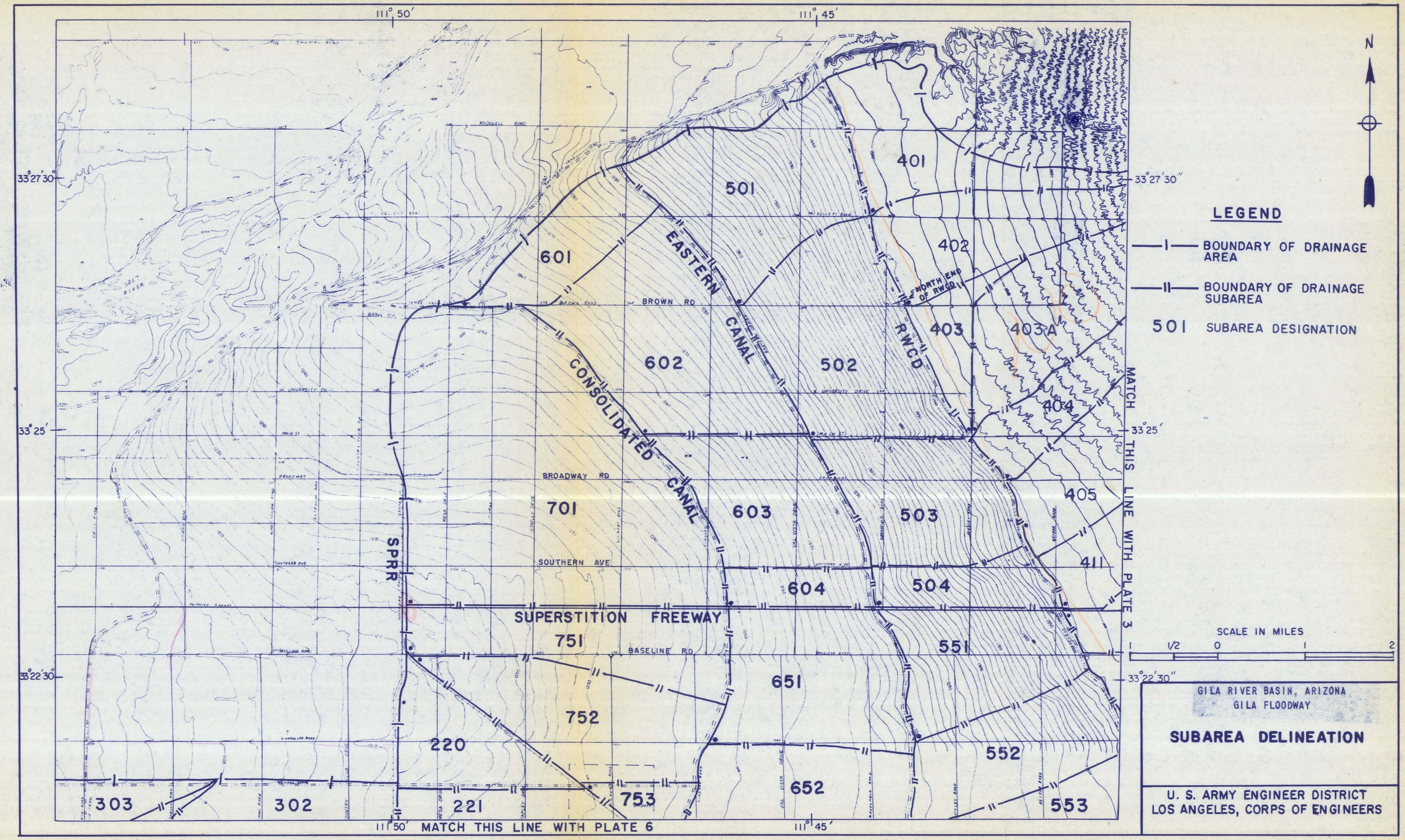
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE 2

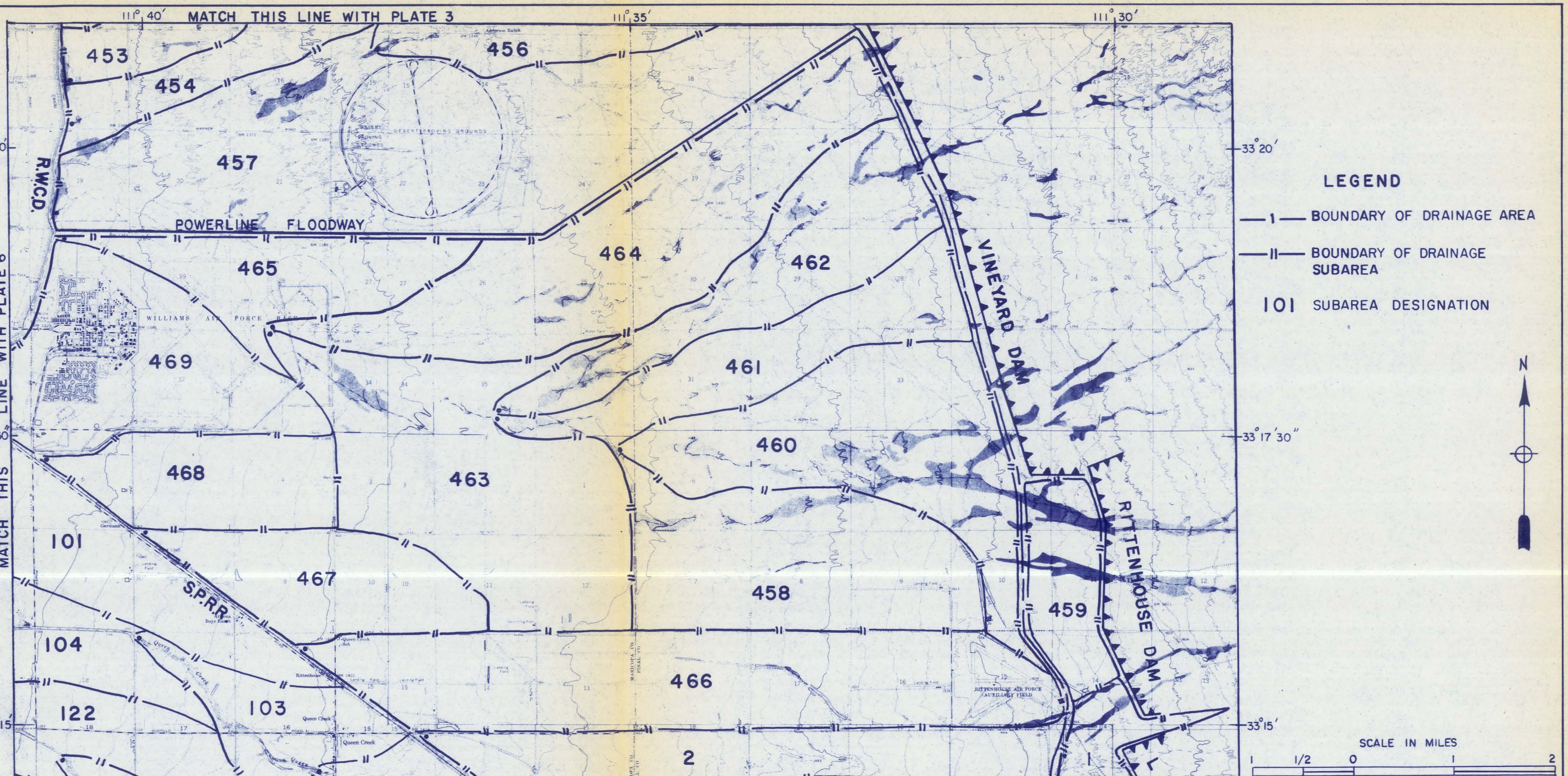


111° 40′
111° 35′
111° 30′
27′ 30″
33° 25′
22′ 30″
33° 27′ 30″
33° 25′
33° 22′ 30″
MATCH THIS LINE WITH PLATE 4
MATCH THIS LINE WITH PLATE 5
CAP
SPOOKHILL DAM
PASS MOUNTAIN DAM
SIGNAL BUTTE DAM
SIGNAL BUTTE FLOODWAY
BULLDOG FLOODWAY
APACHE JCT DAM
APACHE JCT FLOODWAY
WEEKSWASH DAM
APACHE TRAIL
HIGHWAY 80
DIVERSION
SUPERSTITION FREEWAY
POWERLINE DAM
RWCD
401
402
403
404
405A
409
407
407A
411
406
406A
410
408
408A
406B
451
455
452
453
454
456
N
LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
401 SUBAREA DESIGNATION
SCALE IN MILES
1 1/2 0 1 2
NO
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY
SUBAREA DELINEATION
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

PLATE 3



LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
501 SUBAREA DESIGNATION
SCALE IN MILES
1 1/2 0 1 2
MATCH THIS LINE WITH PLATE 3
MATCH THIS LINE WITH PLATE 6
EASTERN CANAL
CONSOLIDATED CANAL
RWCD
NORTH END OF RWCD
SPRR
SUPERSTITION FREEWAY
BROWN RD
BROADWAY RD
SOUTHERN AVE
BASELINE RD
SALT RIVER
401
402
403
403A
404
405
411
501
502
503
504
551
552
553
601
602
603
604
651
652
701
751
752
753
220
221
302
303
111° 50′
111° 45′
33° 27′ 30″
33° 25′
33° 22′ 30″
N
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY
SUBAREA DELINEATION
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS



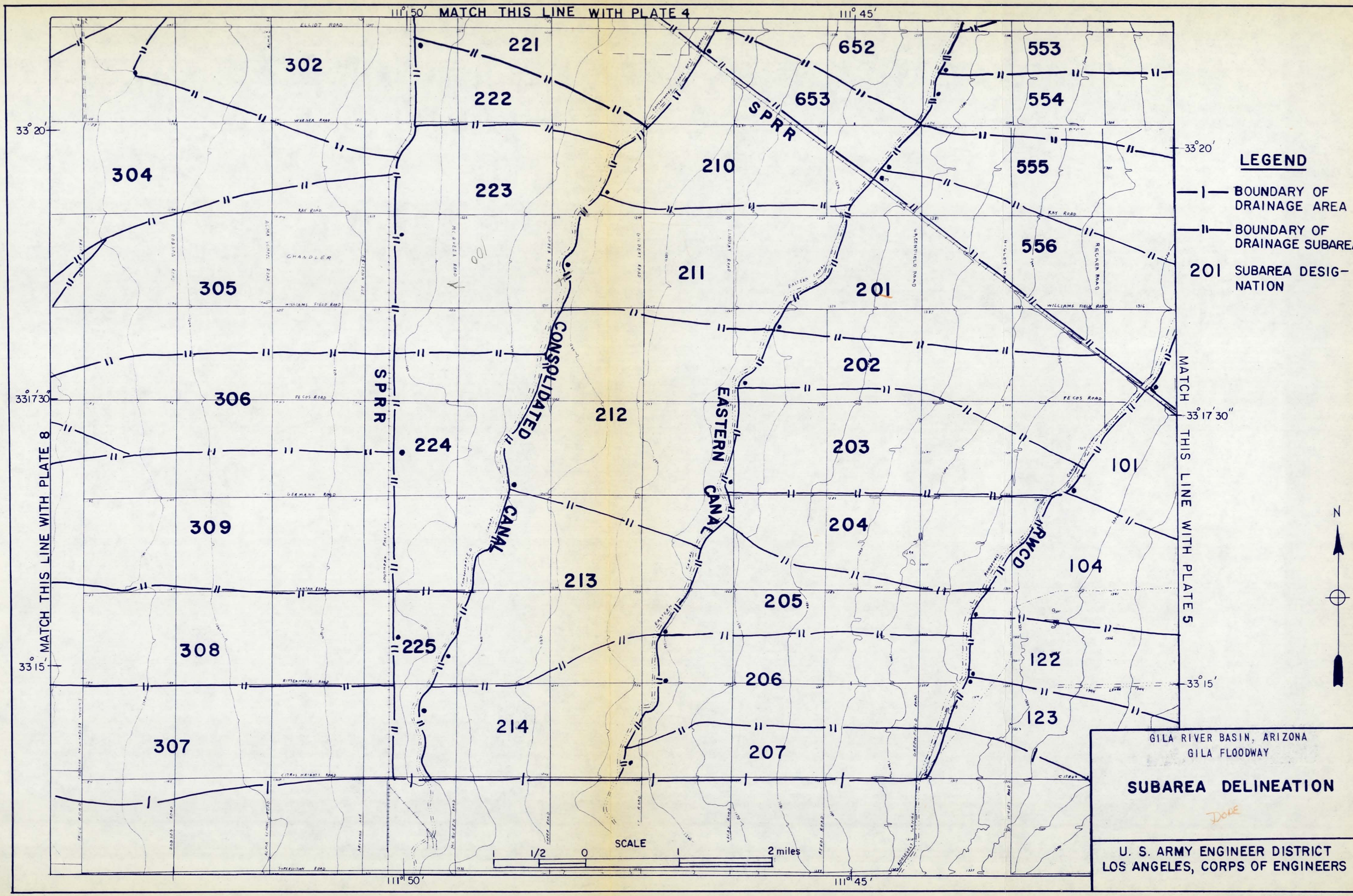
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SUBAREA DELINEATION

NO

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

PLATE 5



GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SUBAREA DELINEATION

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

PLATE 6

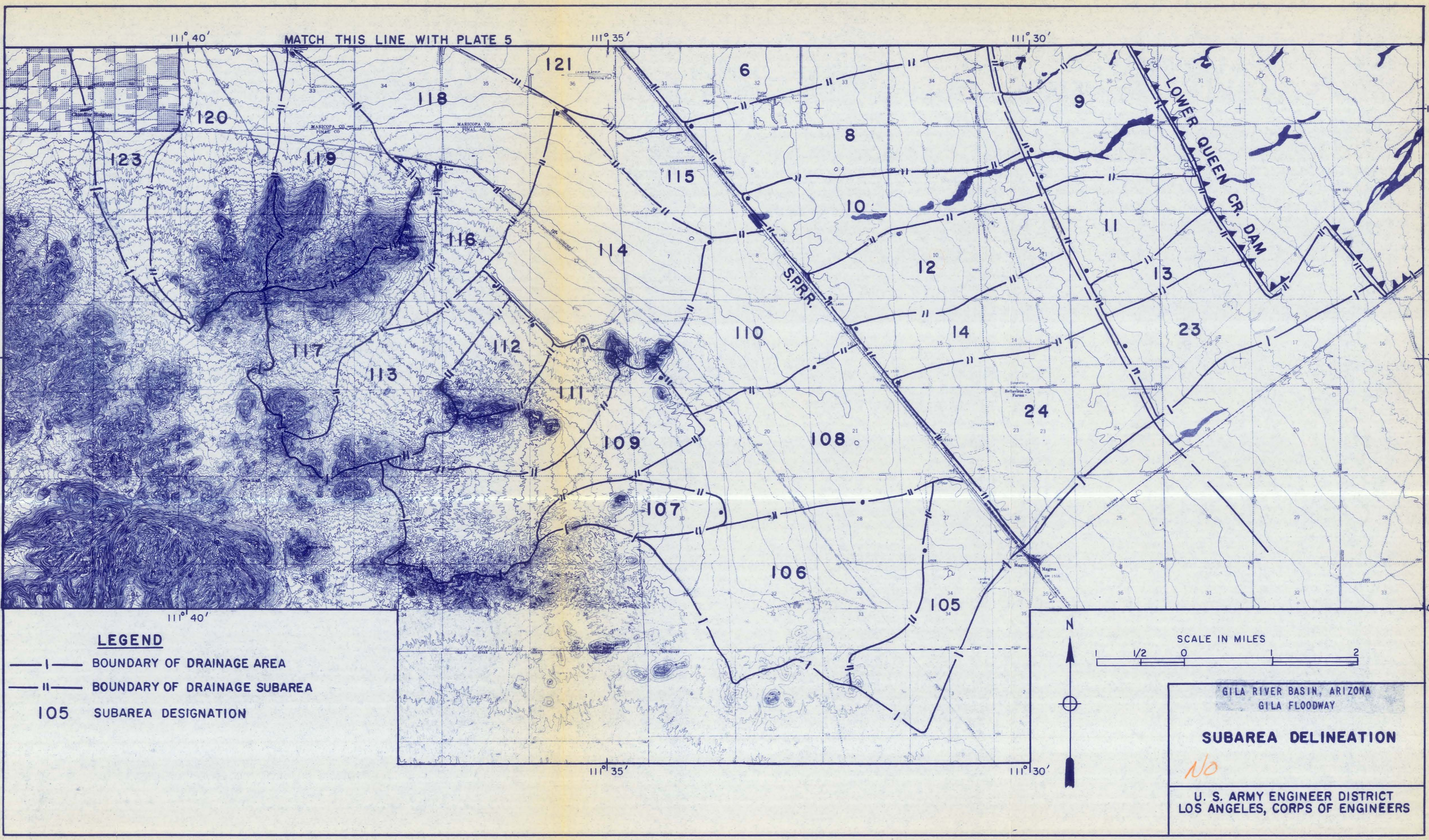
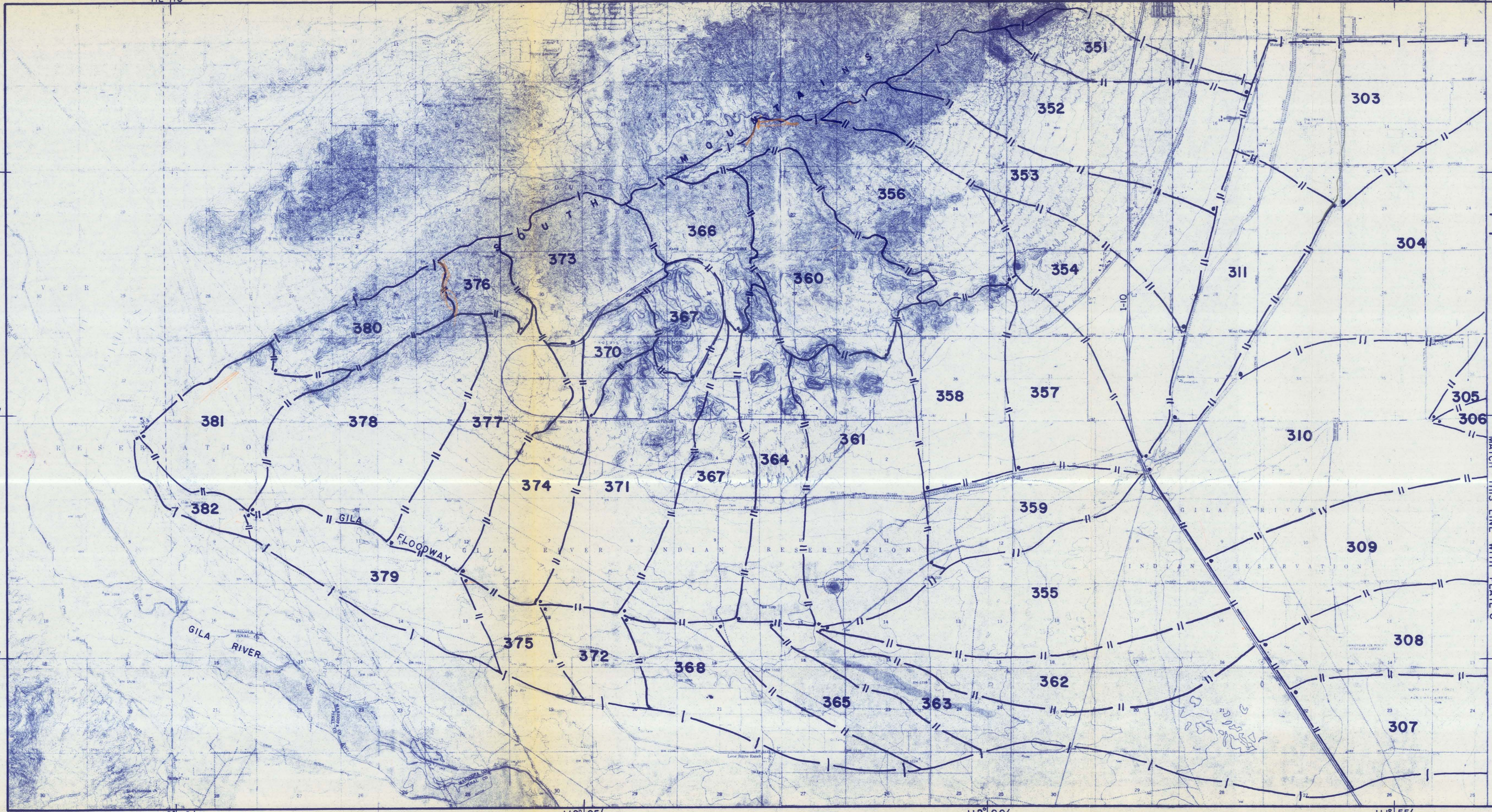

MATCH THIS LINE WITH PLATE 5
111° 40′
111° 35′
111° 30′
33° 12′ 30″
12′ 30″
33° 10′
33° 10′
33° 07′ 30″
07′ 30″
LOWER QUEEN CR. DAM
S.P.R.R.
6
7
8
9
10
11
12
13
14
23
24
105
106
107
108
109
110
111
112
113
114
115
116
117
118
119
120
121
123
LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
105 SUBAREA DESIGNATION
N
SCALE IN MILES
1 1/2 0 1 2
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY
SUBAREA DELINEATION
NO
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

PLATE 7



N

LEGEND

BOUNDARY OF DRAINAGE AREA.
BOUNDARY OF DRAINAGE SUBAREA.
304 SUBAREA DESIGNATION.

SCALE IN MILES
1 1/2 0 1 2

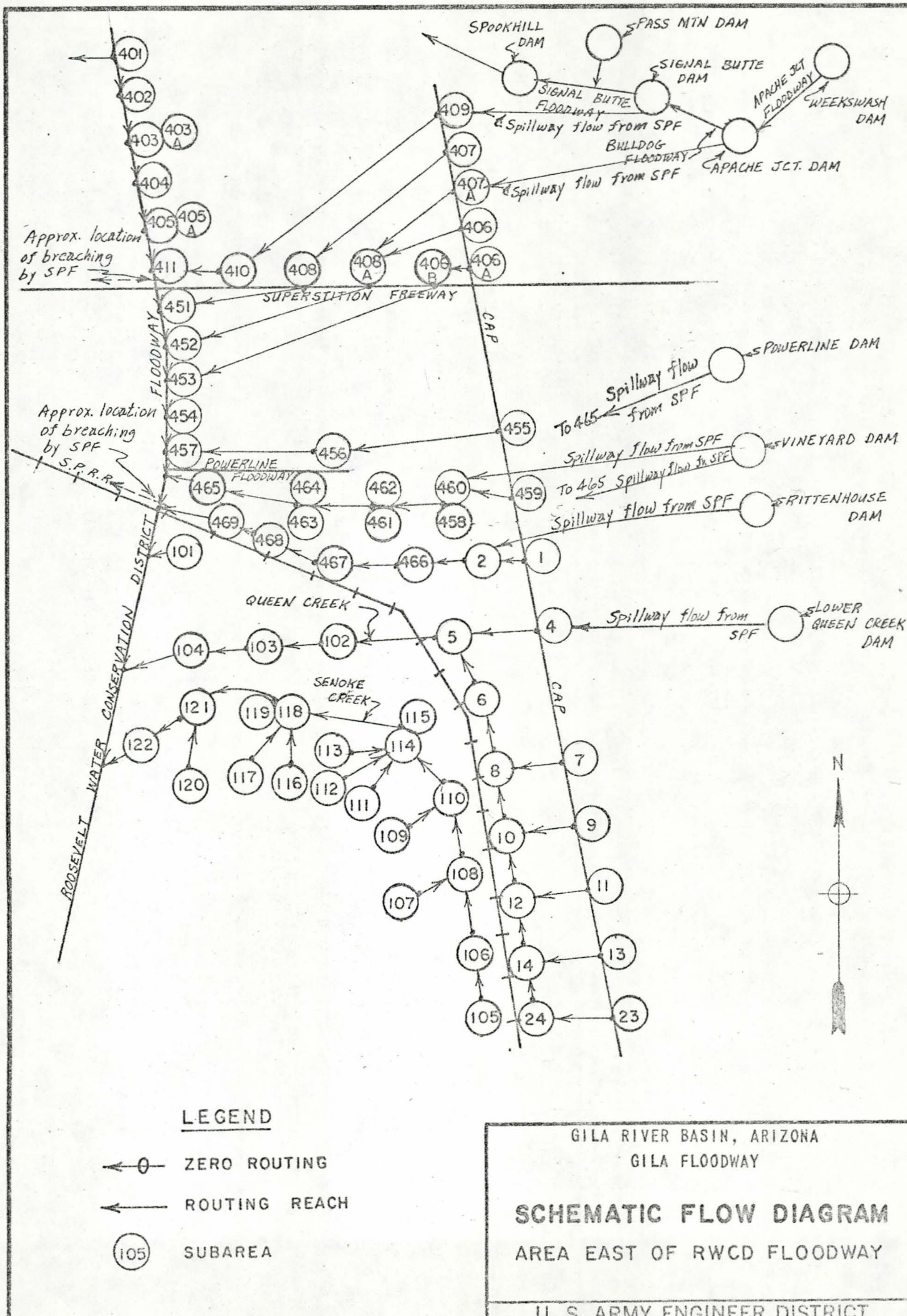
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SUBAREA DELINEATION

Done

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

PLATE 8

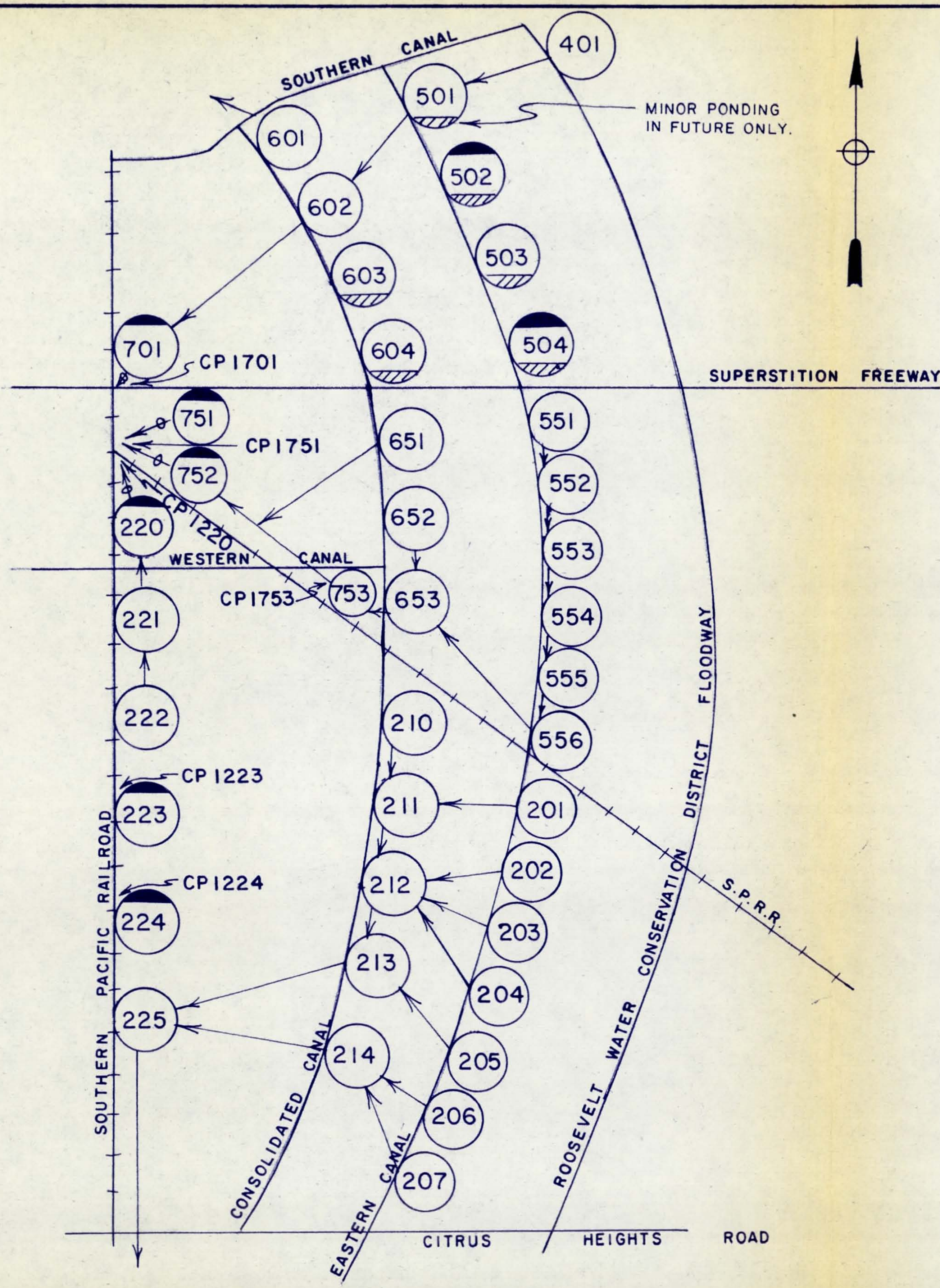


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SCHEMATIC FLOW DIAGRAM

AREA EAST OF RWCD FLOODWAY

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



LEGEND

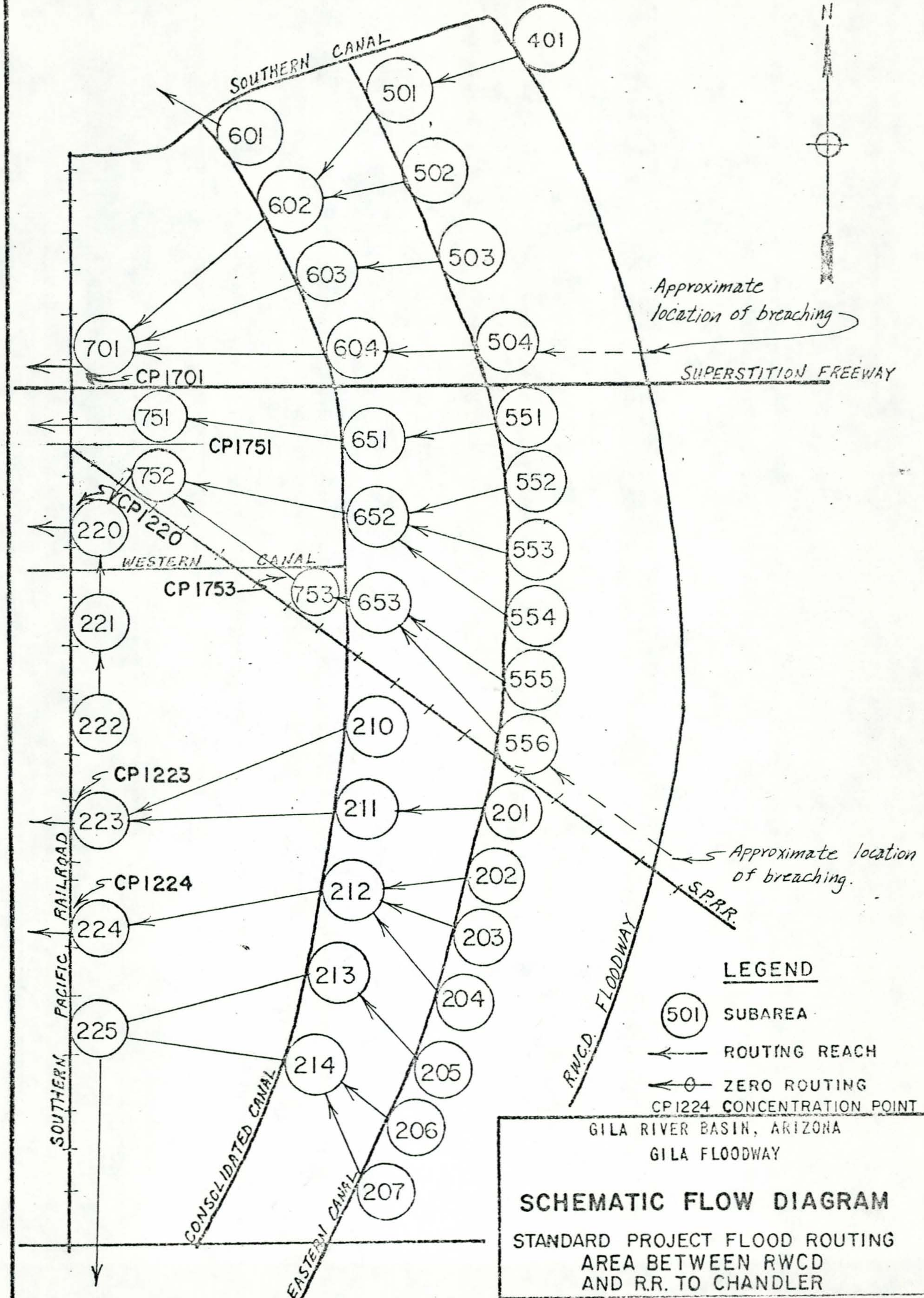
501	Subarea
CP 1701	Concentration Point
←	Routing Reach
←0—	Zero Routing
701	Total runoff volume ponded behind barrier.
604	Minor ponding due to street runoff. On-site storage **ordinance in effect.**
502	Total runoff volume ponded behind barrier in present conditions. Minor ponding due to street runoff in future conditions; on-site storage ordinance in effect.

GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SCHEMATIC FLOW DIAGRAM

100-YEAR FLOOD ROUTING
AREA BETWEEN RWCD
AND R.R. TO CHANDLER

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

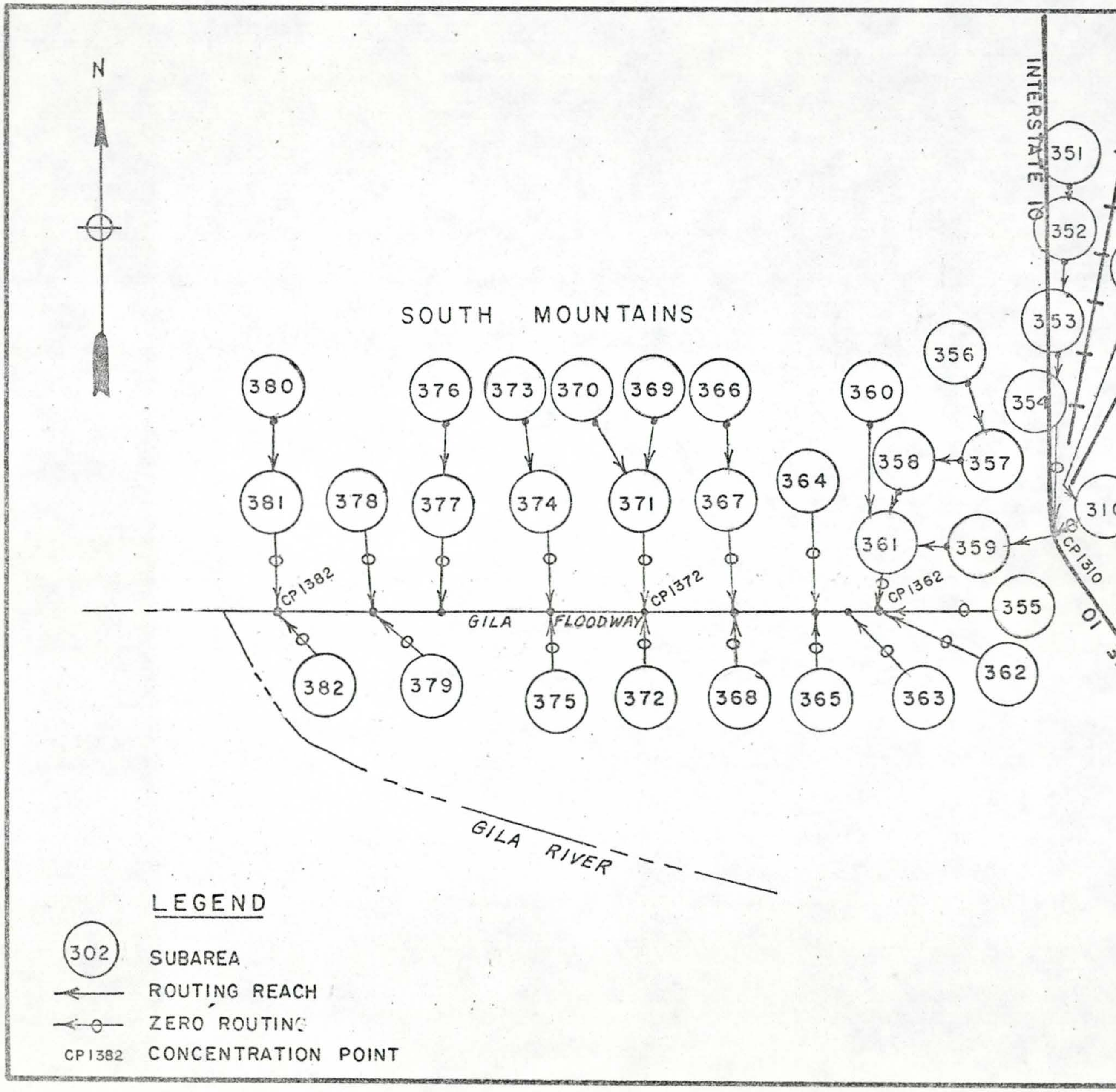


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SCHEMATIC FLOW DIAGRAM

STANDARD PROJECT FLOOD ROUTING
AREA BETWEEN RWCD
AND R.R. TO CHANDLER

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

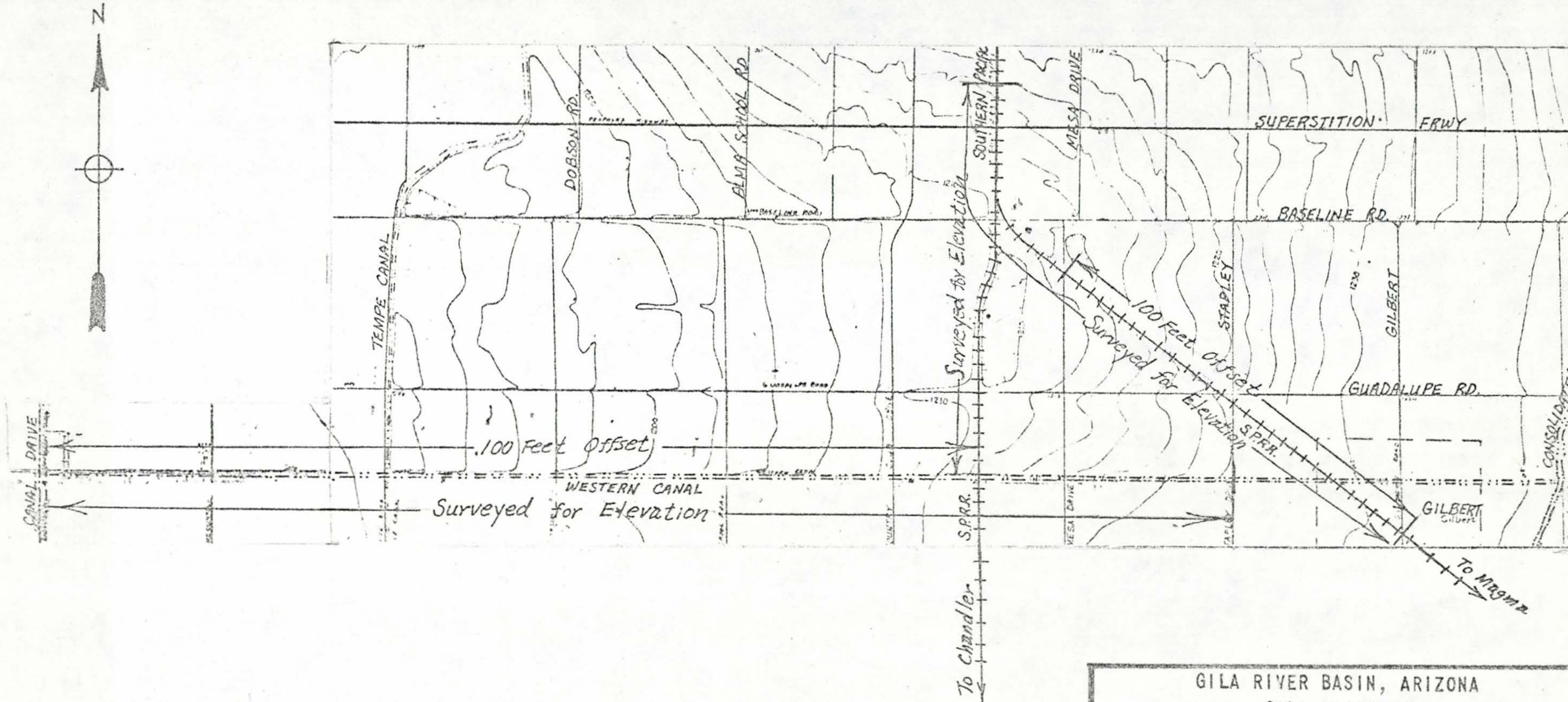


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SCHEMATIC FLOW DIAGRAM
AREA WEST OF RR TO CHANDLER

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

PLATE 12



GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

EXTENT OF CORPS SURVEY

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

PLATE 13

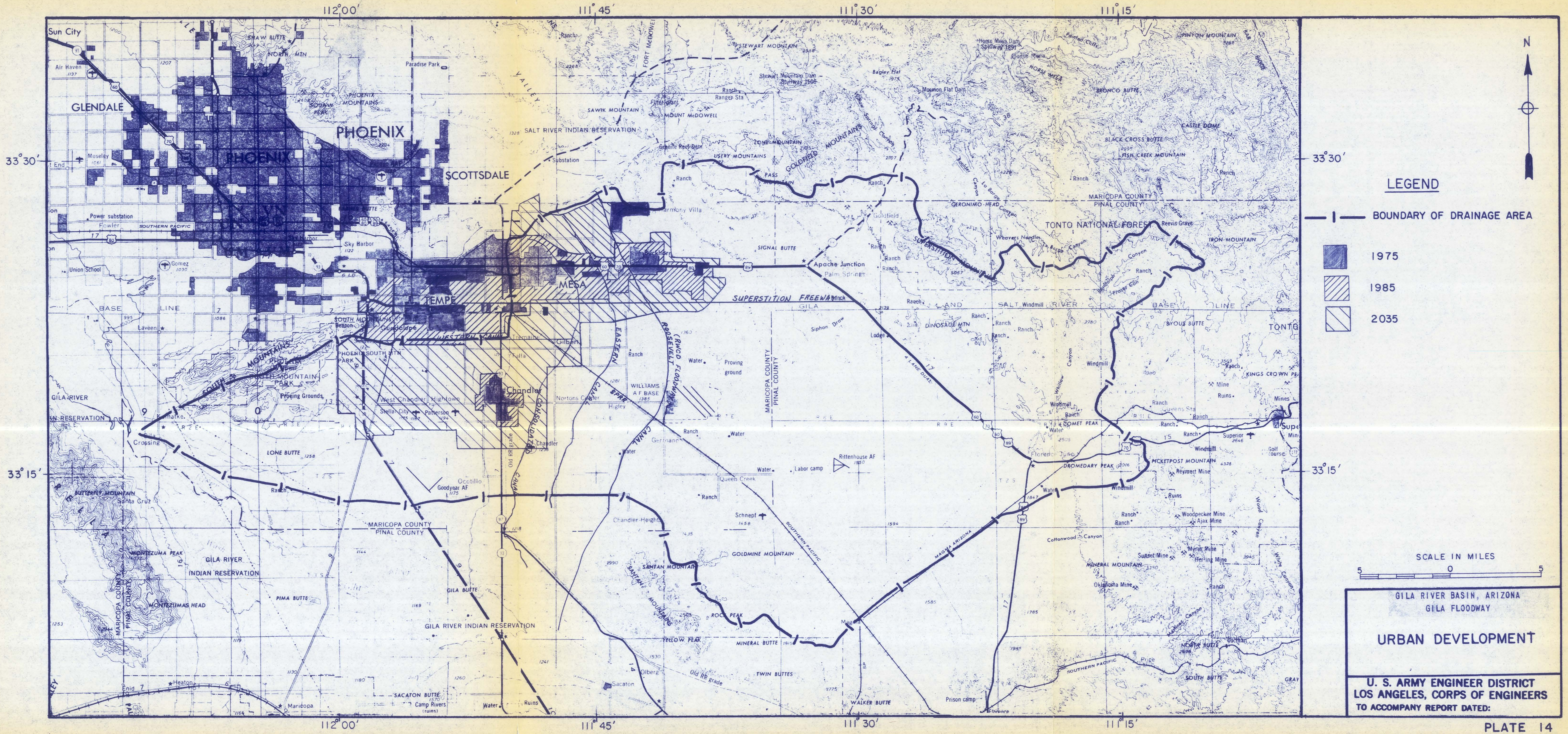
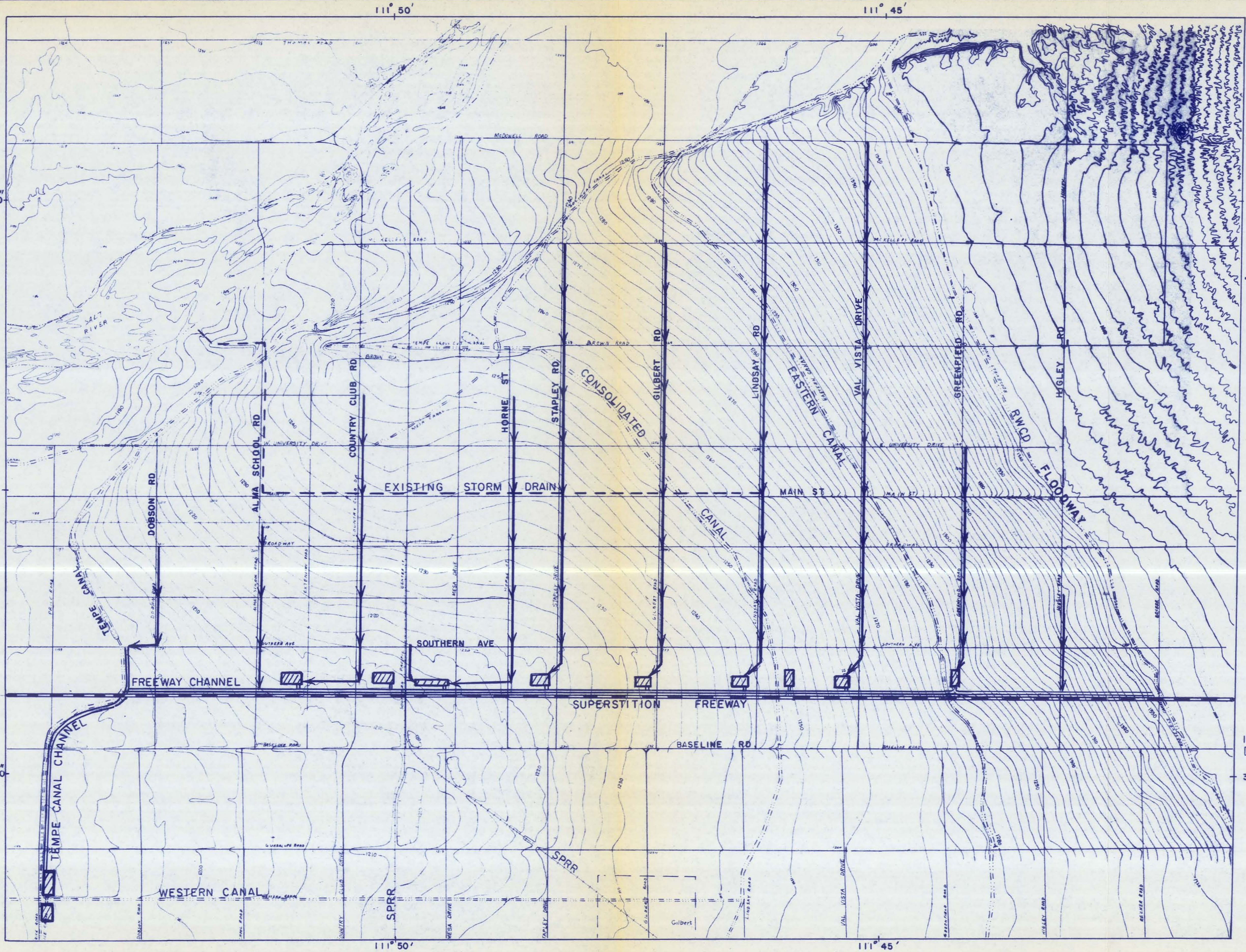

LEGEND
BOUNDARY OF DRAINAGE AREA
1975
1985
2035
SCALE IN MILES
5 0 5
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY
URBAN DEVELOPMENT
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:
112°00'
111°45'
111°30'
111°15'
33°30'
33°15'
PHOENIX
GLENDALE
SCOTTSDALE
TEMPE
MESA
Chandler
Gilbert
Sun City
Apache Junction
SUPERSTITION FREEWAY
SALT RIVER INDIAN RESERVATION
GILA RIVER INDIAN RESERVATION
TONTO NATIONAL FOREST
MARICOPA COUNTY
PINAL COUNTY
SOUTH MOUNTAINS
GOLDFIELD MOUNTAINS
SUPERSTITION MOUNTAINS
SAN TAN MOUNTAINS
WILLIAMS AF BASE
Florence Jct.
Superior
Sacaton

PLATE 14

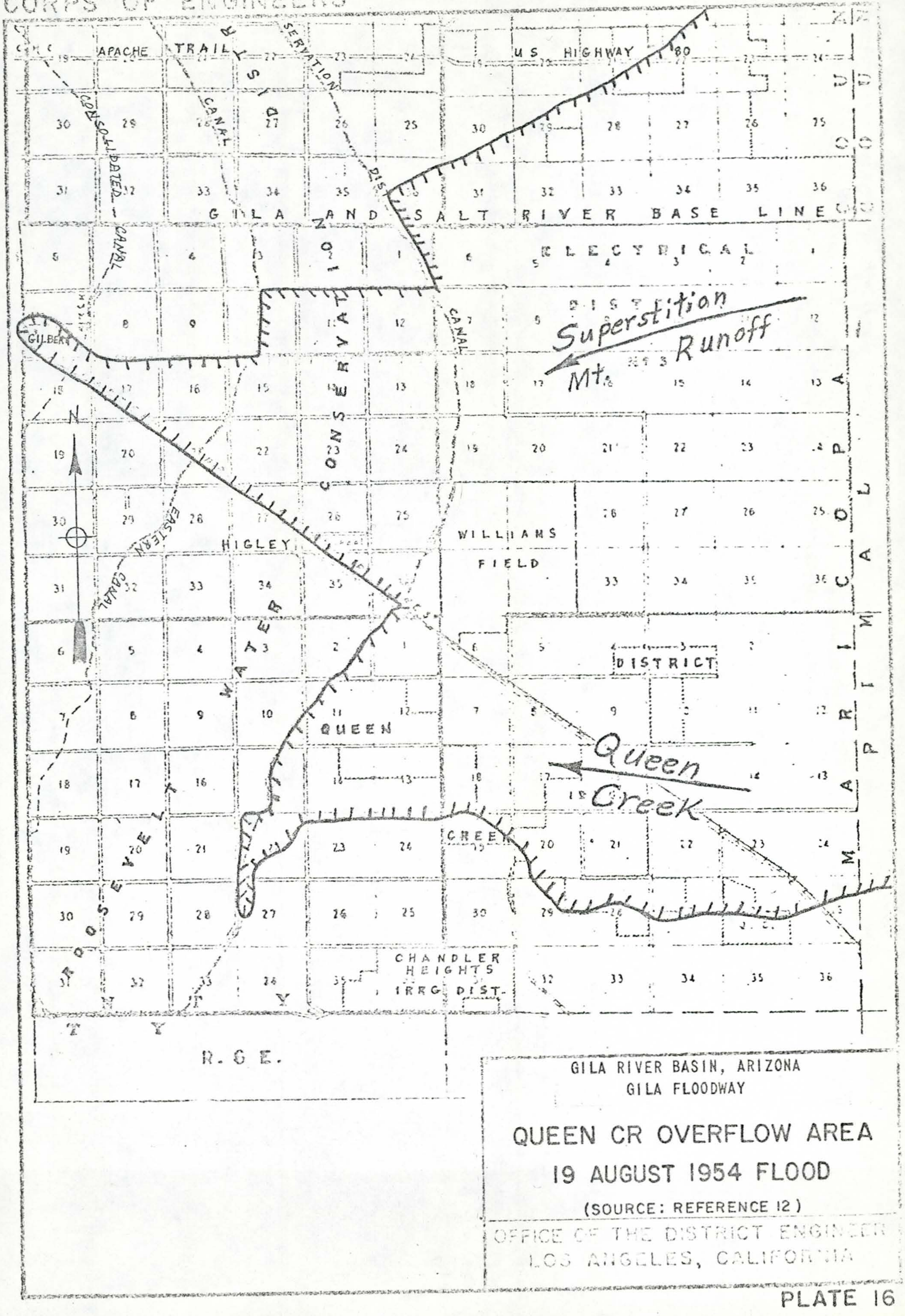


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SUPERSTITION FREEWAY DRAINAGE SYSTEM

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

PLATE 15

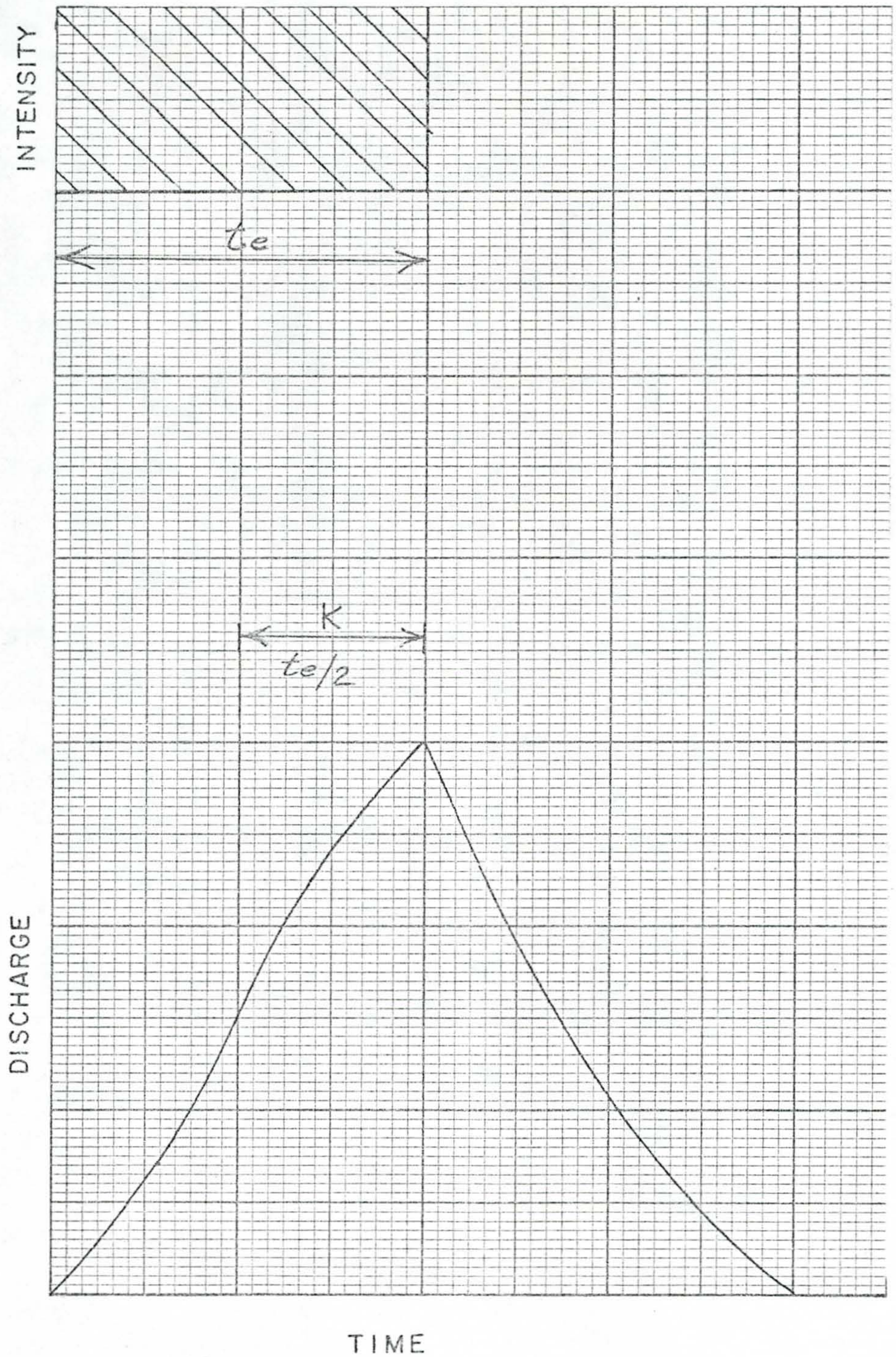


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

QUEEN CR OVERFLOW AREA
19 AUGUST 1954 FLOOD

(SOURCE: REFERENCE 12)

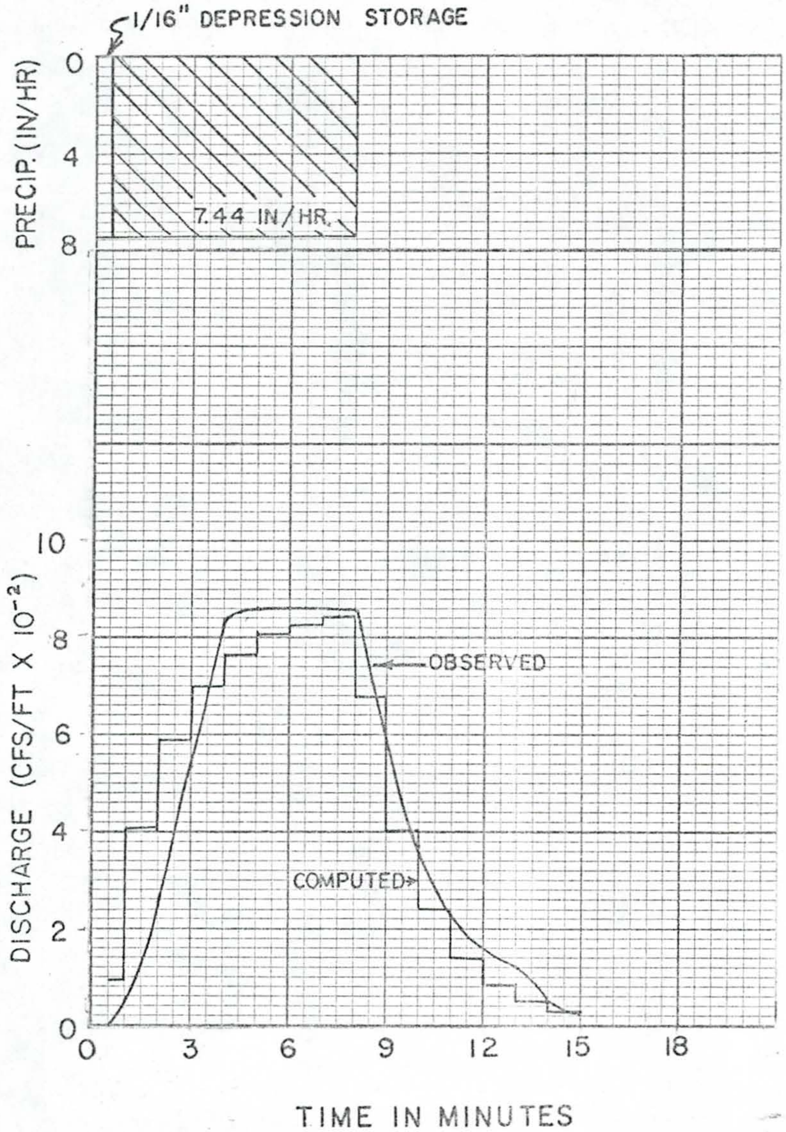
OFFICE OF THE DISTRICT ENGINEER
LOS ANGELES, CALIFORNIA



GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

DETERMINATION OF COEFFICIENT K

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



SURFACE = CONCRETE

n = .012

SLOPE = .02

L = 500 ft.

Te = 3.59 min.

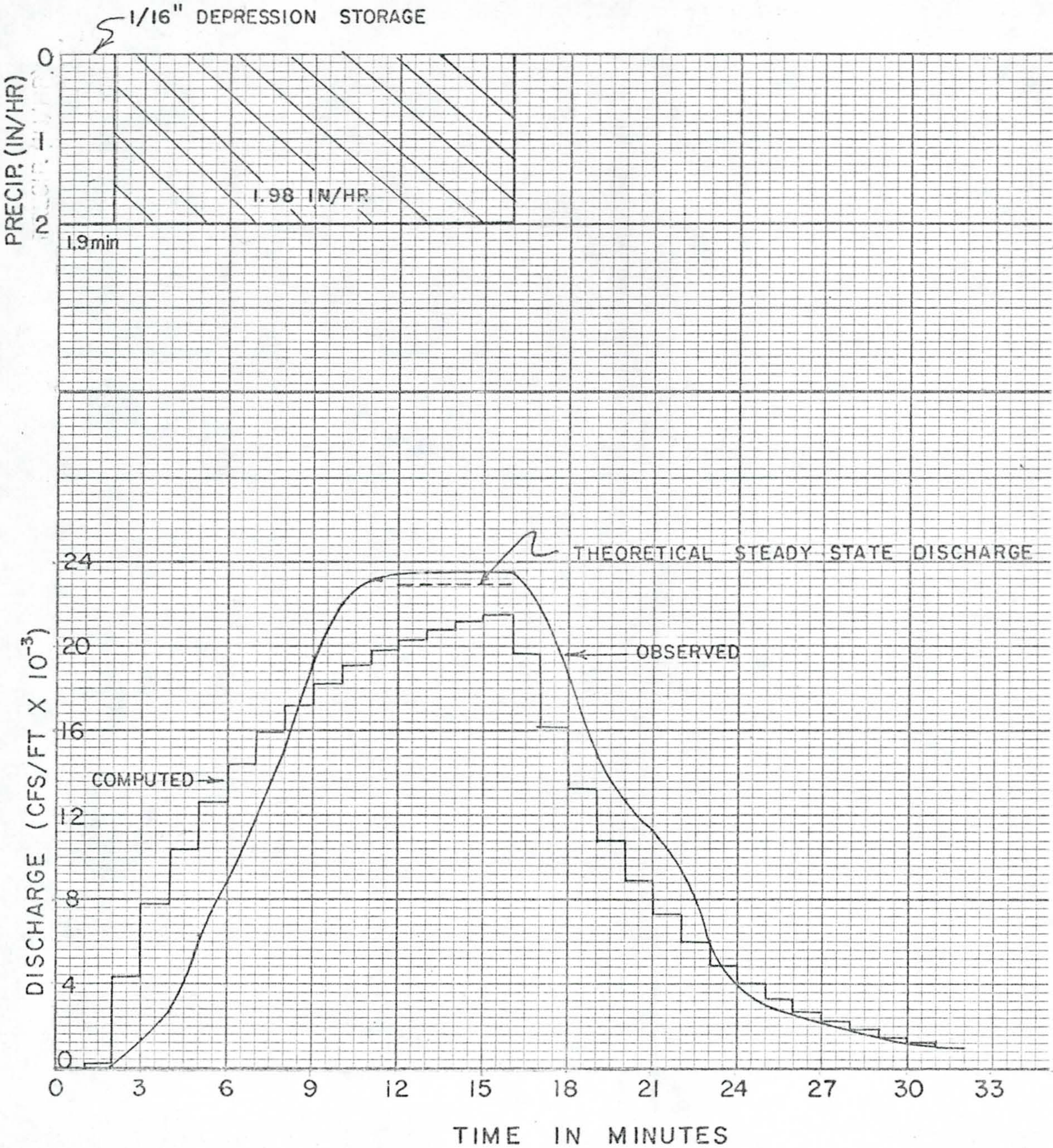
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

RECONSTITUTION

CASE I

(SOURCE: REFERENCE 17)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



SURFACE = CONCRETE

n = .012

SLOPE = .005

L = 500 ft.

Te = 10.16 min

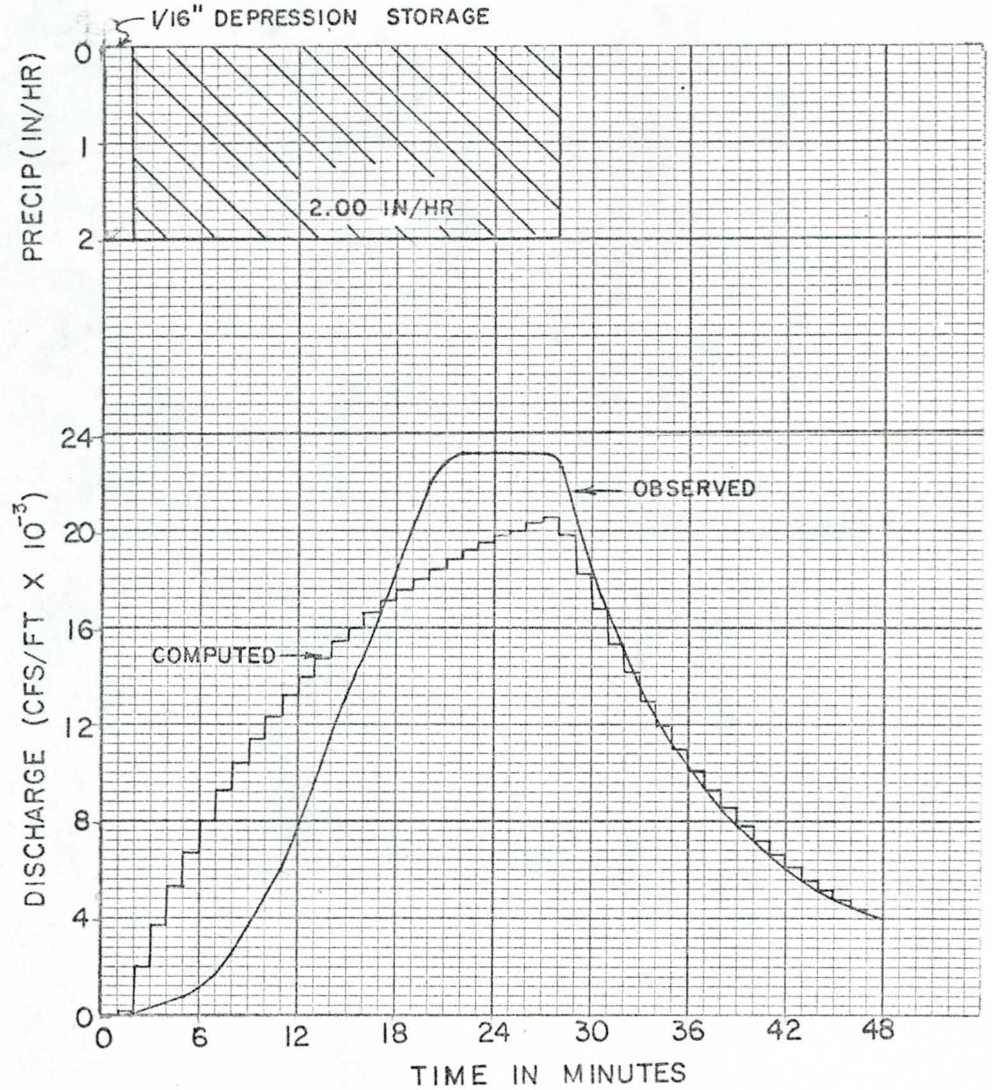
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

RECONSTITUTION

CASE II

(SOURCE: REFERENCE 17)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



SURFACE = SIMULATED TURF

n = .05

SLOPE = .005

L = 500 ft

Te = 23.83 min.

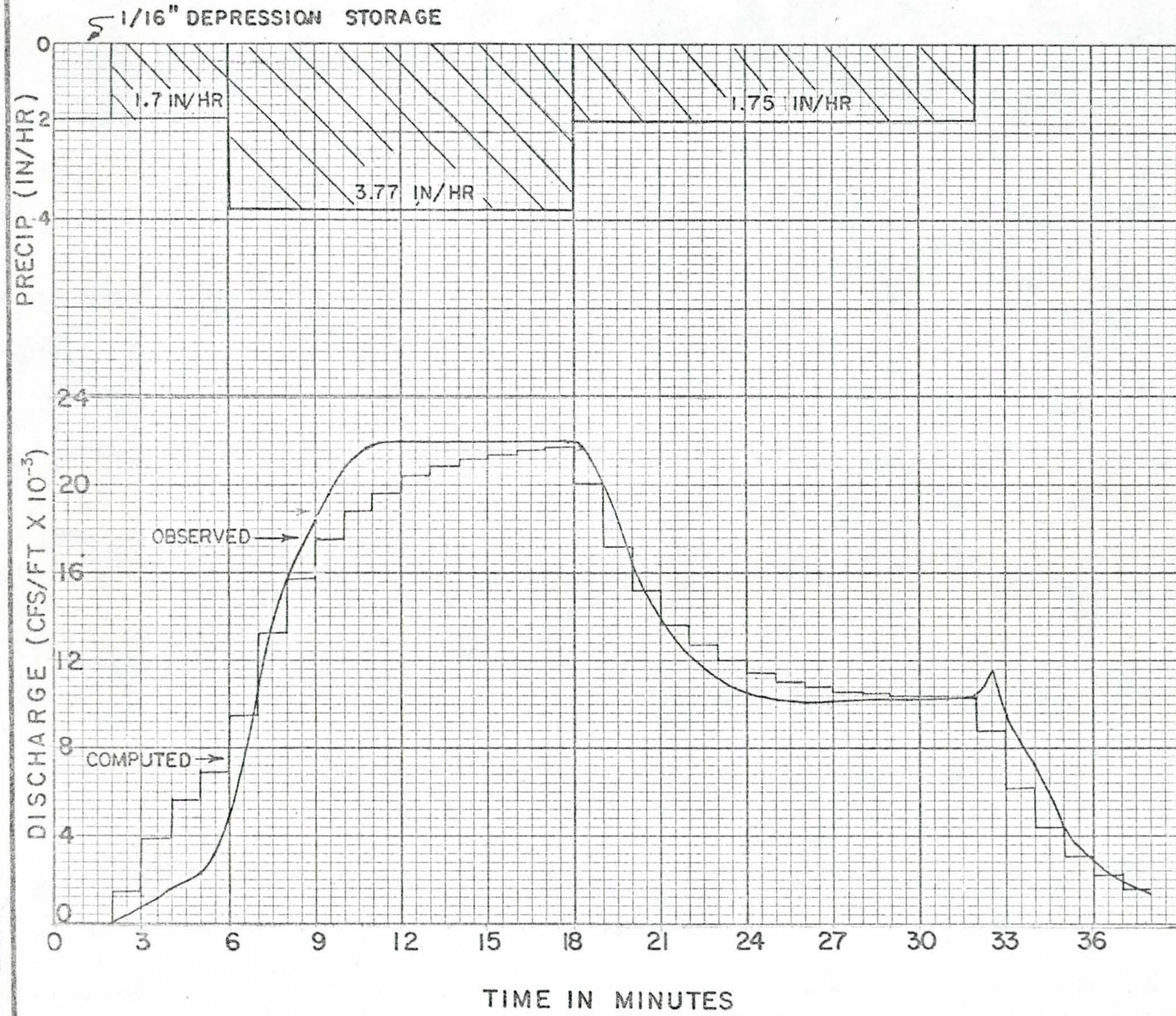
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

RECONSTITUTION

CASE III

(SOURCE: REFERENCE 17)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



SURFACE = CONCRETE

n = .012

SLOPE = .005

L = 252 ft

Te = 7.31 min

GILA RIVER BASIN, ARIZONA

GILA FLOODWAY

RECONSTITUTION

CASE IV

(SOURCE: REFERENCE 17)

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

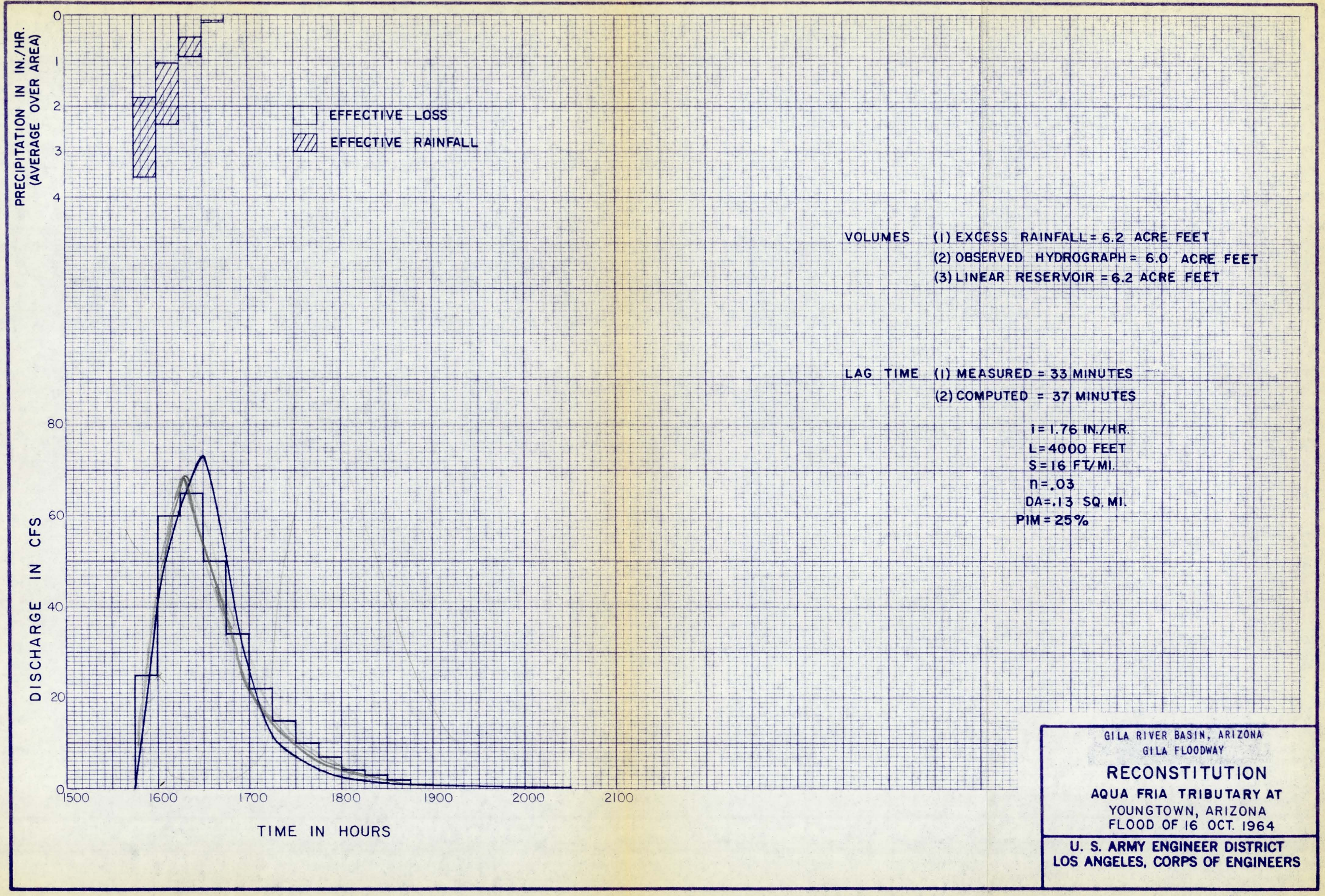
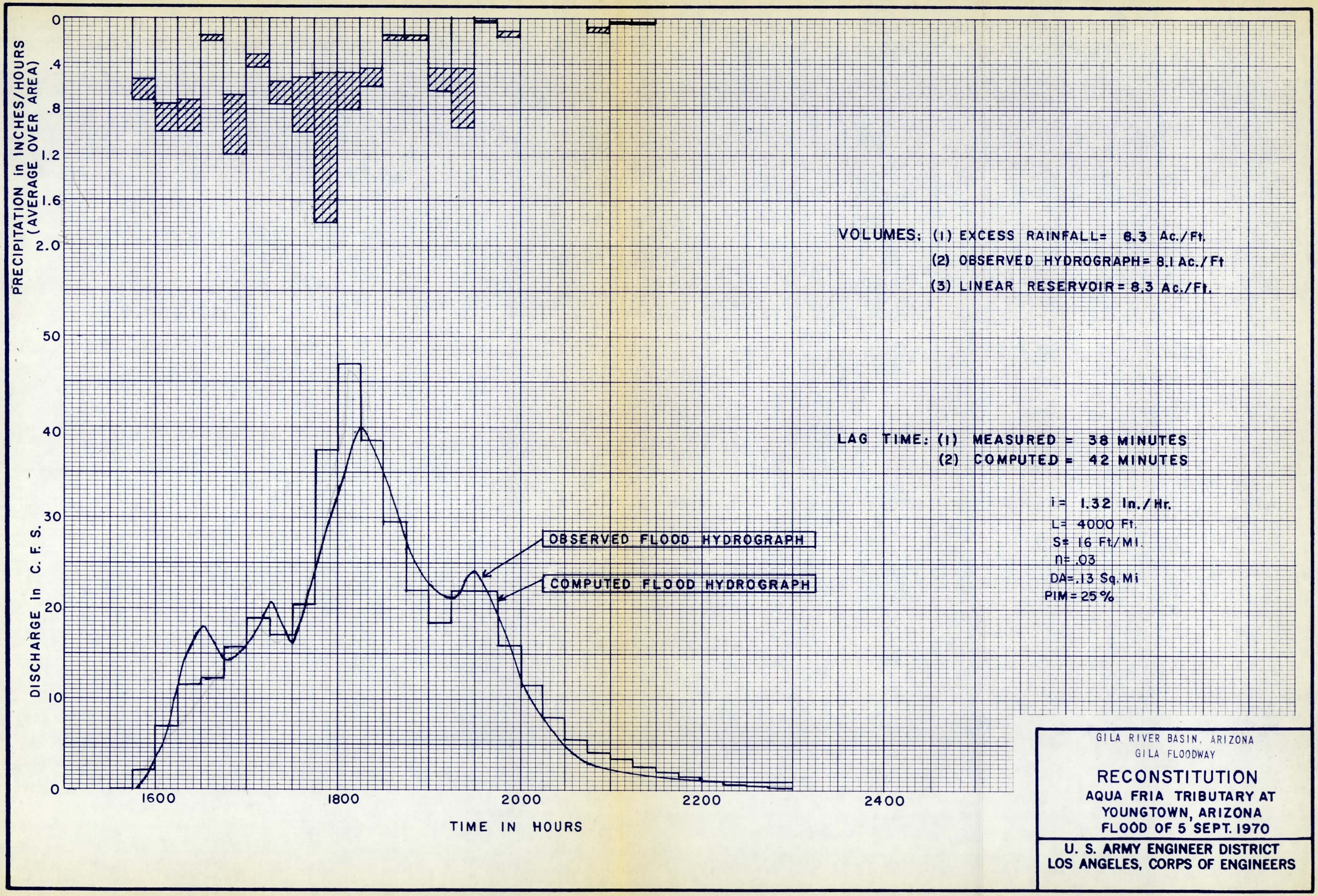
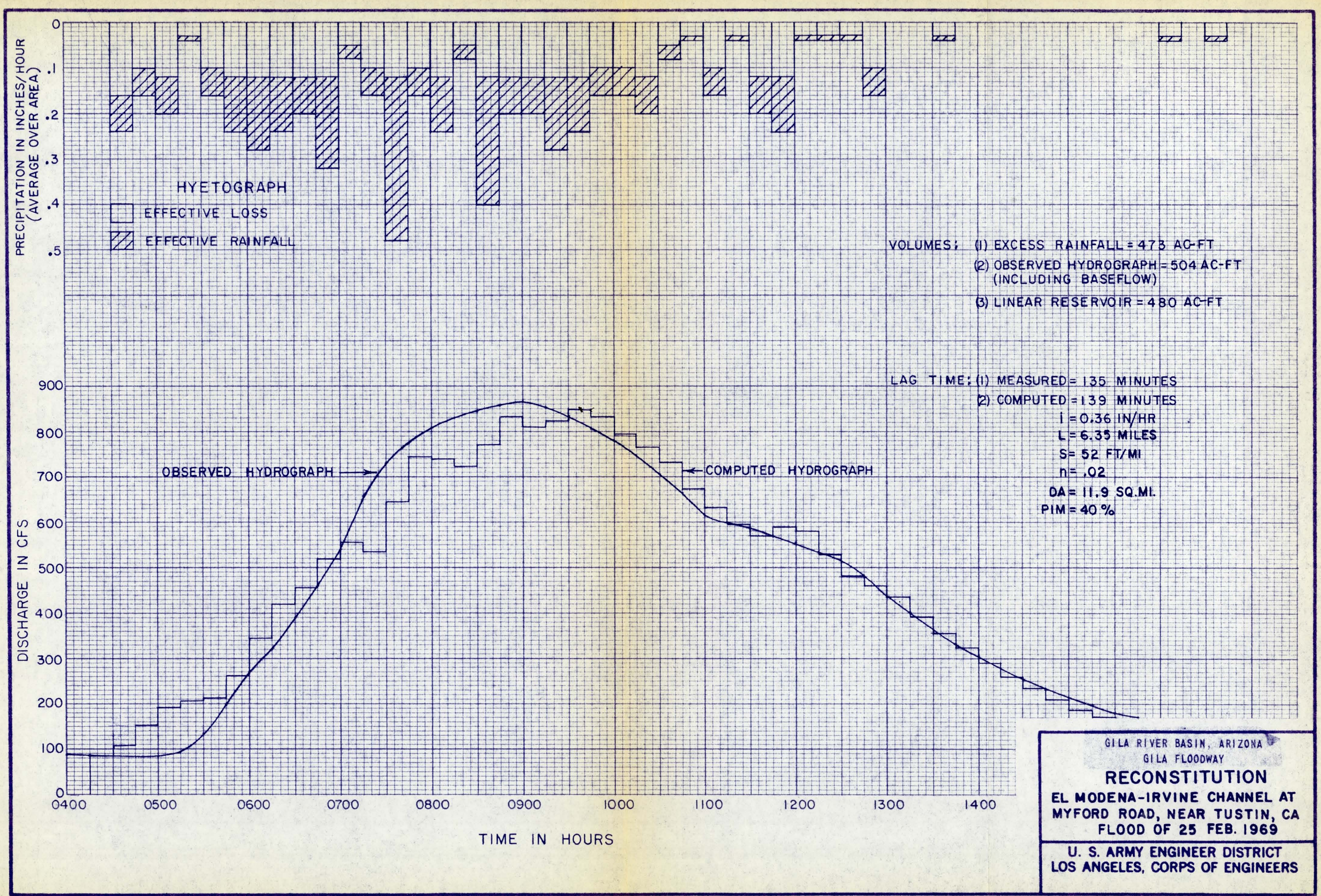

PRECIPITATION IN IN./HR. (AVERAGE OVER AREA)
EFFECTIVE LOSS
EFFECTIVE RAINFALL
VOLUMES (1) EXCESS RAINFALL = 6.2 ACRE FEET
(2) OBSERVED HYDROGRAPH = 6.0 ACRE FEET
(3) LINEAR RESERVOIR = 6.2 ACRE FEET
LAG TIME (1) MEASURED = 33 MINUTES
(2) COMPUTED = 37 MINUTES
i = 1.76 IN./HR.
L = 4000 FEET
S = 16 FT/MI.
n = .03
DA = .13 SQ. MI.
PIM = 25%
DISCHARGE IN CFS
TIME IN HOURS
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY
RECONSTITUTION
AQUA FRIA TRIBUTARY AT
YOUNGTOWN, ARIZONA
FLOOD OF 16 OCT. 1964
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

PLATE 22


PRECIPITATION in INCHES/HOURS (AVERAGE OVER AREA)
0
.4
.8
1.2
1.6
2.0
DISCHARGE In C. F. S.
50
40
30
20
10
0
1600
1800
2000
2200
2400
TIME IN HOURS
VOLUMES: (1) EXCESS RAINFALL= 8.3 Ac./Ft.
(2) OBSERVED HYDROGRAPH= 8.1 Ac./Ft
(3) LINEAR RESERVOIR= 8.3 Ac./Ft.
LAG TIME: (1) MEASURED = 38 MINUTES
(2) COMPUTED = 42 MINUTES
i= 1.32 In./Hr.
L= 4000 Ft.
S= 16 Ft./MI.
n= .03
DA= .13 Sq. Mi
PIM = 25 %
OBSERVED FLOOD HYDROGRAPH
COMPUTED FLOOD HYDROGRAPH
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY
RECONSTITUTION
AQUA FRIA TRIBUTARY AT
YOUNGTOWN, ARIZONA
FLOOD OF 5 SEPT. 1970
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

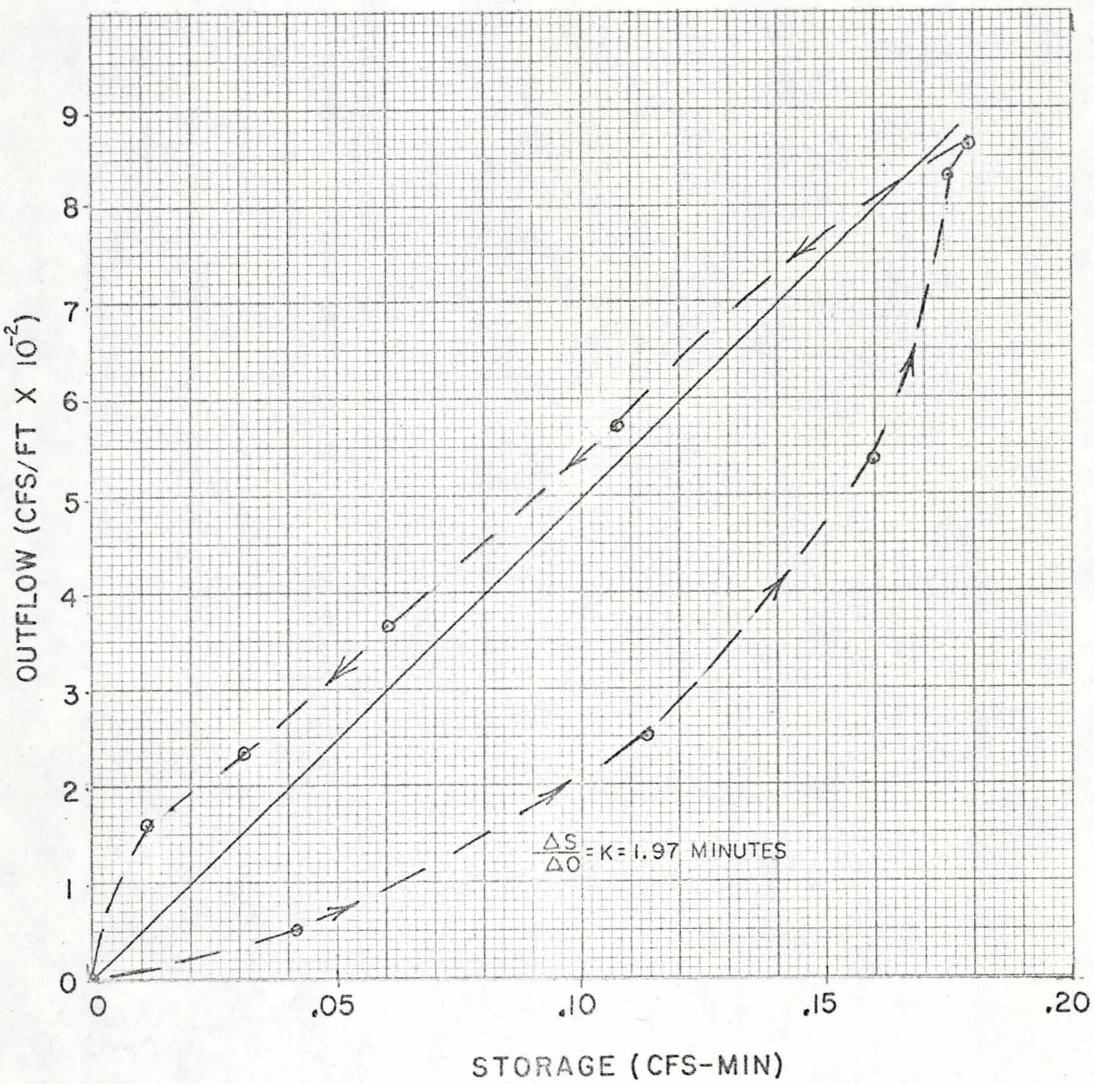


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

RECONSTITUTION

EL MODENA-IRVINE CHANNEL AT MYFORD ROAD, NEAR TUSTIN, CA FLOOD OF 25 FEB. 1969

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

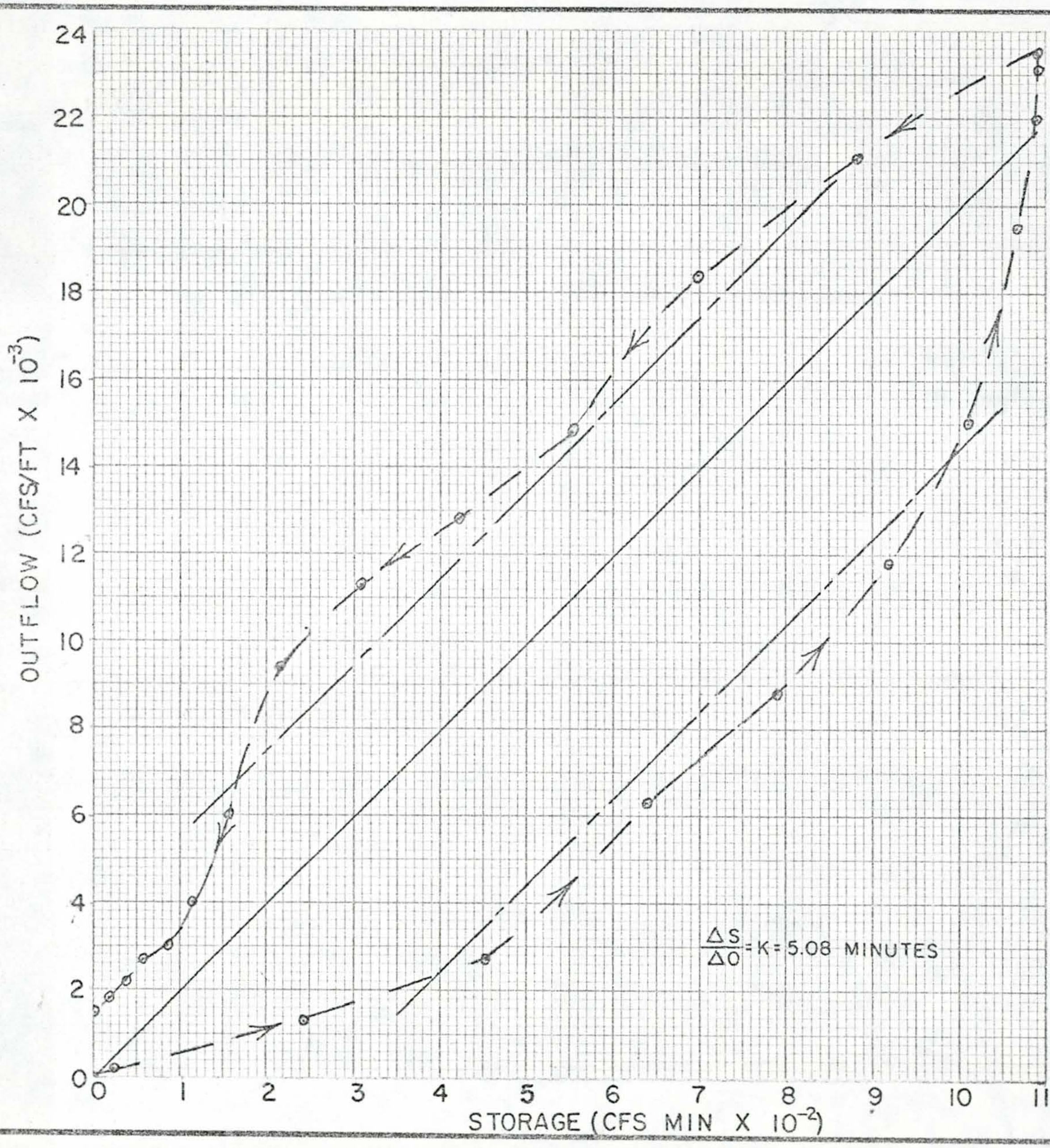


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

STORAGE-OUTFLOW LOOP
CASE I

(SOURCE: REFERENCE 17)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

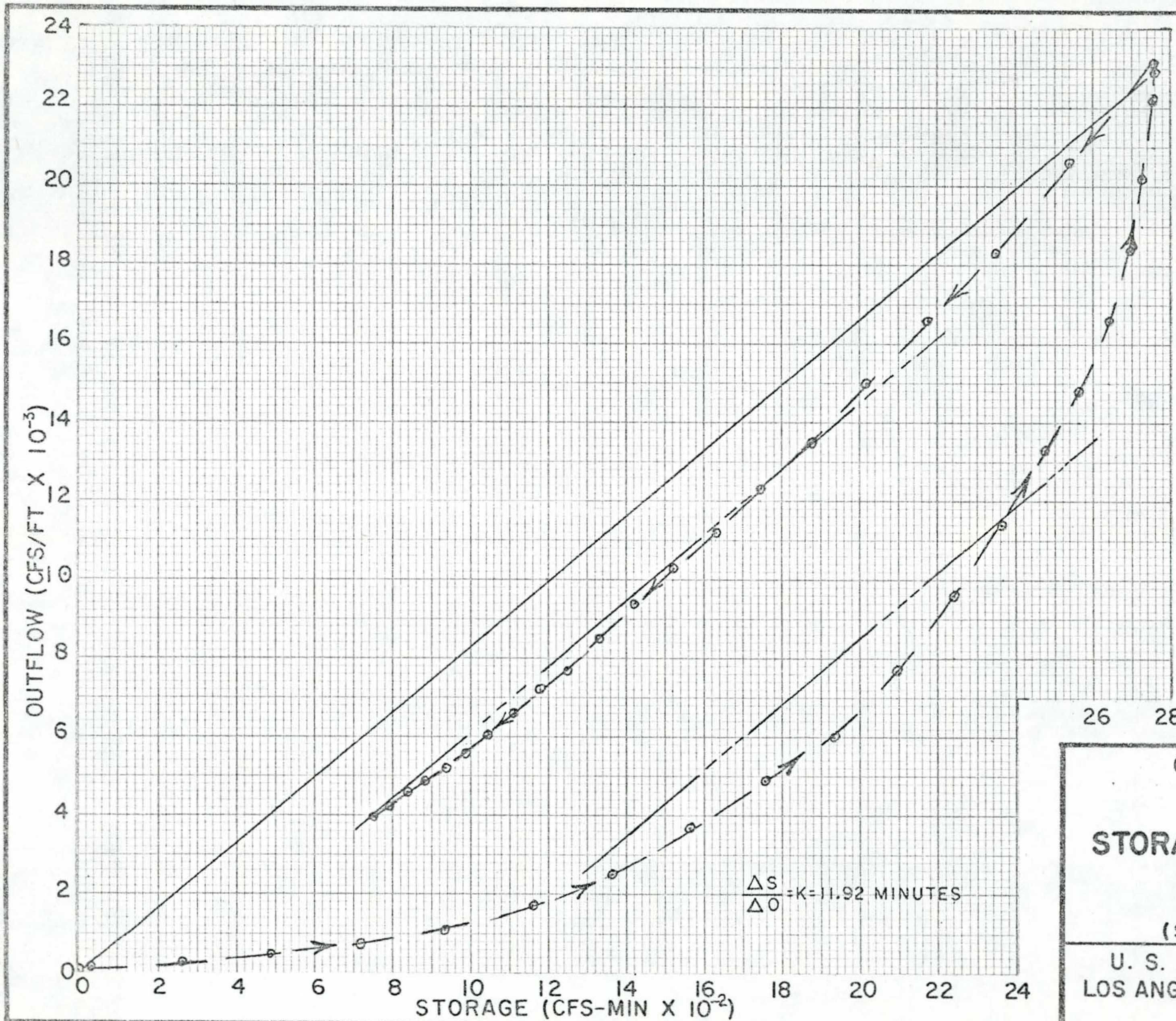


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

STORAGE-OUTFLOW LOOP
CASE II

(SOURCE: REFERENCE 17)

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



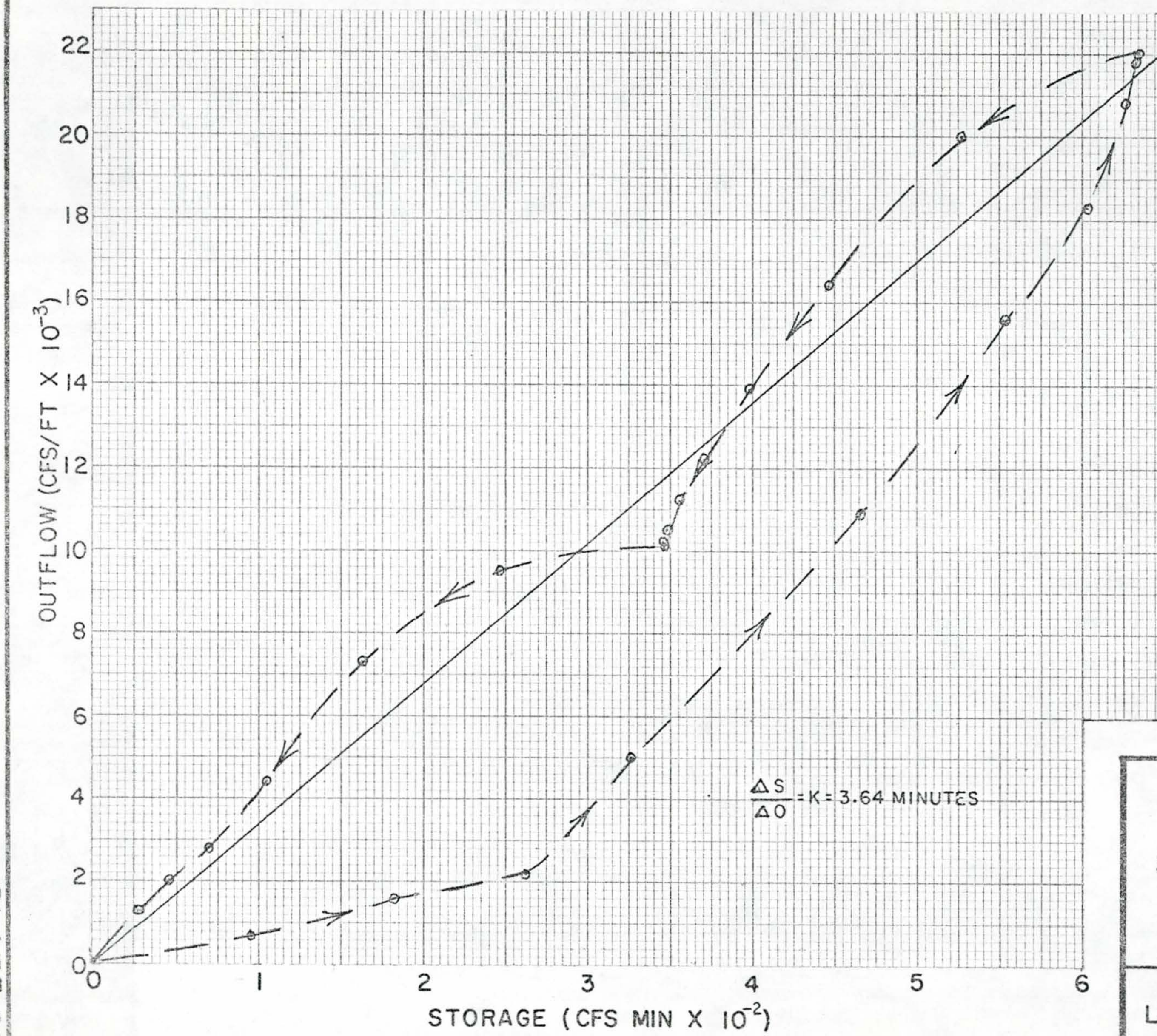
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

STORAGE - OUTFLOW LOOP
CASE III

(SOURCE: REFERENCE 17)

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

PLATE 27

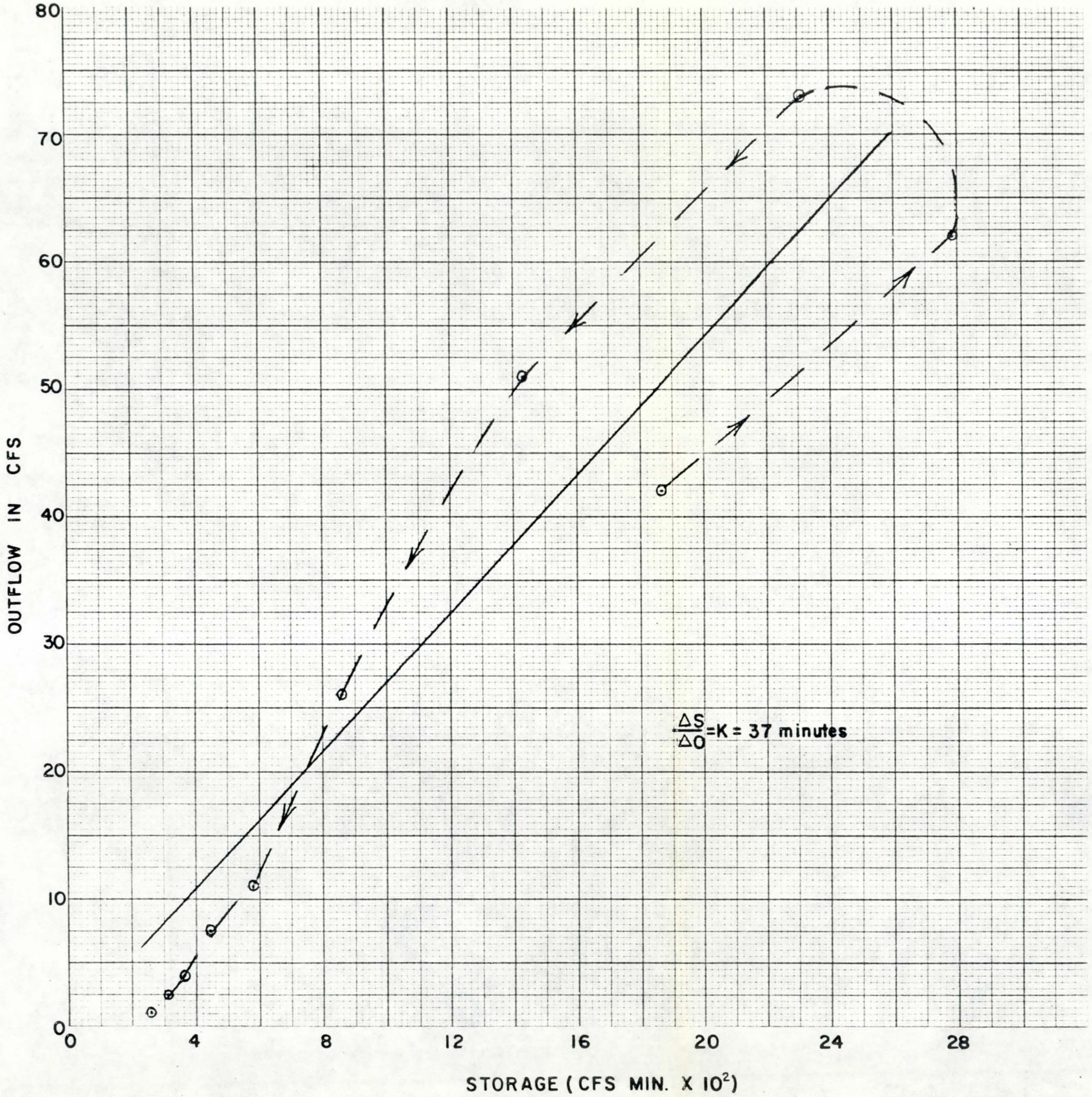


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

STORAGE-OUTFLOW LOOP
CASE IV

(SOURCE: REFERENCE 17)

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

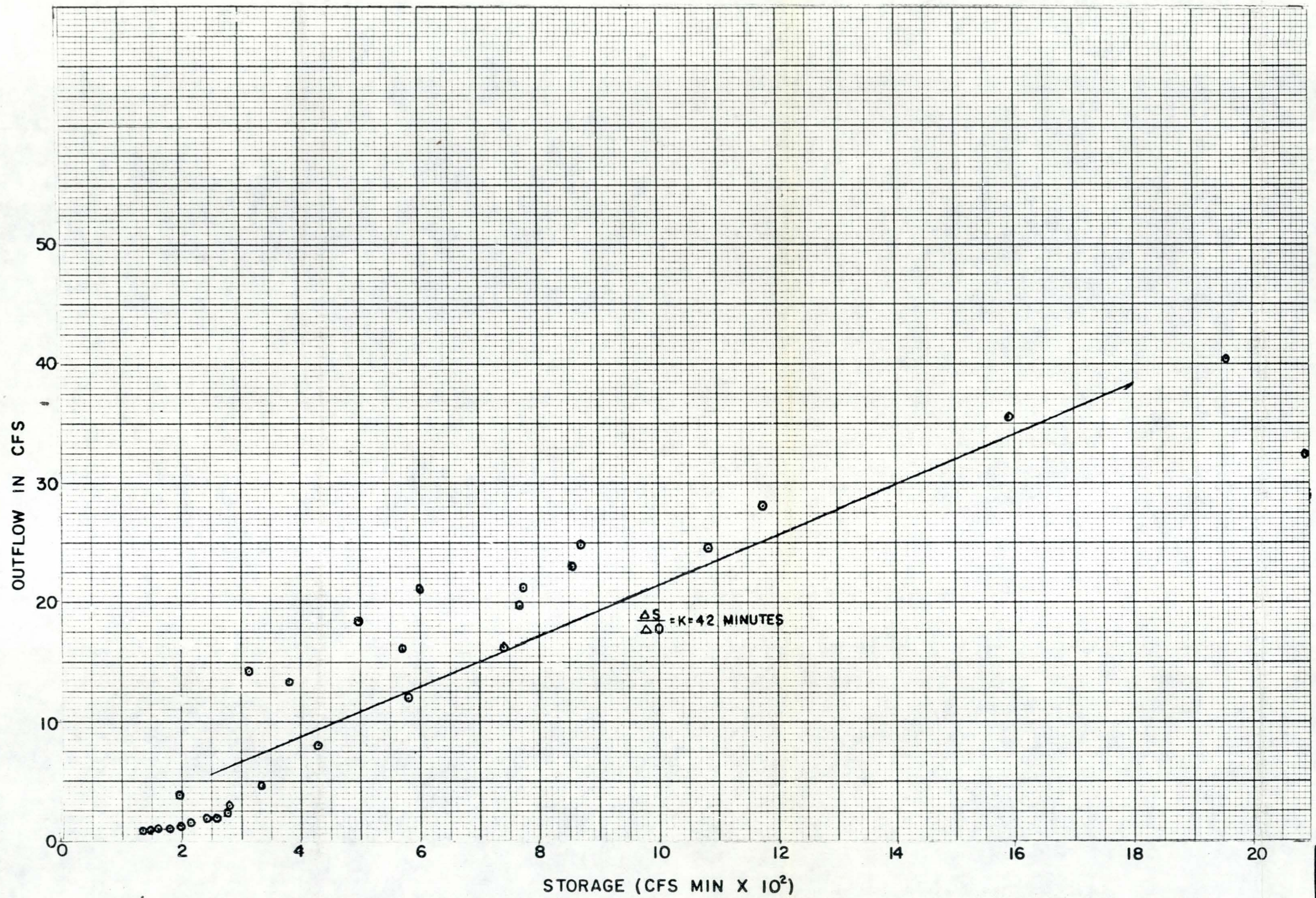


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

STORAGE-OUTFLOW LOOP

AQUA FRIA TRIBUTARY AT
YOUNGTOWN, ARIZONA
FLOOD OF 16 OCT. 1964

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



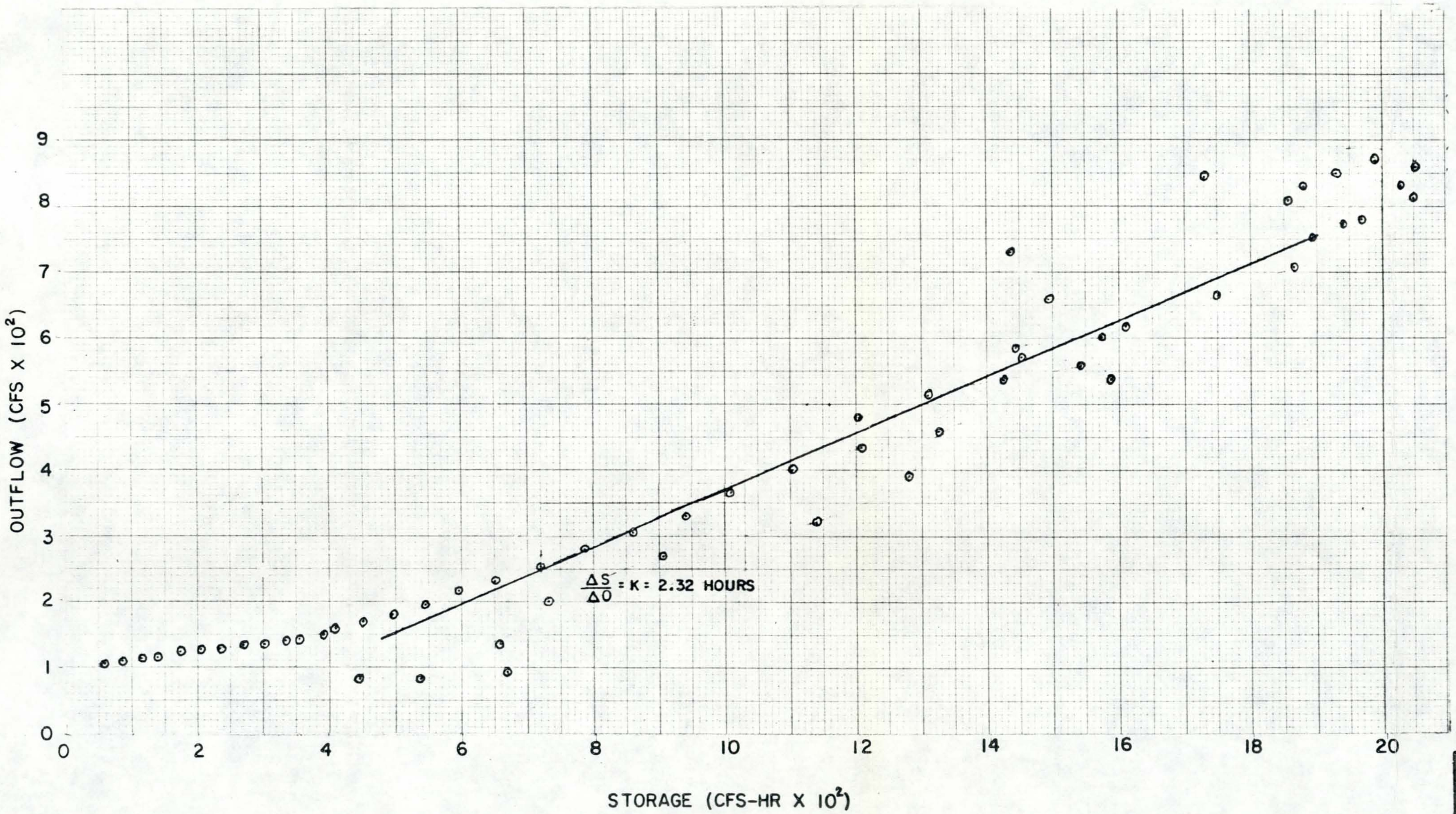
GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

STORAGE-OUTFLOW LOOP

AQUA FRIA TRIBUTARY AT YOUNGTOWN, ARIZONA

FLOOD OF 5 SEPT. 1970

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

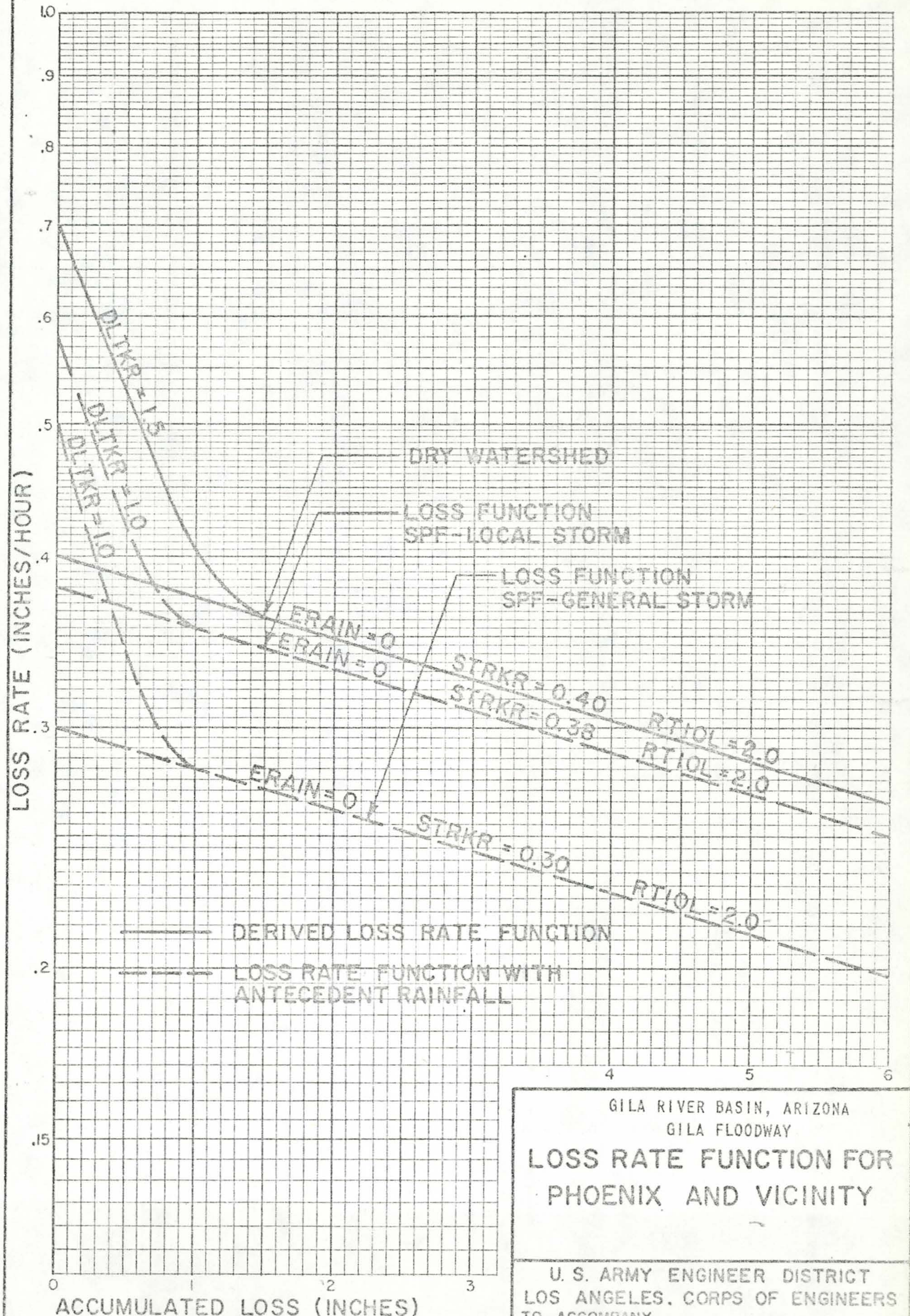


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

STORAGE-OUTFLOW LOOP

EL MODENA IRVINE CHANNEL AT MYFORD ROAD, NEAR TUSTIN, CA. FLOOD OF 25 FEB. 1969

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



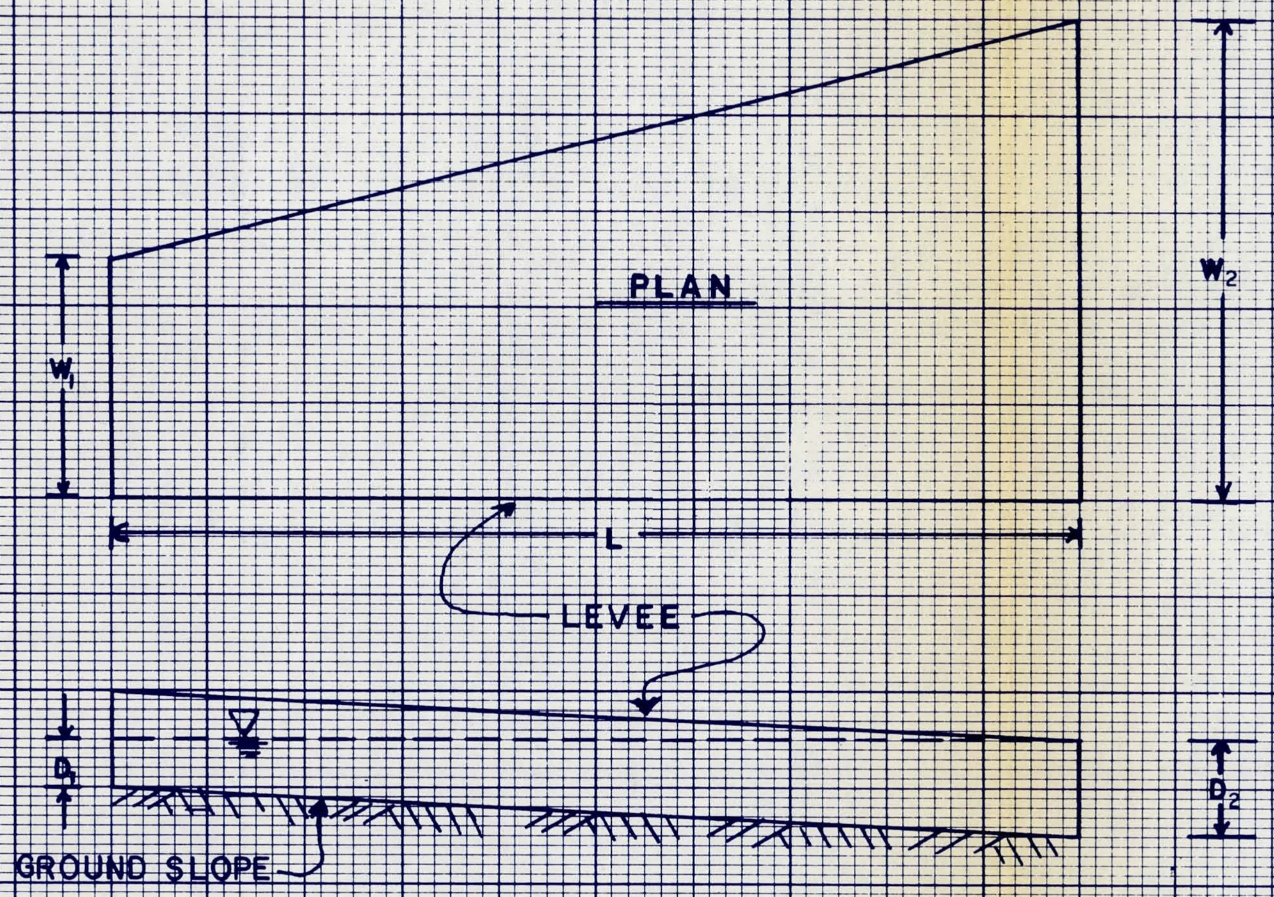
GILA RIVER BASIN, ARIZONA

GILA FLOODWAY

LOSS RATE FUNCTION FOR PHOENIX AND VICINITY

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

THIS PLATE ILLUSTRATES THE DETERMINATION OF VOLUME OF STORAGE BEHIND CANAL LEVEES.



FRONT ELEVATION

VOLUME

W, D and L in FEET.

AVERAGE END-AREA METHOD.
VOLUME in ACRE-FEET.

$$V = \frac{\left\{\left[\dfrac{\left(\frac{1}{2} W_1 D_1 + \frac{1}{2} W_2 D_2\right)}{2}\right] L\right\}}{43{,}560}$$

$$V = \left[\frac{(W_1 D_1 + W_2 D_2)\, L}{174{,}240}\right]$$

e.g. $W_1 = 250'$ $\quad W_2 = 500'$ $\quad L = 5{,}000'$

$D_1 = 1'$ $\quad D_2 = 2'$

$$V = \frac{\left\{\left[250\,(1) + 500\,(2)\right]\; 5{,}000\right\}}{174{,}240} = 35.87 \text{ ACRE-FEET.}$$

RIGHT ELEVATION

GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

LEVEE STORAGE WEDGE

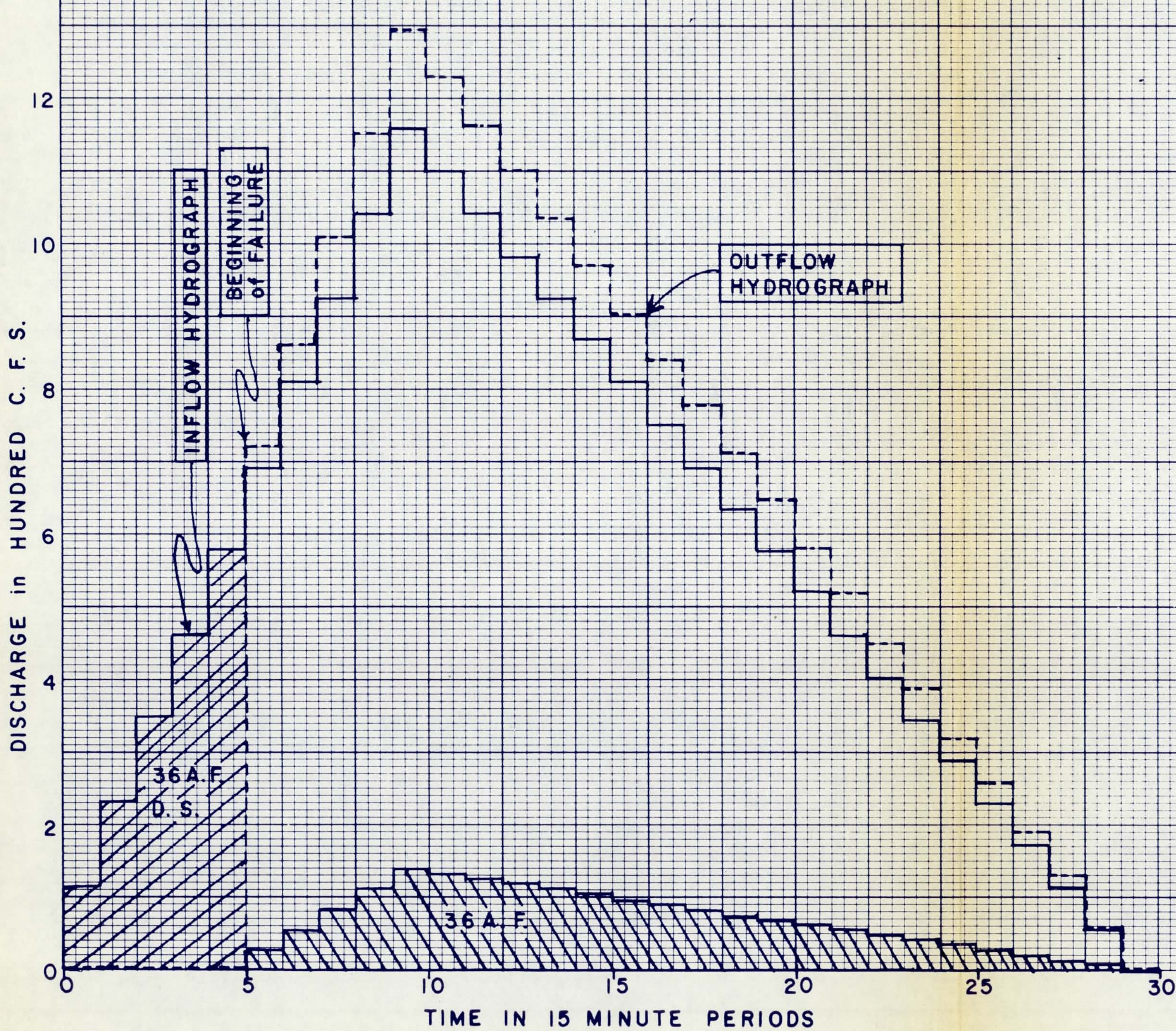
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

TYPICAL CANAL LEVEE FAILURE FLOOD HYDROGRAPH ROUTING

$$A.F.\left(\frac{726}{U.T.}\right) = C.F.S.-PERIOD \quad e.g. \quad 35.87\left(\frac{726}{15}\right) = 1736.11\ C.F.S.-$$

15 MINUTE (DEAD STORAGE)

$$RATIO: \frac{360}{360-36} = 1.1111 \text{ or Increase of } 11.11\% \text{ (CONSTANT)}$$



PERIOD	AVERAGE INFLOW CFS	ACCUMULATION CFS - PERIOD	INCREMENT DISTRIBUTION FACTOR	INCREMENT CFS	AVERAGE OUTFLOW CFS	PERCENT INCREASE
1	116.16	116.16)			0	
2	232.32	348.48)			0	
3	348.48	696.96) D.S.			0	
4	464.64	1161.60)			0	
5	580.80	1742.40)	0	0	0	
6	696.96		0.20	27.88	724.84	4.00
7	813.12		0.40	55.76	868.88	6.86
8	929.28		0.60	83.63	1012.91	9.00
9	1045.44		0.80	111.51	1156.95	10.67
10	1161.60	PEAK	1.00	139.39	1300.99	12.00
11	1103.52		0.95	132.42	1235.94	12.00
12	1045.44		0.90	125.45	1170.89	12.00
13	987.36		0.85	118.48	1105.84	12.00
14	929.28		0.80	111.51	1040.79	12.00
15	871.20		0.75	104.54	975.74	12.00
16	813.12		0.70	97.57	910.69	12.00
17	755.04		0.65	90.60	845.64	12.00
18	696.96		0.60	83.63	780.59	12.00
19	638.88		0.55	76.66	715.54	12.00
20	580.80		0.50	69.70	650.50	12.00
21	522.72		0.45	62.73	585.45	12.00
22	464.64		0.40	55.76	520.40	12.00
23	406.56		0.35	48.79	455.35	12.00
24	348.48		0.30	41.82	390.30	12.00
25	290.40		0.25	34.85	325.25	12.00
26	232.32		0.20	27.88	260.20	12.00
27	174.24		0.15	20.91	195.15	12.00
28	116.16		0.10	13.94	130.10	12.00
29	58.08		0.05	6.97	65.05	12.00
30	0		0	0	0	
TOTAL	17424	1742.40	12.50	1742.38	17423.98	270.53
	360 A.F.	36 A.F.		36 A.F.	360 A.F.	10.82 Avs.

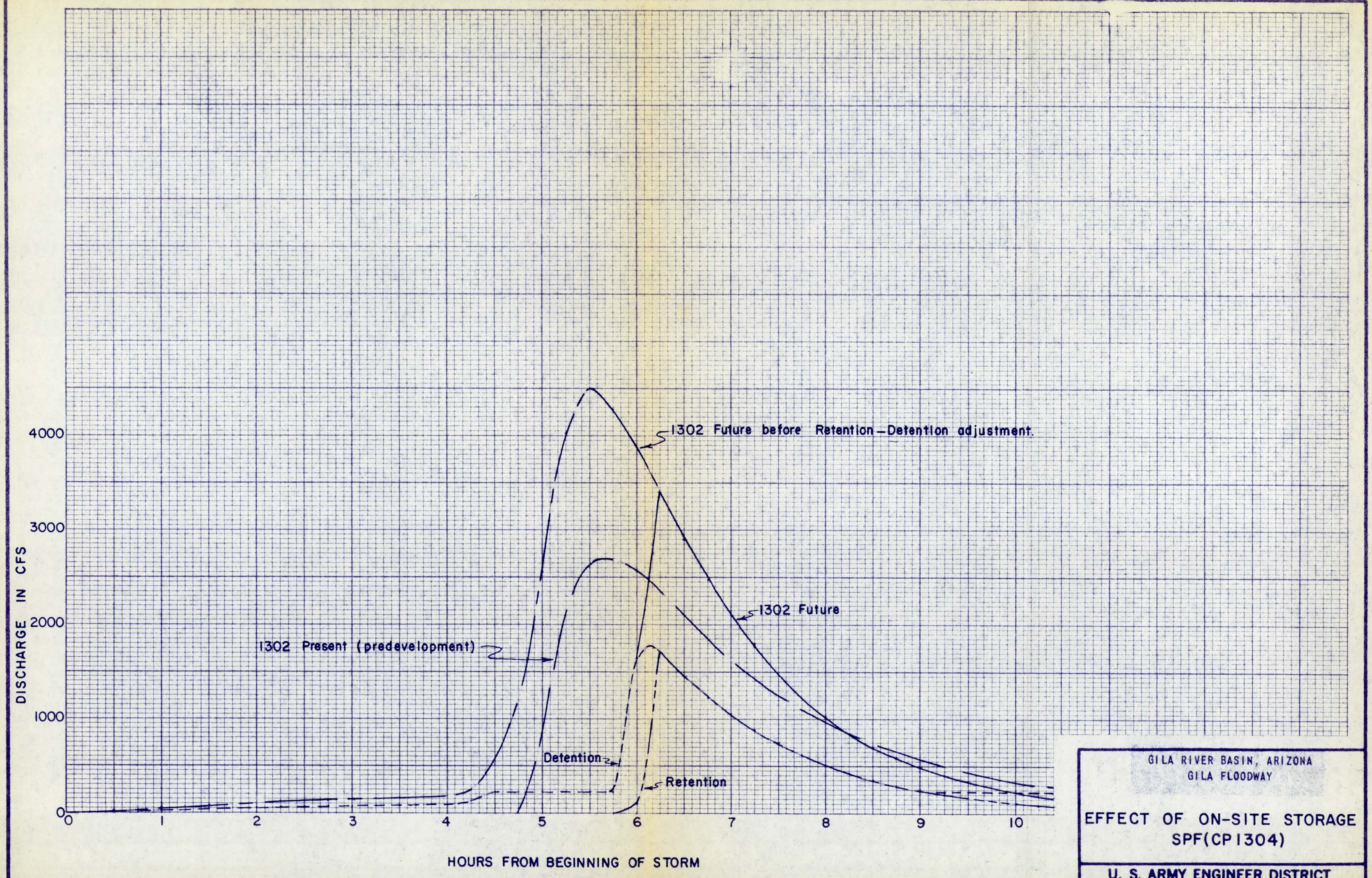
$$\frac{1}{30-10} = 0.05 \qquad \frac{1}{10-5} = 0.20$$

INCREMENT =
1742.40/12.50 = 139.39

GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

LEVEE FAILURE HYDROGRAPH

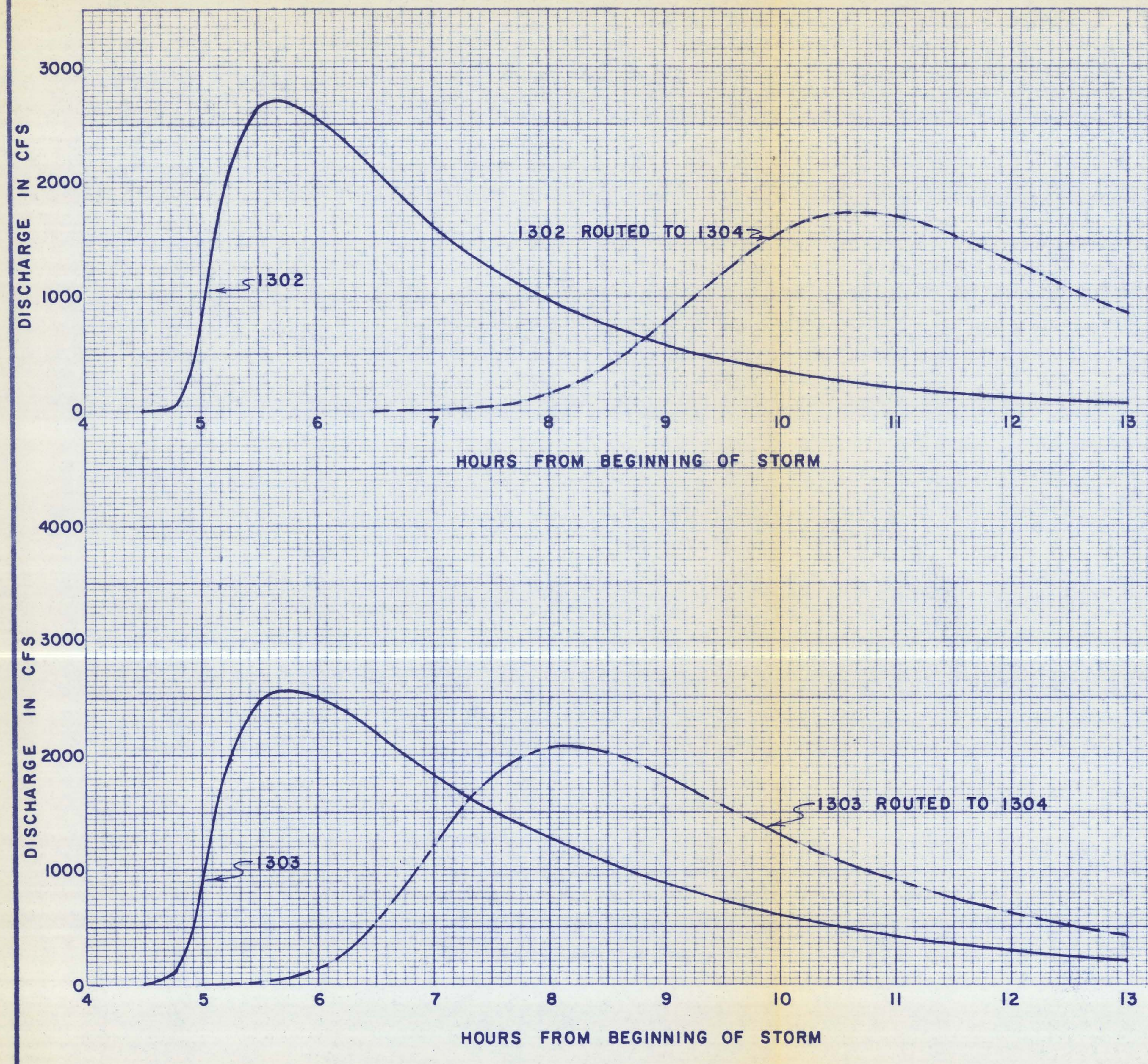
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

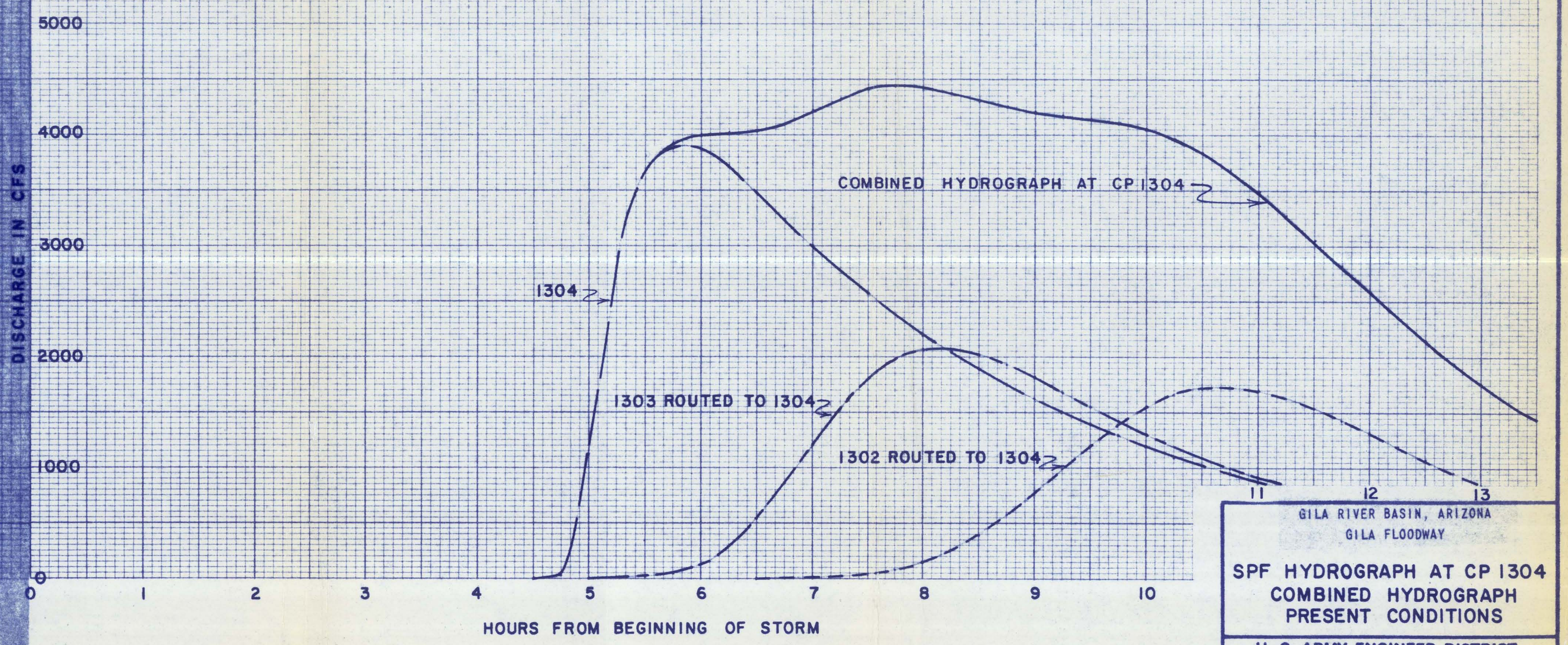


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

EFFECT OF ON-SITE STORAGE
SPF(CP 1304)

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



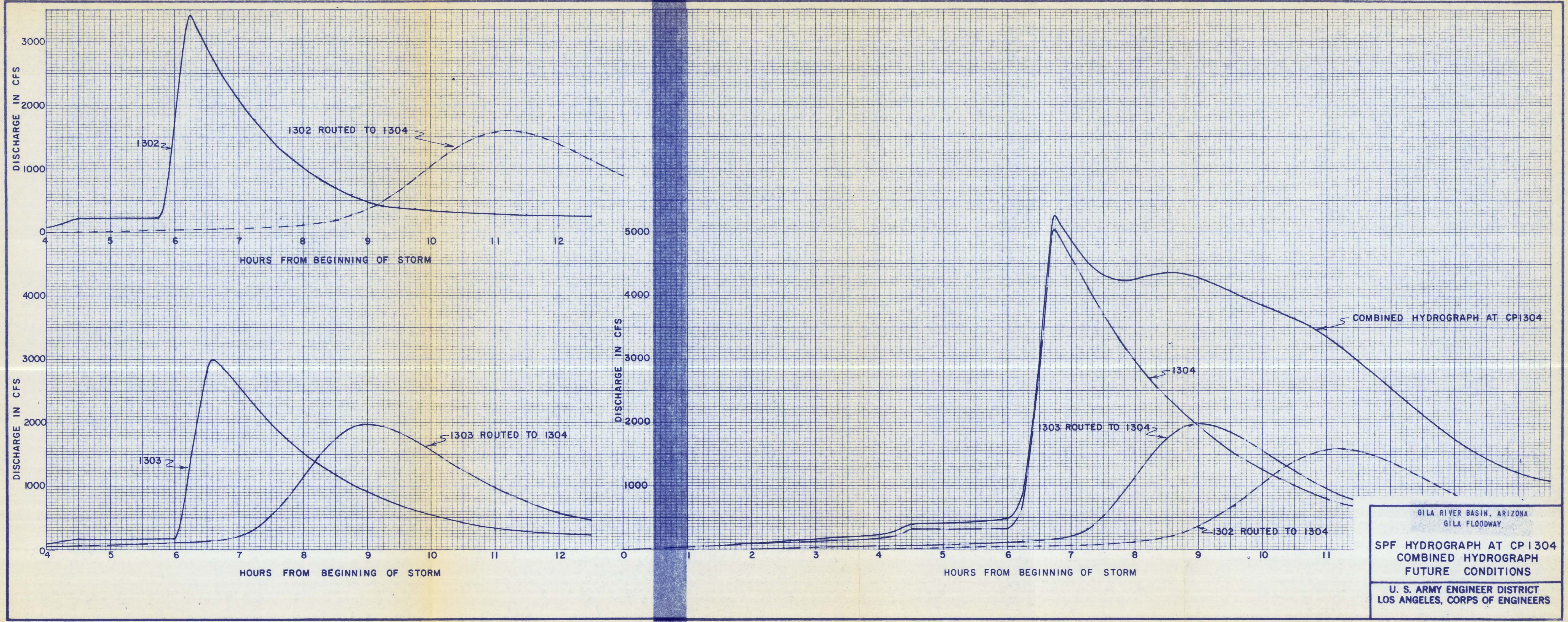


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SPF HYDROGRAPH AT CP 1304
COMBINED HYDROGRAPH
PRESENT CONDITIONS

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

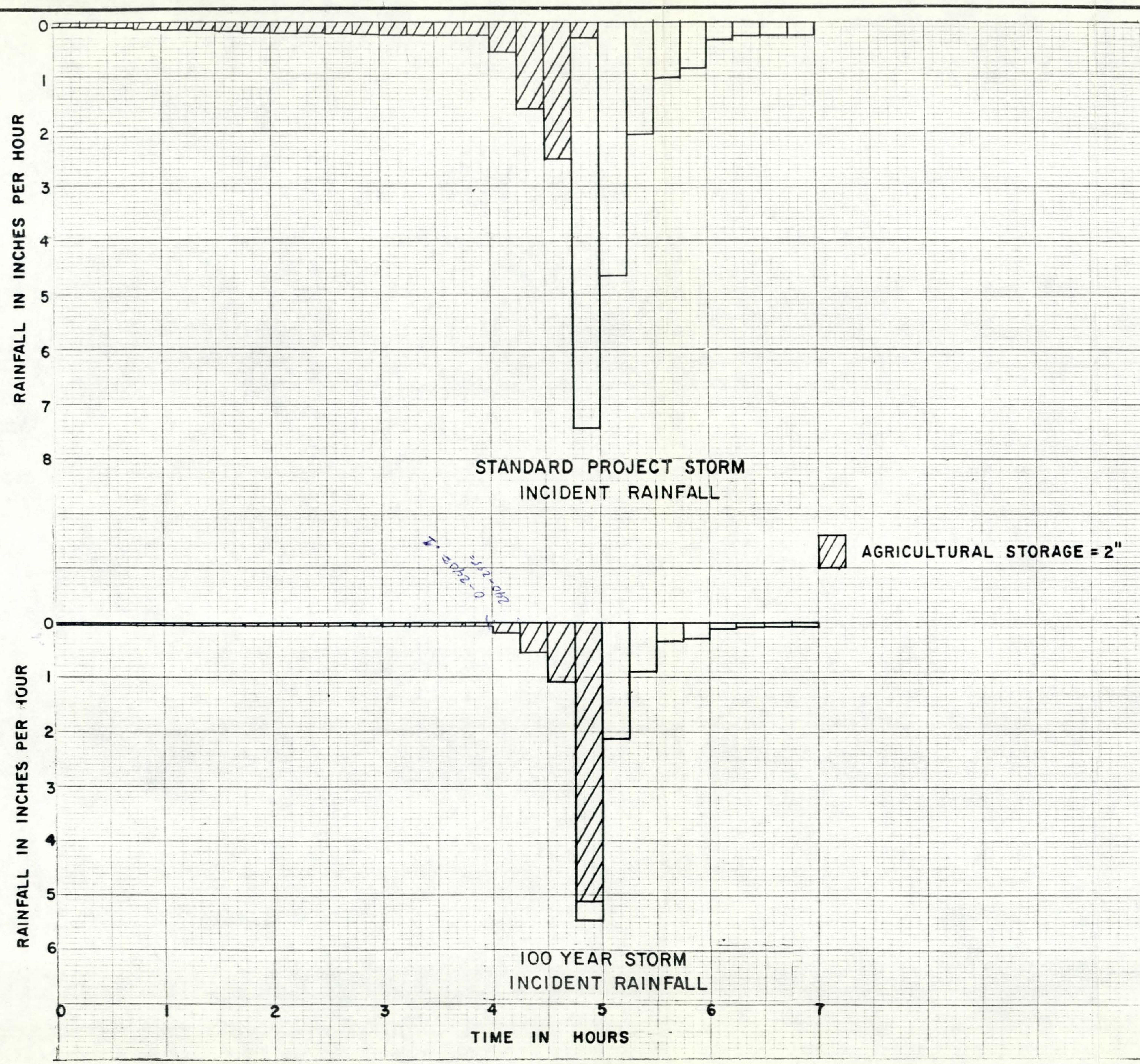
PLATE 36



GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

SPF HYDROGRAPH AT CP 1304
COMBINED HYDROGRAPH
FUTURE CONDITIONS

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

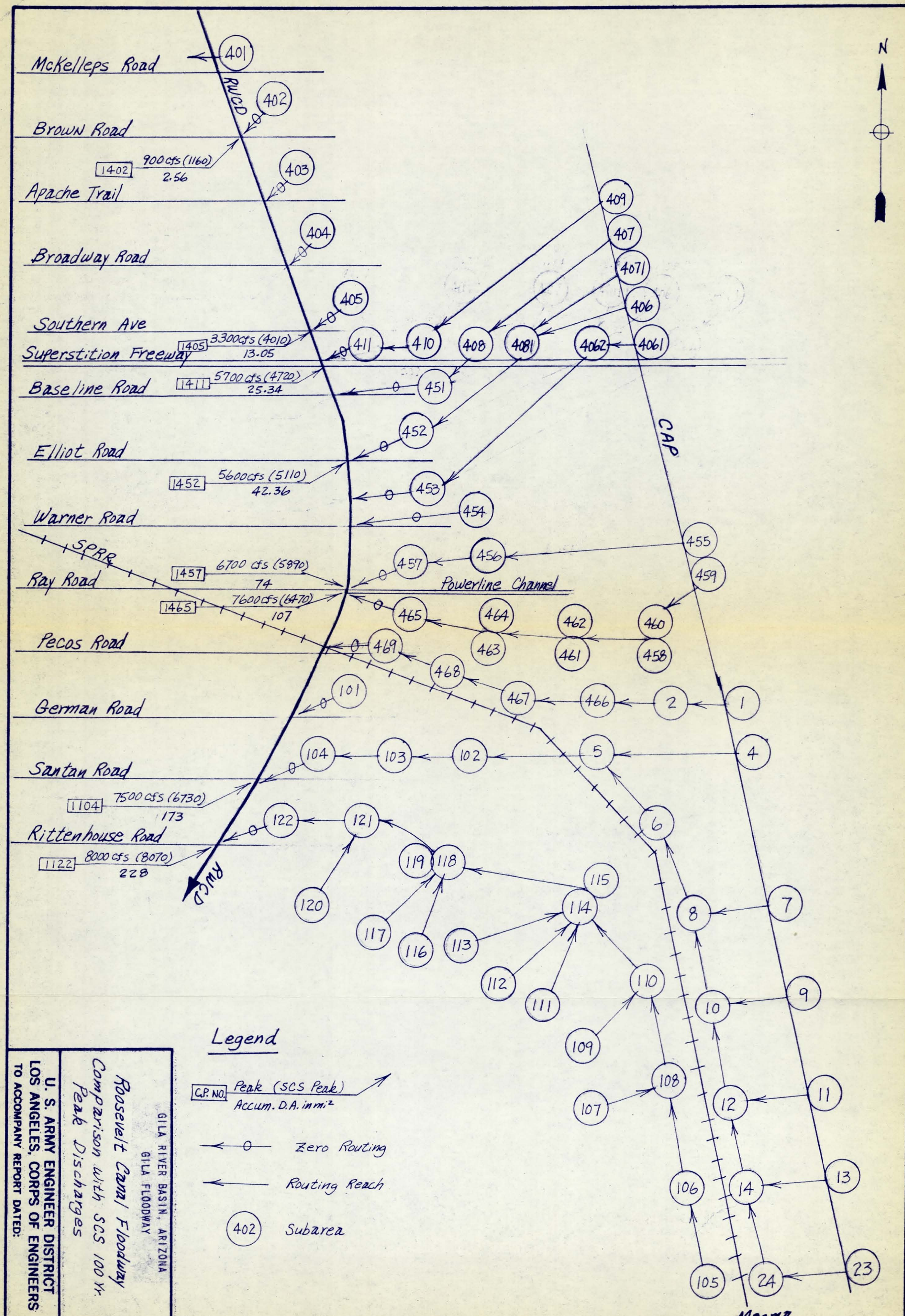


GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

POINT RAINFALL HYETOGRAPH

(SOURCE: REFERENCE 2)

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:



GILA RIVER BASIN, ARIZONA
GILA FLOODWAY

Roosevelt Canal / Floodway Comparison with SCS 100 Yr. Peak Discharges

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE 38 39

APPENDIX 2

Review and Evaluation of Rainfall-Runoff Model
by the Hydrologic Engineering Center
Including Comments by the Soil Conservation Service

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
6029 Federal Building, 230 North First Avenue, Phoenix, Arizona 85025

December 16, 1975

Mr. Garth A. Fuquay
Chief, Engineering Division
U.S. Army, Los Angeles District,
Corps of Engineers
P.O. Box 2711
Los Angeles, California 90053

Dear Mr. Fuquay:

We appreciate having had the opportunity to review the "Gila Floodway Survey Report, Hydrology, Part I," and offer the following comments for your consideration:

From our review, it appears that although the methodology as used by the SCS and the Corps of Engineers in evaluating the runoff potential of the study area differs considerably, comparable results have been obtained. This can be attributed in part to the continuous coordination efforts conducted by our staffs. Areas of general agreement include: present and projected land use, percent impervious by type and density of urbanization, velocity of overland flows, volume of depression storage on agricultural lands, and the effect of existing and proposed SCS projects on the runoff potential for the study area.

The only significant difference in our evaluations is in the treatment of onsite storage. We have reached agreement with the Maricopa County Flood Control District that one-inch of onsite storage will be required for all new land developed after 1975 for the area between the existing and proposed SCS floodwater retarding structures and the proposed RWCD Floodway. This requirement has been included as part of the Supplemental Work Plan Agreement for the Buckhorn-Mesa Watershed Project.

The only other comment with respect to the subject report is referenced to page 2, paragraph 2, of the HEC Special Project Memo No. 441, dated August 29, 1975. In this paragraph the statement is made that the relationship $K=0.5t_c$ "bears similarity to another relationship used in Soil Conservation procedures $tp=0.6t_c$." The equation $tp=0.6t_c$ is incorrect; but in conversation with Mr. John Pederson of your staff, it appears that the error is simply in the misuse of SCS standard symbols.

Mr. Garth A. Fuquay

The SCS uses the symbol "tp" to refer to the "time to peak" of the runoff hydrograph and "L" is used to symbolize "basin lag," i.e. $L=0.6t_c$. Basin lag (L) as defined by SCS is the time from center of mass of excess rainfall to the peak rate of runoff and is similar to the definition of "K" as defined in your report. Additional definition and description of relationships and symbols used by SCS in hydrologic analysis can be found in Chapters 15 and 16, Section 4, Hydrology of the SCS-NEH.

We hope that these comments will be of benefit to you; and if we can be of further assistance, please let us know.

Sincerely,

John W. Patterson

Thomas G. Rockenbaugh
State Conservationist

cc: Herbert Donald, Maricopa County
Flood Control District