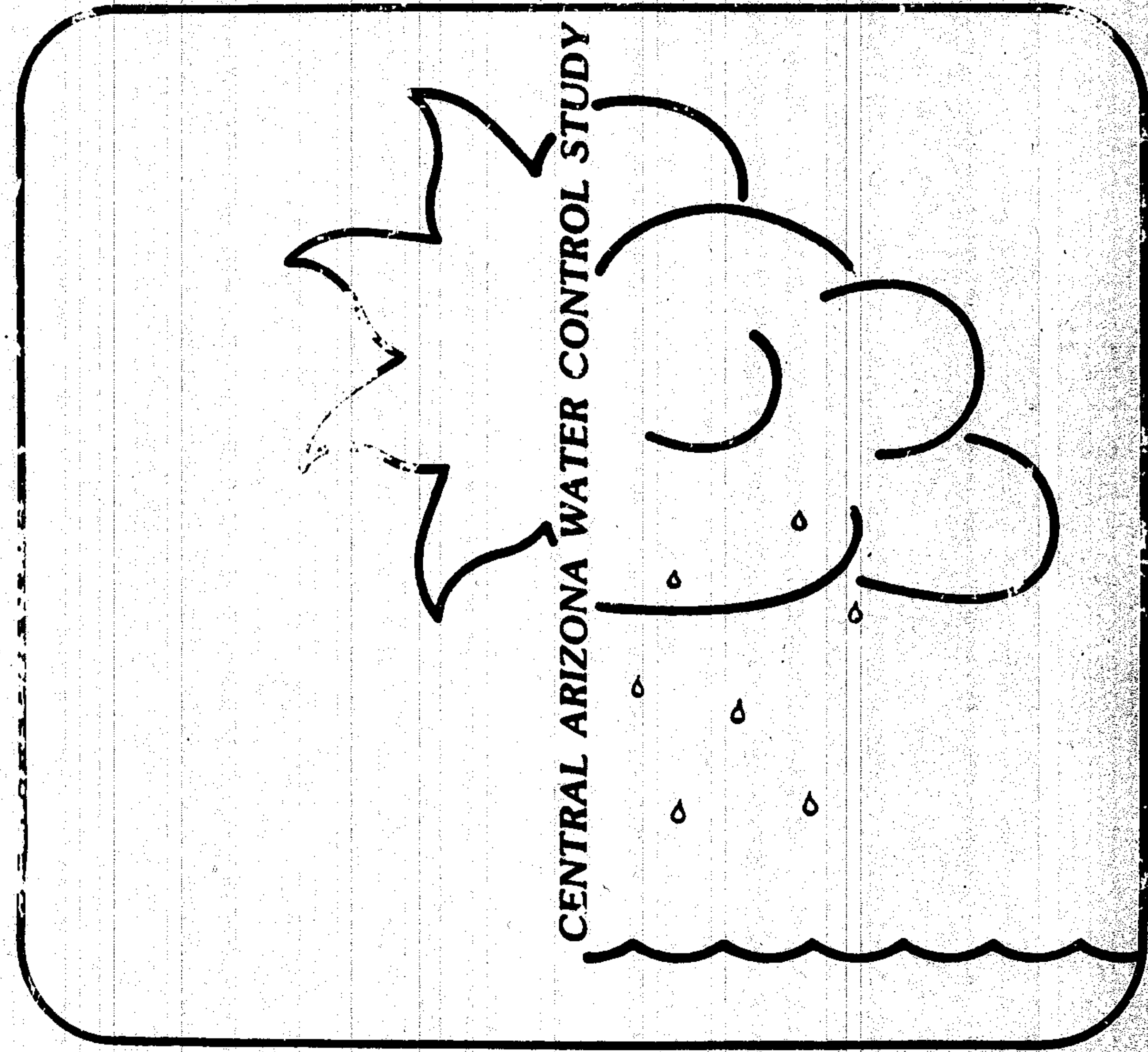
US Army Corps
of Engineers
Los Angeles District

Gila River & Tributaries



HYDROLOGY

May 1982

GILA RIVER AND TRIBUTARIES

CENTRAL ARIZONA WATER CONTROL STUDY

HYDROLOGY REPORT

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

May 1982

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLES

TABLE OF CONTENTS (Continued)

TABLES (Continued)

PLATES

PLATES (Continued)

APPENDICES

I. INTRODUCTION

1-01. PURPOSE AND SCOPE. This report presents the hydrologic analysis of the Gila River drainage basin above Painted Rock Dam in support of plan formulation studies for the Central Arizona Water Control Study (CAWCS). General topics addressed are:

a. basin description and prior storm and flood occurrences;

b. description of analytic tools used to model the runoff process;

c. determination of standard project flood for existing and project conditions;

d. derivation of probable maximum flood at required locations;

e. estimation of sediment pool requirements for proposed Verde River dams;

f. determination of system flood control storage and outlet requirements for project alternatives;

g. development of discharge frequency results for "existing conditions" and "project conditions";

h. discussion of dam safety and the effect of proposed safety-of-dams solutions on existing condition and project condition discharge frequency results;

i. effect of including regulatory storage at confluence site on project conditions discharge frequency results; and

j. preliminary operating criteria for project conditions.

The general location of the study area is shown on plate 1, along with delineation of the drainage area. Peak discharges for existing, project, and seasonal conditions are given in tables 14, 15, 23, 26, and 27. Throughout this report, the phrase "existing conditions" refers to present land use and structures which retain, retard, or divert flow as they currently exist and are operated. The structures which influence flooding in the basin are located geographically on plates 1 and 2; physical characteristics of each structure are given in table 1 and shown on plates 3 through 6.

The hydrologic analysis was conducted in three separate stages. Only the final results appear within the main body of this report, while the results from Stage I and II analyses which were modified during Stage III or were only intermediate, appear in the appendices. Stage I and II hydrologic results which were modified or superceded in Stage III are being published because certain alternatives considered during the plan formulation process were eliminated prior to as well as during Stage III. In addition, Stage I and II results were components of the final results from Stage III.

1-02. STAGE I. The focus of Stage I hydrology was to determine the discharge frequency relationship for the Salt River thru the City of Phoenix based on both "complete control" at Horseshoe and Roosevelt dams and also for existing conditions. Complete control was defined as no release from either structure. Therefore, the frequency of flows emanating solely below Horseshoe and Roosevelt Dams, hereafter referred to as "local flow", was determined. The frequency analysis of total flow, which includes upstream releases from Salt River Project (SRP) reservoirs as well as local flow, and is hereafter referred to as existing conditions, was the major effort of Stage I hydrology and involved continuous simulation of SRP reservoir inflow, release, and spill from August 1888 thru February 1980.

1-03. STAGE II. During the second stage of CAWCS hydrology the major effort was spent in determination of the effect of various proposed projects upon the existing conditions discharge frequency relationship. Project alternatives studied were classified into two sets, structural and reregulatory; the elements and combinations of elements considered under project conditions are presented in table 2. All alternatives were intended to provide flood control on the Salt River through the City of Phoenix, although reduction of flooding on the Gila River below the Salt River would also occur.

a. Structural Alternatives. Structural alternatives comprise new or replacement structures on the Salt and/or Verde Rivers. The individual structures, or elements, and combinations considered are shown on plate 7 and in table 2. To simplify the analysis, the three proposed Verde River elements, New Horseshoe Dam, Cliff Dam, and New Bartlett Dam, were considered to be only two unique elements. Since Cliff is downstream of Horseshoe Dam and upstream of Bartlett Dam, and the intervening drainage areas are minor compared to the total drainage area (less than 5 percent), the New Bartlett flood control requirements (both storage and outlet) were substituted for Cliff; New Bartlett requirements were chosen rather than New Horseshoe criteria because they were more reflective of conditions downstream of Horseshoe Dam. However, construction of a Cliff Dam would result in inundation of existing Horseshoe Dam. For this reason, the conservation space for Cliff Dam would be equivalent to existing Horseshoe conservation space, while the flood control requirements would be equivalent to New Bartlett requirements. Therefore, only New Horseshoe and New Bartlett were analyzed. The other elements examined for Stage II plan formulation were a New Roosevelt and a confluence site dam. Each single element was analyzed as a separate alternative. Several combinations of elements were also considered, thereby combining flood control on the Salt River with flood control on the Verde River. The combinations included New Roosevelt and each of the proposed Verde River Dams; as before, Cliffs would be equivalent to New Bartlett. Finally, an investigation of the feasibility of a Verde River flood control dam along with a confluence site flood control dam was conducted.

b. Reregulatory Alternatives.

(1) In this report, "reregulation" refers to the concept of allocating seasonally inviolate flood control storage in three of the six Salt River Project (SRP) storage reservoirs. Preliminary studies indicated that sufficient flood control space could be provided on the Salt River by

reregulating Roosevelt Dam only, while it may be necessary to allocate space at both Horseshoe and Bartlett Dams to gain sufficient flood control space on the Verde River. All three dams have low level outlet works designed for maximum water supply releases of 2,200 to 4,000 cfs. Additionally, each of the three dams has a gated spillway which is capable of maximum flood releases ranging from 150,000 to 290,000 cfs (plates 3 through 5).

(2) The Stage II study considered four types of reregulation schemes. The first type used only the gated conservation space with no modification to the low level outlet. The second type used the gated conservation pool and some portion of the ungated conservation space (space below the spillway crest) with new low level outlets designed to more rapidly drain the flood control pool between storms. These outlets were called "drawdown" outlets. The third type of system used the gated conservation space, some portion of the ungated space, and enlarged outlets below the gated space, designed to operate as flood control outlets during a flood event and thus minimize the required flood control pool. The final type of system used gated space only, and an improved spillway at Roosevelt Dam which would provide greater release capability at lower head and avoid dedication of large amounts of storage for flood control. Seasonal flood control space at each dam would be dedicated from the existing water conservation space starting at the top of the water conservation pool, which is the normal water surface (NWS). Table 3 presents key water surface elevations associated with the dedication of various amounts of water conservation space for flood control.

(3) The changes studied involved the dedication of various amounts of water conservation space for flood control between the first of December and the last of March each year. Water in this space at the beginning of the flood season would be evacuated prior to 1 December. The space would then be used to detain flows until they could be safely released downstream. At the end of March, the space would again be allowed to fill for water conservation. Most of the schemes studied also required modifications to the low level outlets to increase the flood control effectiveness.

(4) The analysis was based on three important assumptions. First, it was assumed that the SRP system remains as it is today, without operational or structural modifications which may be required for dam safety. Second, it was assumed that the dams are operated by SRP primarily for water conservation as described by the June 1979 Salt River Flexible Operating Criteria (SRFOC, reference 1). Finally, it was assumed that the dams are operated without the benefit of flood forecasting.

1-04. STAGE III.

a. 500-Year Level of Protection. In the final stage of hydrology the original intent was to refine Stage II project conditions discharge frequency results for a select group of alternative "projects" which withstood Stage II scrutiny. However, economic analyses of the array of proposed projects did not indicate an optimum design. Because of this, early Stage III hydrology addressed a higher level of protection, 500-year, for the projects still remaining in plan formulation.

b. Local Flow. At the same time much attention was being directed toward comparison between upstream flood control, e.g. a Cliff-New Roosevelt system and downstream flood control below the Salt and Verde River confluence. Questions were raised concerning the ability of an upstream flood control system to provide an equal level of protection as a confluence structure. The issue concerned the intervening drainage, i.e. the uncontrolled drainage area between the upstream structures and the confluence site, and whether the runoff from the intervening drainage would be in excess of target flood control releases.

A further complication in the upstream versus downstream flood control question resulted from misunderstanding of Stage II terminology. Both upstream and downstream systems were able to control the Standard Project Flood (generated by a storm centered critically over the entire Salt-Verde watershed to produce the maximum runoff at the point of concern, i.e. below the Salt and Verde River confluence) to a target discharge of 50,000 cfs. Thus both systems were presented as being capable of providing Standard Project Flood (SPF) protection to a target discharge of 50,000 cfs. However, the upstream and downstream systems were not equivalent due to the fact that flows emanating below the upstream system were uncontrollable, and their relative merits were to be judged by their impact on the discharge frequency relationships below the Salt and Verde River confluence. The SPF level of protection was investigated, as were others, to provide a basis for comparison, both hydrologically and economically. Because of the multiplicity of Stage II alternatives, refinement of the project conditions discharge frequency relationships for each alternative was postponed until Stage III when a small group of viable alternatives would be compared. Therefore, Stage III hydrology encompassed a more accurate analysis of selected project conditions discharge frequency relationships to include local flow.

c. Safety of Dams. Another major issue which had not been addressed during Stage II was the impact of potential Safety-Of-Dams (SOD) problems and ensuing solutions upon discharge frequency results. It had been determined by the Bureau of Reclamation (BUREC) that the existing spillways for the SRP reservoirs were inadequate to pass their revised Inflow Design Floods (IDF). At mid-Stage II two recommendations for spillway fixes to the existing SRP system were proposed by the BUREC. These proposed changes became alternatives under Stage III plan formulation, not only as stand-alone systems, but also while incorporated into either upstream or downstream flood control analysis. As a consequence, Stage III hydrology addressed this issue in detail for the most promising flood control systems.

d. Regulatory Storage at the Salt-Verde Confluence. Since it was decided that a system combining upstream flood control with a small confluence structure for regulatory storage was a viable plan, it became an additional task to determine whether the small confluence structure would affect discharge frequency results of an upstream flood control system.

e. System Redesign. Economic analysis of Stage II structural alternative designs revealed that flood control storage (embankment height) was less expensive than large flood control outlets, contrary to Stage II assumptions. For this reason the upstream system involving SPF level of protection was reevaluated to define more economic sizing requirements.

f. Operational Criteria. A preliminary encapsulation of operational criteria based on the computer simulation design process was documented for flood operation. In addition a seasonally varying criteria was presented.

1-05. PREVIOUS REPORTS. Previous reports published by the Corps of Engineers which are pertinent to this study are: "Interim Report on Survey, Flood Control, Gila River and Tributaries above Salt River, Ariz. and N. Mex.," 1 December 1945; "Design Memorandum No. 1, Hydrology for Painted Rock Reservoir, Gila River, Arizona," 1 August 1954; "Interim Report on Survey for Flood Control, Gila and Salt Rivers, Gillespie Dam to McDowell Dam Site, Arizona," 4 December 1957; and "Stage II Report, Hydrology Appendix, Gila River and Tributaries, Central Arizona Water Control Study," December 1980.

II. BASIN DESCRIPTION

2-01. PHYSIOGRAPHY AND TOPOGRAPHY.

a. The Gila River Basin, which is an irregular area of 58,200 square miles (57,900 excluding all closed drainages) extending from the Continental Divide in southwestern New Mexico to Colorado River at Yuma, Ariz., includes practically all the southern half of the State of Arizona and constitutes a region of widely varying topographical and climatological characteristics. The river, which is 654 miles long, rises in an area of high mountains and plateaus, and flows westward in a generally central course through the basin.

b. Much of the northern part of the basin is drained by the Salt River, the largest tributary, which joins the Gila River at mile 198, near Phoenix (plate 8). The Salt River Basin, with a drainage area of 13,700 square miles (13,400 excluding all closed areas), is extremely irregular and rugged. Elevations commonly rise to more than 7,000 feet and, at the San Francisco Peaks in the Verde River Basin, to more than 12,000 feet. The Verde River is the main tributary to the Salt River and comprises 6,620 square miles (6,320 excluding all closed areas) of the Salt River drainage area. The eastern portion of the southern part of the Gila River Basin consists largely of long desert valleys lying between north-south ranges of rugged mountains; here the elevations, although rising in places to above 10,000 feet, are generally lower. The southwest portion of the basin consists essentially of broad, flat, low-lying desert valleys and isolated mountains of relatively low relief; comparatively few localities are more than 4,000 feet in elevation, and a large part is below 1,000 feet; the elevation of the river mouth near Yuma is about 130 feet. Gillespie Dam is in the upstream part of this basin, at river mile 164. Soils and vegetative types vary widely throughout the basin.

2-02. CLIMATOLOGY.

a. General. The climate of the Gila River Basin as a whole is semiarid but, depending principally upon elevation, ranges from hot and arid in some parts to cool and humid in others. The average annual precipitation ranges from less than 4 inches in the lower desert to 30 inches or more in the highest mountains. Most of the precipitation occurs in two distinct seasons, summer (July through September) and winter (December through March), and is about equally divided between them. Little rain normally falls during spring and autumn. During any season there may be many successive rainless days.

b. Summer Precipitation. Summer precipitation may be placed in two general classifications. The first classification includes the sporadic showers and cloudbursts of small areal extent that occur, usually from insolational heating of tropical maritime air that frequently invades the region from the Gulf of Mexico or the Gulf of California and the South Pacific. The second classification includes the general rains that result from convergence, orographic lift, and frontal lift in situations where frontal systems, with associated tropical maritime and polar continental or maritime air, pass through the region; thunderstorms may or may not be associated with general rains in this classification.

c. Winter Precipitation. In winter, most precipitation results from general storms that are associated with extratropical cyclones of North Pacific origin. Relatively localized showers commonly occur near the end of such storms. Both the general winter and the general summer storms may result in rain over the entire Gila River Basin. On the average, the general winter storms are longer in duration. They sometimes produce rain that is more or less continuous for several days. In winter, snow may accumulate to considerable depths at elevations above 4,000 feet but practically never falls at elevations below 2,000 feet.

d. Precipitation Records. Precipitation records are available for more than 600 rainfall stations in and near the Gila River Basin. The earliest record (Fort McDowell) begins in July 1866. The longest continuous record (Yuma) begins in 1870. The longest continuous autographic record (Phoenix) begins in 1906. Most of the autographic stations have been established since 1939. Many of the records since 1900 include information on snowfall, and snow-course observations have been made since about 1937 at several locations in the drainage areas of the Verde, Salt, and upper Gila Rivers.

2-03. FLOODS OF RECORD.

a. General Characteristics. Hydrologic records indicate that on the lower Gila River the greatest floods have resulted from storms of the general winter type, and studies of rainfall and runoff relationships indicate that the most critical runoff quantities would probably result from such storms. In winter, the ground throughout the basin is most likely to be wet from other general rains; the upstream reservoirs are most likely to be full, or nearly full of water for conservation use; and the runoff due to snowmelt may be potentially great. In major storms the duration of appreciable floodflows varies, but seldom exceeds 8 days. The records show no large floods in the lower Gila River in summer. There are indications that general summer storms approaching the winter storms in magnitude could occur over the entire river basin, but probably the attendant ground conditions would be less severe than those to be expected in winter. The size of the basin tends to preclude the probability of a great flood resulting from a series of thunderstorms.

b. Runoff Records. Runoff records are available for approximately 100 gaging stations on the Gila River and tributaries. The longest record, Verde River below Bartlett Dam, dates back to 1888 and is nearly continuous since 1903. Records of discharges at some stations during flood periods are often incomplete.

c. Floods. Historical accounts indicate that general floods occurred in 1833, 1862, 1869, 1880, 1884, 1886, 1889, 1890, 1891, 1893, 1895, and 1903. Records since 1904 show that floods and/or storms occurred in March 1905, April 1905, November 1905, March 1906, December 1906, December 1914, January 1915, January 1916, October 1916, November 1919, February 1920, December 1923, September 1926, February 1927, February 1937, March 1938, March 1941, September 1946, August 1951, December 1965, September 1970, October 1977, March 1978, November 1978, December 1978, January 1979, and February 1980. The flood of 1884 was the earliest for which a reasonable estimate of severity can be made. It probably was comparable to the greatest floods of record, those of February 1891 and January 1916. The magnitudes of major floods of record in the Salt River below the Verde River confluence for simulated existing conditions and "natural" conditions are shown in table 4.

III. RAINFALL-RUNOFF

3-01. RECONSTITUTIONS.

a. The procedures developed for computation of standard project flood (SPF) and probable maximum flood (PMF) for the 1957 McDowell Dam Interim Report (reference 2) were used as the basis for rainfall-runoff calculations. Confirmation of the unit graphs and loss rates generated from application of the 1957 criteria was attempted during Stage II studies. This involved reproduction of the March 1978 flood on the Salt and Verde Rivers by applying the 1957 unit graphs and loss rate criteria to rainfall depths and time distributions developed for the March 1978 storm. The resulting computed inflow hydrograph at Roosevelt Dam agreed well with the observed hydrograph, but the computed inflow hydrograph at Horseshoe Dam did not reflect the recorded inflow. There are several reasons for this disagreement:

(1) inadequate representation of rainfall in the Verde basin;

(2) inability to model the snowmelt function;

(3) lack of accurate data for observed inflow to Horseshoe Dam - the 174 square mile drainage area below the Tangle Creek gage contributed a significant inflow to Horseshoe Dam which may not have been indicated by the gaged flow at Tangle Creek.

Since these limitations, rather than unit graph and loss rate criteria, prevented accurate reproduction of the event, the 1957 unit graph and loss rate criteria on the Verde River were felt to be an adequate representation of rainfall-runoff processes through Stage III plan formulation.

b. A detailed storm analysis for significant runoff events in the Salt and Verde basins has since been undertaken, and reconstitution of these floods will be performed during 1982. It is believed that the SPF and PMF estimates based on the 1957 rainfall-runoff criteria will not be greatly altered by the results of the reconstitutions.

3-02. LOSS RATES.

a. SPF. In the absence of detailed analyses of relationships of runoff to rainfall in recorded storms, the total amounts of precipitation that would appear in the streams as runoff (effective rain) during the standard project floods were computed on the basis of a study made for the 1957 report of the volumes of runoff estimated to have occurred at various locations throughout the Gila River Basin as a result of the larger storms of record. The volumes of runoff were expressed as percentages of total precipitation for various storm periods. Such percentages reflect, in a general way, the amounts of rainfall lost by surface detention, infiltration, evaporation, and channel percolation losses in the various tributary areas. They also reflect the accretions to streamflow resulting from ground-water return flow and from melting snow. The percentages for the storms examined indicated that, in general, proportionately the greatest amounts of runoff were from the areas of higher elevation, where rainfall and snowmelt are usually greater. On the basis of the 1957 study, average percentages that would represent ground conditions reasonably conducive to runoff from each subarea were assumed. The assumed percentages, which ranged from 25 to 35 percent, are considered to include adequate allowances for snowmelt and base flow. Also, they collectively constitute an overall degree of severity slightly greater than that existing in the 1916 and 1938 storms.

b. PMF. The SPF percentages of total rain that would run off were increased 10 percent, such that areas with 35 percent runoff would be increased to 45 percent.

3-03. EFFECTIVE PRECIPITATION.

a. SPF. To determine the amounts of effective rain (including base flow and snowmelt) by unit periods in the standard project flood, the results of 1957 unit-hydrograph studies for two areas in southern California for which relatively detailed hydrologic data are available (one area of high rainfall and one area of low) were utilized. For each study, a curve was plotted showing accumulative storm rainfall versus accumulated effective rainfall, both by unit periods throughout the storm. In each case the plotted points could be reasonably well fitted by a straight line. Using this method for estimating effective rainfall in the 1957 corroborative studies, performed in connection with the applicability of the adopted lag curve and S-graph, indicated that the straight-line relationship would give reasonably satisfactory hydrograph reproductions for the Gila River Basin floods studied, namely, the 1916, 1937, and 1941 floods on the Salt River near Roosevelt Dam, and the 1937 flood on the Verde River. Accordingly, such a straight-line relationship was adopted for the standard project flood computations. The computed rainfall-loss rates for the periods of heaviest rain ranged from 0.05 to 0.10 inch per hour. These rates appear reasonable. Deviation from the straight-line relationship would tend to affect the shape of the computed flood hydrograph for each subarea and perhaps modify the peak discharge slightly, but would not affect the total volume of runoff. A sample computation is shown in table 5.

b. PMF. The effective precipitation calculations were originally done using the same procedure outlined for calculation of SPF effective rainfall (para. 3-03.a). However, because of the magnitude of the PMF estimates and the difference between BUREC and Corps methods for computing the spillway design flood (PMF), the rainfall-runoff criteria which were acceptable for SPF calculations, were scrutinized more closely. The 1957 loss rate used in Corps studies was satisfactory on a volumetric basis, but because of its discontinuous nature, questionable to determine the time distribution of effective precipitation. Unit graph criteria used by BUREC and the Corps to compute spillway design flood agreed well. Therefore, the loss rate was reevaluted in the following context:

(1) The loss volumes (35 to 45 percent of the rainfall) were felt to be reasonable since they include snowmelt and base flow as well as effective precipitation - by contrast, total March 1978 runoff was 28 percent of the computed rainfall.

(2) Since PMF implies saturated conditions, the high initial losses would have been met, and a limiting or constant rate reached; therefore a constant loss rate was used such that runoff equalled 45 percent of the precipitation.

3-04. UNIT GRAPHS.

A unit graph is a runoff hydrograph which represents the response of a basin or sub-basin to one inch of effective precipitation occurring uniformly over that area in a specified time period. The concept of a unit hydrograph, in conjunction with a loss rate for determination of effective precipitation, permits computation of a runoff hydrograph for any duration and depth of uniform precipitation over that basin. The runoff hydrograph itself is determined by combining linearized hydrographs for each effective precipitation time period described by the unit graph storm duration. The linearized hydrographs are ratios of effective precipitation for each period (in inches) to the unit graph ordinates and are combined sequentially by use of the superposition principle to determine the actual flood hydrograph for the total storm for each basin. The unit hydrograph procedure used by the Los Angeles District has its basis in an S-graph which is the time distribution of runoff as a function of basin lag time. Lag time is defined as the time in hours for 50 percent of the total volume of runoff of the unit hydrograph to occur. The basin lag time can be approximated for ungaged watersheds by the use of the lag relationship presented on plate 9. The basin n-value is a proportionality factor in the equation for lag time which permits adjustment of lag time depending on type of ground cover and surface characteristics affecting basin response to effective rainfall. Synthetic distribution graphs (unit graphs whose ordinates are expressed as runoff in percent of unit runoff) for each of the subareas were derived from data developed in 1957 unit-hydrograph studies made of several areas in the Gila River Basin and in southern California. A single basic S-graph representing an average of time-distribution characteristics of four comparable regional streams was assumed to be applicable for determining each of the required synthetic distribution graphs. The required lag values for the subareas (for use in converting the S-graph to distribution graphs) were taken from the lag curve applicable to the areas for which the original unit-graph studies were made. Pertinent unit-graph data are displayed in table 6 and 7. These basin parameters were input to the Los Angeles District Flood Hydrograph Package (LADFHP, reference 3), a computer simulation model developed by the Los Angeles District, Corps of Engineers. This model computes unit graphs as well as flood hydrographs.

3-05. FLOOD ROUTING.

a. Reservoir Routing. Typical reservoir routing methods were employed, using the "HEC-5 Simulation of Flood Control and Conservation Systems" computer program (reference 4).

(1) SRP System. Due to the function of the SRP reservoirs--water conservation and hydroelectric power generation--and to their limited outlet capacity below spillway crest, the SRP system is often "full" or nearly full", i.e., near normal water surface. The lower three reservoirs on the Salt River (Horse Mesa, Mormon Flat, and Stewart Mountain Dams) are hydroelectric

generation facilities and are kept at 90 percent full or higher, except during periods of extreme low flow. The nature of SPF and PMF, resulting in both cases from intense, general winter storms, includes antecedant runoff which would bring the other reservoirs into a "full" condition. Therefore, starting water surface elevations for SPF and PMF flood routings were at NWS (whether the reservoirs are nearly full or completely full has no effect on the peak). Estimated releases were made such that outflow was equal to inflow, the limit being the hydraulic capacity of the gated spillways. This type of reservoir operation maintains surcharge space for dam safety, if possible, and follows the 1979 SRP SRFOC (reference 1).

(2) Other reservoirs. The starting conditions and reservoir routing techniques for Coolidge and Waddell Dams were similar to those for SRP reservoirs, since they also operate primarily for water conservation. Coolidge Dam differs in spillway configuration because the gates are no longer operational and are frozen in closed position. The NWS at Coolidge Dam is actually established by concrete flashboards. During reservoir routing for the Painted Rock SPF, the flashboards were considered to fail. The failure was assumed to be complete, thus increasing the spillway capacity by lowering the crest and enlarging the spill area. The magnitude of the discharge at failure was only 25,000 cfs, compared to the peak spill of 92,000 cfs later in the flood; therefore, time of failure did not affect the peak.

b. Channel Routing. Modified Puls routing procedures were used to channel route frequency hydrographs as well as the SPF and PMF hydrographs. A summary of storage-discharge relationships for 1-hour and 6-hour time intervals, as required, is presented in tables 8 through 11 for each routing reach.

3-06. PERCOLATION LOSS. Not only are flood peaks in the Gila River system attenuated through effects of reservoir and channel routing, but they are also diminished volumetrically due to infiltration of streamflow into the river channel and overbank areas. This type of infiltration is apparent in several recent floods of varying peaks and volumes such as Dec 1965-Jan 1966, Feb-May 1973, March 1978, Dec 1978, Jan-April 1979, and February 1980. As evident from these floods, the rate of percolation is dependent on antecedent conditions, duration of flow, shape of hydrograph, and magnitude of peak and volume. An exponential type decay function similar to Horton's infiltration equation was hypothesized as a model to explain the percolation mechanism. This model predicts an ultimate or limiting infiltration rate based on a higher initial rate decaying over time (plate 10). A limiting infiltration rate of 0.2 cfs per wetted-acre of channel yielded good results based on studies of the aforementioned floods. The Hydrologic Engineering Center (HEC) also studied percolation in the Salt River near Phoenix (reference 5). Using a similar exponential decay function, an "average infiltration rate" of 1.3 in/hr was computed for flow in a one day period. The limiting value was 0.2 inches per hour. (1 inch per hour equals 1 cubic foot per second per wetted acre.) Since the results of both studies agreed, a percolation rate of 0.2 cfs per wetted acre for all normally "dry" channel reaches on the Salt, Agua Fria, and Gila Rivers was selected. The constant percolation rate was felt acceptable because the constant or limiting rate would be achieved prior to arrival of the peak, thus having no effect on the degree of attenuation.

Furthermore, the volumetric effect was minimal, since bank returns at the end of the flood tended to restore the water lost in the early stages of a real event. Percolation rates in cfs per acre-foot of channel and bank storage are shown for each normally dry reach in tables 9 through 11.

IV. 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.

a. Studies of available hydrologic data for the Gila River Basin and adjacent southwest areas have shown that the storms of record with potentially the most critical flood-producing characteristics for the drainage areas above Painted Rock Dam and a confluence dam site were the first storm of January 1916 and the storm of February-March 1938. These storms, as actually oriented over the basin, produced rainfall amounts moderately less critical than those that would have resulted if the storms had been centered over the area. The standard project flood on the Gila River between Painted Rock Dam and the Salt River has been synthesized on the basis of the assumed occurrence of a storm equivalent to the 1916 storm centered (approximately 50 miles northwest of actual occurrence) over the area above Painted Rock Dam. The standard project flood on the Salt River between its mouth and the Verde River has been synthesized on the basis of the assumed combined occurrence of a storm equivalent to the 1938 storm and the 1916 storm centered (approximately 20 miles northeast and 80 miles northwest, respectively, of actual occurrence) over the area above the confluence of the Salt and Verde Rivers.

b. Determination of the magnitude of the storms that would be equivalent to the 1916 and 1938 storms but would have a critical centering to the northwest and northeast, respectively, was accomplished by (1) expressing the actual rainfall amounts in the 1916 and 1938 storms as percentages of the mean rainfall amounts for the period of October through May, (2) constructing insopercentual maps based on those percentages, and (3) shifting the insopercentual lines to such a position over the basin as would result in more critical amounts of rainfall over the drainage area above the respective concentration points. Use of the mean precipitation for the months of October through May as the transposition factor for determining standard project storm precipitation is considered warranted in view of the fact that most precipitation in those months in Arizona results from storms of the general winter type, and thus such mean seasonal precipitation is an indication of the effects of basin topography on precipitation in general storms. Preliminary analysis of the March 1978 storm shows relative precipitation amounts and isohyetal patterns very similar to the standard project storm adopted. The standard project storm rainfall amounts are shown on tables 12 and 13.

c. One hour was selected as the smallest time interval for which information on rainfall intensities would be required in developing the standard project floods. The time distribution of the rainfall intensities for the respective parts of the standard project storms over the different subareas was made equal (with 1-hour amounts expressed as percentages of total storm amounts) to the time distribution of rainfall in the same relative parts of the original 1916 and 1938 storms as computed from intensity patterns determined under assignments SP 1-20 and SP 2-8, respectively, of the cooperative storm-study program of the United States Weather Bureau and the Corps of Engineers conducted prior to the 1957 report.

4-03. DETERMINATION OF SPF.

a. SPF Computation. The standard project floods for both centerings were computed in several identical steps as follows:

(1) effective rainfall (para. 3-03.) for each subarea was calculated by application of the 1957 loss rate (para. 3-02) to the standard project storm precipitation totals;

(2) unit graphs for each subarea were determined as discussed in para. 3-04;

(3) flood hydrographs for each subarea were determined by inputting effective rainfall to LADFHP, which combines computed unit graphs with effective rainfall to determine subarea hydrographs;

(4) the respective subarea component flood hydrographs were input to HEC-5, the reservoir operation program, wherein all reservoir routing, channel routing, hydrograph combination, and percolation losses were taken into account (para. 3-05 and 3-06). A schematic diagram of the routing and combining procedure is shown on plate 11.

b. SPF Results. SPF peak discharges, computed as described above, are presented in table 14 and 15 for both existing conditions and the hydrologically viable project alternatives listed in table 16.

V. PROBABLE MAXIMUM FLOOD

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

5-02. PROBABLE MAXIMUM PRECIPITATION. Probable maximum precipitation (PMP) is considered the practical upper limit of available precipitable water over an area as estimated by the Hydrometeorological Branch (HMB) of the National Weather Service. Techniques for determination of depth of general storm PMP for 72-hour durations for drainage areas between 10 and 5,000 square miles for locations within the Colorado River and Great Basin drainage is given in Hydrometeorological Report No. 49 (HMR-49, reference 6). Guidance for time distribution of PMP is provided in Hydrometeorological Report No. 36 (HMR-36, reference 7). Permission was given by HMB to carefully extrapolate relationships for PMP provided in HMR-49 beyond the 5,000 square mile limit for Roosevelt (5,830 sq. mi.) and Horseshoe Dams (5,660 sq. mi.). The PMP depth and time distribution for both summer and winter 72-hour general storms for the 12,900 square mile confluence site was also provided by HMB (reference 8). The PMP depth and time distribution for both summer and winter storms are shown in tables 17 and 18.

5-03. DETERMINATION OF PMF.

a. The probable maximum floods for the reservoir sites being studied i.e., Roosevelt, Horseshoe, Cliff, Bartlett, and the Salt-Verde confluence, were computed using the methods outlined in sections III and IV. The winter PMP produced a more severe flood than the corresponding summer PMP for all sites, due to the increased runoff potential caused by factors such as snowmelt, frozen soil, saturated soil, base flow, decreased demand, "full" reservoirs, and smaller abstractions such as evapotranspiration.

b. The Verde River sites (Horseshoe, Cliffs, and Bartlett) were considered as a single site because of the minor impact of the 195 square mile drainage area between Horseshoe and Bartlett Dams. Horseshoe Dam was assumed to pass the PMF component for Bartlett Dam and the confluence site dam without attenuation or failure.

c. The PMF for the confluence site was computed assuming the top of the existing Roosevelt Dam would be raised as necessary to prevent overtopping; the remaining SRP dams were assumed to pass the inflow components without attenuation or failure.

d. A summary of PMF values is given in table 19.

VI. SEDIMENT PRODUCTION - VERDE RIVER DAMSITES

6-01. SEDIMENT RATE DETERMINATION.

a. An estimate of the 100-year volume of sediment accumulation behind each the alternative Verde River damsites is needed for design of the proposed flood control structures. The estimates are based on sediment inflow records for Horseshoe and Bartlett Reservoirs and recorded data for other streams and existing reservoirs in the general area. The estimates were also checked using the techniques described in reference 9.

b. Data from reservoir surveys of Horseshoe and Bartlett Reservoirs appear to indicate a moderate rate of sediment production potential. (Note: Moderate is Class #3, 0.05-1.00 AF/sq.mi./yr., reference 9.) Reservoir surveys were conducted in October 1950 (reference 10), November 1963 (references 10 and 11), and October 1978 (reference 11) for Horseshoe Reservoir. Bartlett Reservoir was surveyed in November 1950 (reference 10), January 1964 (references 10 and 11), and June 1977 (reference 11). Table 20 gives the results of the surveys. Sediment yield rates for other existing reservoirs in the region are also given in table 20.

c. There are two major reasons for the low sediment yield rate observed at Horseshoe Reservoir.

(1) A substantial portion of the watershed is tributary to long, wide valleys which have mild streambed gradients. Rivers on a mild slope have relatively low capacity to transport sediment, and a moderate-to-low sediment yield rate would be expected from the area.

(2) An observed sediment yield rate is also closely related to the number of floods that occur in the observation period, as seen in the case of Horseshoe Reservoir. Between October 1950 and November 1963, only one major flood occurred on the Verde River (1952, maximum 1-day flow = 42,300 cfs). The next highest 1-day flow during this period was 17,300 cfs in 1958. From November 1963 to October 1978, three floods had maximum 1-day flows ranging from 45,000 to 60,000 cfs. For the remaining years of this period, maximum 1-day flows were less than 27,000 cfs. The sediment rate for the latter period with more frequent high flows is nearly twice the rate for the earlier period, 0.093 ac.ft./sq.mi./yr. to 0.049 ac.ft./sq.mi./yr. (table 20).

6-02. DESIGN ESTIMATES.

a. Assuming of the sediment derived from the Chino and Verde Valleys is deposited upstream, the major contribution would come from the area below Verde Valley, approximately 600 square miles, although there would be some sediment from the upstream areas. Choosing an equivalent contributing area of 1,000 square miles, the average annual sediment yield computed from the survey data is 0.407 acre-feet per square mile per year. This is a reasonable rate in view of the small number of major floods that occurred during the period covered by the surveys. For design purposes, a sediment yield of 0.65 acre-feet per square mile per year for an equivalent contributing area of 1,000 square miles above Horseshoe Dam was adopted, thus accounting for some sediment from the area above Verde Valley and the small number of major flows in the survey data. The same annual sediment production, 650 acre-feet per year, can be derived by increasing the annual sediment production during the period from November 1963 to October 1978 by only 25 percent. This period better represents the magnitude and number of major floods on the Verde River over the long term, but it is not considered typical.

b. The same yield rate would apply to the proposed Cliff Reservoir, with the equivalent contributing area increased to 1,081 square miles since Horseshoe Dam would be breached for this alternative.

c. For the New Bartlett Dam alternative, Horseshoe Dam would remain in place, reducing the area directly contributing sediment to New Bartlett Reservoir to (based on published DA's) 195 square miles. The sediment potential of this area is similar to the area between Verde Valley and Horseshoe Reservoir. Therefore, the yield rate would be about the same, assuming similar trap efficiencies of Horseshoe and New Bartlett Dams.

d. Estimates of 100-year sediment volume for the proposed dam alternatives are given in table 21.

VII. ADEQUACY OF RESULTS.

7-01. STANDARD PROJECT FLOOD.

a. Salt River. The adequacy of the standard project flood for Salt River can best be shown by comparison of the magnitudes of this flood with floods of record. The peak flow for the standard project flood, assuming no upstream dams, would be 350,000 cfs, or 50,000 cfs larger than the uncontrolled peak of 300,000 cfs for the 1891 flood. The peak flow of the standard project flood modified by existing dams (295,000 cfs) is about equal to the estimated uncontrolled peak of the 1891 flood, which is probably the greatest flood of record, below the confluence of the Verde and Salt Rivers. The return period for this hypothetical flood peak discharge is approximately 200 years.

b. Gila River. Similarly, the greatest flood of record, estimated to have peaked at about 250,000 cfs near the present Gillespie Dam, also occurred in Feb. 1891 without any upstream reservoir control. The SPF peak computed for the Gila River (300,000 cfs) is likewise 50,000 cubic feet per second larger than this uncontrolled peak. The return period for this hypothetical flood on the Gila River is greater than 250 years.

c. Summary. The SPFs developed during Stage II are considered adequate for design purposes based on the severity of the standard project storm, magnitude in comparison to historical events, and infrequent recurrence interval.

7-02. PROBABLE MAXIMUM FLOOD. The adequacy of probable maximum flood for the damsites is best indicated by the severity of the various hydrologic factors (storm magnitude, precipitation-intensity pattern, and loss rate) on which the flood estimate is based. The occurrence of any of these factors in the severity assumed would be infrequent, and obviously a flood resulting from a combination of all of these conditions would be very severe. Confirmation of the hydrologic parameters through reconstitutions of observed flood events has yet to be performed. Because the PMP will remain intact, no major change in PMF values is anticipated.

7-03. SEDIMENT PRODUCTION. The adequacy of the design sediment estimates for the Verde River dam alternatives is illustrated by parity with sediment yield rates for reservoirs in the general vicinity. Also, sediment yield rates determined by the procedures described in reference 9 produced values very similar to the adopted rates.

VIII. DISCHARGE FREQUENCY ANALYSIS

8-01. STREAMFLOW RECORD. Streamflow records were available for locations along the Salt River on a nearly continuous basis since August 1888. The streamflow record for the Gila River is intermittent during the period 1889-1914; continuous recorded inflow at Coolidge Dam is available from 1914, and at Gillespie Dam from 1921. Instantaneous discharge estimates are available at various locations along the Gila River for major observed floods beginning with the flood of February 1891. Periods of streamflow record for the Gila and Salt Rivers and their major tributaries, as used in this report, are listed in table 22 and shown on plate 11. Missing years of record at required locations were estimated using cross-correlation with known or estimated discharges for other durations, with upstream or downstream-mainstem stations, and with stations on other streams.

8-02. EXISTING CONDITIONS. To develop discharge frequency relationships for existing conditions, the recorded streamflow record for the Salt and Gila Rivers had to be converted to a sequence of "standardized" existing conditions discharges. Standardization, i.e. converting all streamflow to the same base, existing conditions, was required because the recorded data was published for a non-homogeneous period of record. Reservoir construction began with Roosevelt Dam, built during the period 1905 through 1913, and continued throughout the basin through 1945 when Horseshoe Dam was completed (table 1). The analysis was conducted in two parts, one for the Salt River through the City of Phoenix, and the other for the Gila River between the Salt River confluence and Painted Rock Dam.

a. Salt River through the City of Phoenix.

(1) Simulation of Existing Conditions. To standardize flow in the Salt River, SRP reservoirs were modeled using the HEC-5 computer program. Reservoir characteristics, channel routing parameters, and percolation losses were established as discussed in para. 3-05 and 3-06. A monthly release schedule was established based on "average" downstream surface water demands derived from surface water and pumping requirements for present and expected "near future" conditions provided by and with cooperation from SRP. (reference 1). Average monthly reservoir evaporation rates were established from National Weather Service pan evaporation data for SRP reservoirs. These parameters were then incorporated into the HEC-5 program to simulate SRP operation under existing conditions. The model was calibrated using the Dec. 1965-Jan. 1966 and March 1978 floods.

(2) Monthly Screening. Monthly flow data for the Salt and Verde Rivers and Tonto Creek were adjusted to produce a continuous sequence of inflows to Roosevelt and Horseshoe Dams for the period from August 1888 to the present. Starting storages for SRP reservoirs in August 1888 were estimated from inferences of historical floodflows prior to 1888 and "normal" demand and evaporation losses during the intervening period. The monthly inflows were then routed through the computer simulation model to determine in which months spills would likely occur under existing conditions. Streamflow in the Salt River below the SRP reservoirs of magnitude greater than the monthly demand was considered a "spill". Streamflow less than or equal to the monthly demand

was diverted at Granite Reef Dam. Spills determined were catalogued and screened to determine the month of the maximum event for each water year.

(3) Determination of Discharge Frequency Relationships. Daily inflows to SRP reservoirs were developed from gaged streamflow for the maximum spill events determined by the monthly screening. In addition, the daily flows were broken into 6-hr average discharges for flood events. These daily and multi-hourly inflows were routed through the simulated SRP system, with initial reservoir storages for the month(s) determined by the monthly screening results. In water years when the maximum spill event was in doubt based on monthly screening alone, all the months in question were analyzed in this same manner. Resulting annual maximum values for peak, 1-day, 2-day, 3-day, 5-day, and 10-day durations were then ordered and plotted on log-probability frequency paper, using median plotting positions, for each concentration point along the Salt River through the City of Phoenix. Curves were fit through the plotted data using as guide a set of "natural" discharges, generated along with the existing conditions discharges by the HEC-5 simulation program. Balanced hydrographs for the 500-year, 200-year, 100-year, and 50-year events were determined for the combined inflow to Roosevelt and Horseshoe Dams and routed through the SRP simulation model to aid in developing the shape of the discharge frequency curves. A sample peak discharge frequency curve developed using these procedures is shown on plate 13. Results of the analysis are displayed in table 23.

b. Gila River Between the Salt River Confluence and Painted Rock Dam.

(1) Simulation of Existing Conditions. Coolidge Dam and Waddell Dam on the upper Gila and Agua Fria Rivers, respectively, are the only structures which affect major flood flows on the Gila River between the Salt River and Painted Rock Dam, other than the SRP system. To simulate existing conditions in this reach required combining the preceding existing conditions streamflow data generated for the Salt River through the City of Phoenix with synchronous existing conditions streamflow in the upper Gila and Agua Fria Rivers. The upper Gila River has greater potential for contributing to flow in the lower Gila River than the Agua Fria River because of its greater drainage area and subsequent runoff volumes. However, Coolidge Dam has effectively controlled all inflow since its closure in 1928 until 1980. Thus inflows to Coolidge Dam were only analyzed prior to 1928. The HEC-5 computer program was used to simulate average monthly demand, evaporation, and reservoir characteristics for Coolidge Dam. Average monthly demand was based on USGS stream gage record for the Gila River below Coolidge. Evaporation data was taken from pan evaporation data for Coolidge Dam, National Weather Service, and reservoir characteristics were provided by BUREC, and Bureau of Indian Affairs.

(2) Monthly Screening. Monthly inflow to Coolidge Dam was extended back to the 1903 water year by correlating gaged record at Coolidge Dam with other gaged streamflows on the Gila River, after adjustment to account for Salt River flows (plate 14). Only one spill, February 1891, would have occurred between 1888 and 1903. Starting storage for 1903 was estimated from available annual runoff and precipitation records and "normal" depletions since February 1891. Continuous existing conditions monthly flows were generated for the 1903 to 1928 period and routed through the reservoir to determine spills.

(3) Winter Flood Hydrograph Analysis. The procedure for analysis of floods on the Gila River between the Salt River and Painted Rock Dam involved four major streamflow components: the Salt River above the confluence with the Gila River (para. 8-02. a. (3)); the Gila River below Coolidge (spills); the Gila River between Coolidge and Gillespie Dam; and the Agua Fria River.

(a) 1888-1903. Since no spills would have occurred from Waddell or Coolidge Dams during this period other than in February 1891, flow in the Gila River between the Salt River and Painted Rock Dam was based only on Salt River routed flows. For February 1891, Salt River discharge was combined with estimated spills from Coolidge and Waddell Dams.

(b) 1903-1928. During this period, the Salt River component was combined with synchronous floods routed through Coolidge Dam during the period 1903-1928 on a daily and multi-hourly basis. Estimated spills from Waddell Dam and flow in the Gila River between Coolidge and Gillespie Dams were then combined with these discharges and the results routed to Painted Rock.

(c) 1928-Present. Since 1928, there have been no significant spills from Coolidge Dam. In the period during April 22 through May 20, 1979, Coolidge spilled or released water at a rate less than 2000 cfs. Of this total, less than 10 cfs reached the Gila River below the Salt River. A slightly larger spill/release occurred in 1980 (4000 cfs). Neither of these events impact the frequency relationships for the Gila River at CP-1310. Therefore, for this period Salt River streamflow was combined with Waddell Dam spills plus Gila River flow between Coolidge Dam and Gillespie Dam.

(4) Summer Flood Hydrograph Analysis. Simulation results indicated none of the major storage facilities, SRP reservoirs and Coolidge and Waddell Dams, would have spilled in the summer during the period of record. However, there would have been summer floods in the Gila River emanating from areas not controlled by these reservoirs, e.g., the September 1926 flood in the San Pedro River. Streamflow in the Gila River between the Salt River and Painted Rock Dam from the major uncontrolled sources--the San Pedro, Santa Cruz, and Hassayampa Rivers, and Centennial Wash--is reflected in gaged record below Gillespie Dam. Therefore, gaged streamflow at Gillespie, adjusted, if necessary, to exclude streamflow from the controlled sources, was examined to determine the annual maximum summer runoff for durations of interest.

(5) Derivation of Discharge Frequency Relationships. The annual maximum summer and winter discharges discussed above were then compared, and a single set of annual maximum discharges were determined. This standardized set of discharges between the Salt River and Painted Rock Dam were ordered and plotted on log-probability paper, using median plotting positions. A consistent family of discharge frequency curves was constructed for points of interest in the reach, using the curve for the Salt River above the Gila River confluence as a guide. A sample frequency curve is shown on plate 15, and the results of the frequency analysis are summarized in table 23.

8-03. PROJECT CONDITIONS (Stage II). Discharge frequency relationships for the Salt River through the City of Phoenix and the Gila River between the Salt River and Painted Rock Dam under project conditions depend foremost on the design of the project. The elements, either structural or reregulatory, have certain characteristics which affect flooding differently. This topic will be addressed in two distinct phases--conceptual project design and how the particular design affects the frequency curve.

a. Project Design. During Stage II the elements were designed such that they were able to control the design floods (the 50-year, 100-year, and standard project floods) to target discharges of 50, 100, 150, and 200 thousand cfs through the City of Phoenix. The 50-year and 100-year floods were determined from balanced hydrographs based on a frequency analysis of reservoir inflow.

(1) Structural Elements.

(a) Single element projects were screened to determine their hydrologic feasibility by comparing the combined uncontrolled streamflow components of the design flood to the target discharge. For example, if the structure was a new Verde River flood control dam, the Salt River component was combined with the local flow below the Verde structure to see if it was less than or equal to the target discharge. If this discharge was greater than the target, the element could not serve as a single element project for that design flood and target. This initial screening process was followed for every element. After selection of the single element projects, they were sized by determining what flood control pool would be required for given outlet sizes. Since no design criteria or cost data were available, an attempt was made to minimize the flood control pool required. Sufficient head to produce large flood control releases would be available due to water already stored in the conservation pool. This philosophy was also extended to dual element alternatives, and later to reregulation.

(b) Multiple Elements. No screening was necessary for multiple element projects, since they could always be adequately sized to control the design floods to any of the targets. The philosophy expressed above, maximum outlets and minimum flood control pool, was again used. But, because another degree of freedom had been introduced due to the additional element, another assumption was required to permit a single solution for each possibility in the matrix (3 by 4, 3 design floods and 4 design targets). Again, in lieu of complete economic data with which to optimize design, the system of flood control reservoirs were designed not only to minimize the individual reservoir space required, but also to minimize the total flood control space in both reservoirs. This was accomplished through an iterative design procedure using the real-time interactive version of the HEC-5 program (HEC-5R) to provide graphics and to streamline decision-making.

(c) Stage II design results are shown in Appendix II.

(2) Reregulation. This process involved dedication of varying amounts of flood control among the three elements considered--Roosevelt, Horseshoe, and Bartlett Dam--as described in para. 1-03.b. The philosophy of

design was to minimize not only flood control space, but also structural modifications, and duration of dedication of conservation space for flood control, and to prevent any encroachment into storage space between the top of the gates and the top of the dam, which is reserved for dam safety. Using this approach, systems of elements and designs were analyzed iteratively until a compatible solution was reached for each alternative in the 3 by 4 design matrix. Some design constraints could not be met because of the limited available flood control space on the Verde River (see table 16 for viable alternatives). Because of uncertainty as to the relative worth of storage space versus cost of outlets, more than one design was developed for alternatives requiring very large amounts of flood control space. The additional designs used large outlets, or even involved spillway modifications, to restrict required space. Designs were again analyzed through use of HEC-5R. Stage II results are displayed in Appendix II.

b. Project Condition Frequency Analysis.

(1) Balanced Hydrographs. The existing conditions discharge frequency curves were determined by converting non-homogenous streamflow record into a standardized set of existing conditions discharges along the Salt and Gila Rivers through use of a period-of record analysis of inflow to the reservoirs (para. 8-02). For Stage II of CAWCS, the number of alternative projects, both structural and reregulatory, for each of the designs in the 3 by 4 design matrix was far too unwieldy to analyze on such a rigorous basis. Instead, balanced hydrographs were used to modify the existing conditions frequency curves. The balanced hydrographs were generated from recorded inflow to Roosevelt and Horseshoe Dams, and should produce results similar to period-of-record simulations. The existing conditions frequency curves were modified as follows:

(a) Events less than or equal to the design target discharge were considered as "non-damaging" and thus remain the same as for existing conditions.

(b) Events less than or equal to the design flood, and greater than the target discharge, were set equal to the design target. (Example-- given a 50-yr design with a 100,000 cfs target: a flood of 150,000 cfs is less than the peak 50-yr design flood, 175,000 cfs; therefore, it was set equal to the target discharge of 100,000 cfs).

(c) Events greater than the design flood could be represented accurately by routing discrete balanced hydrographs for frequencies greater than the design through the proposed project. (Example--a 100-year design would be evaluated by running the 200-and 500-year balanced hydrographs through the design system.)

This type of analysis compared all projects on an equal basis.

(2) Salt River through the City of Phoenix. As stated previously, each proposed project alternative was analyzed through use of balanced hydrographs to determine the peak discharges for n-year floods under project conditions. The values were plotted as n-year events and a smooth curve drawn

for events greater than design. The remainder of the curve is based on the previous discussion, 8-03. b. (1), (a), (b). An example is shown in Appendix II, as well as a summary of the with project peak discharges for various return periods.

(3) Gila River between the Salt River and Painted Rock Dam. The with project frequency floodflows in the Salt River were routed to the Gila River. A comparison was then made between n-year flows under existing conditions and under project conditions for the Salt River above the Gila River, and the corresponding n-year discharges in the Gila River were then reduced by the difference. The reduced n-year flows were plotted for each design alternative, and smooth curves drawn through the with project data (typical curve, Appendix II.). A summary of the with project peak discharges for various return periods is also shown in Appendix II.

8-04. INTERVENING DRAINAGE. The drainage areas on the Salt and Verde Rivers below the furthest upstream existing dams, Roosevelt and Horseshoe, were analyzed separately in Stage I. The methods used and results are detailed in Appendix I. *In general the approach taken was to determine the peak and* volume of runoff which emanated entirely below Roosevelt and Horseshoe Dams.

There is gaged streamflow data on the Verde River below Tangle Creek (above Horseshoe Dam), below Bartlett Dam, and at Scottsdale. In addition there is streamgage information on the major tributary of the lower Verde, Sycamore Creek. Also, there is record for the Salt River near Roosevelt, and below Stewart Mountain Dam, as well as on Tonto Creek above Gun Creek. Finally, indirect peak discharge measurements made by the United States Geological Survey (USGS) are available for some significant events along tributaries of the lower Salt and Verde Rivers. Besides the streamflow record, there is published record of storage in SRP reservoirs by both the USGS and SRP.

This collection of data was analyzed to provide estimates of the difference between operational releases from the most upstream reservoirs and the flow at the downstream stations on both the Salt and Verde stems. In the case of Verde River releases, routing methods were employed to account for the channel losses encountered between Horseshoe Dam and the gage near Scottsdale.

On the Salt River this type of accounting was unnecessary since the tailwater from the downsteam lake extends to the upstream outlets; therefore there is no channel routing, only reservoir routing on the Salt.

The difference between upstream release and downstream discharge, i.e., side drainage or local flow, was determined using procedures described here:

(a) direct computation when upstream and downstream flows were available;

(b) direct computation and adjustment for duration, e.g. if only daily flows were available, peaks were estimated from peak versus 1-day relationships determined for the stream;

(c) indirect computation of reservoir releases using inflow record and change in storage (Note: $O_{AVG} = I_{AVG} - \Delta S/\Delta T$); and

(d) indirect computation of local flow using correlation techniques between mainstem gaged discharges and/or correlation with tributary discharges.

Based upon these techniques, the ensuing annual maximum discharges were ranked and ordered, and a frequency analysis was performed upon them, after adjustment of the length of record for the March 1978 flood. This flood on the intervening drainage was estimated to be the greatest event since 1916 based upon available streamflow and precipitation records. Following this the local flow discharge frequency results for the Salt and Verde Rivers above the confluence were combined to provide a consolidated frequency analysis for discharges on the Salt River below the Verde River confluence. The combination of the separate frequency curves was done using joint probability techniques to determine the probability of an event, E_2, occurring on the Salt River given an event, E_1, occurring on the Verde River. Durations other than peak flow were determined and balanced hydrographs were computed. These balanced n-year hydrographs, were then routed down the Salt River thru the City of Phoenix using Modified Puls storage routing to determine the n-year discharges at various locations. Employing a similar technique to that used in combining the Salt with the Verde River, Indian Bend Wash discharges were combined with Salt River discharges. There are no other sources of significant lateral inflow to the Salt River because of planned or constructed Phoenix flood control projects. Frequency curves established for local flow are presented in Appendix I.

8-05. PROJECT CONDITIONS (STAGE III). Frequency analyses for project elements at the Stage II level were based on inflow to the proposed system coupled with coincident local flow from the intervening drainages. Because of the large difference between existing conditions discharges and local flows, it was first believed that flows from the intervening drainages would be insignificant. However, it became apparent that this assumption might not be valid for frequency discharges under project conditions, especially for projects which provide a high level of protection (SPF), and reduce the downstream discharge to small target flows (50,000 cfs). These extreme cases have the effect of making large releases more infrequent. Due to this shift in the discharge frequency curve for project conditions, the local flow frequency curve becomes relatively more important (plate 16). Stage III hydrology, therefore, evaluated the impact of local flow on the with project discharge frequency.

a. Project Design. The design floods (50-year, 100-year, and SPF) were expanded to include the 500-year flood in an attempt to optimize the outlet and flood pool sizes for the structural alternatives. In addition the New Roosevelt-Cliff system was redesigned based on economic analysis of Stage II designs. This information indicated the flood control system should minimize outlet size and maximize flood pool size to provide the least cost. The only alternatives affected by these economic indicators were those providing SPF level of protection. Reregulatory projects were not analyzed to provide 500-year protection due to their limited size, and were not subject to reanalysis for economic optimization for the same reason. The structural alternatives carried forward to Stage III were New Roosevelt-Cliff (NRNB) and a large confluence site (ORME).

(1) 500-year level of Protection. The upstream alternative, NRNB, was designed to attempt to control the 500-year flood to target discharges of 50,000, 100,000, 150,000, and 200,000 cfs through the City of Phoenix. Outlets were restricted to one-half the target discharge at both New Roosevelt and Cliff to minimize structural costs. However, it was determined during computer simulation of the system that a 50,000 cfs target was unattainable due to the magnitude of contemporaneous inflow below the flood control reservoirs. This local flow component of the 500-year flood was 90,000 cfs. Therefore, a target of 90,000 cfs was selected as the minimum attainable discharge at the Salt-Verde confluence (CP-40). The proposed confluence site dam (ORME) was not subject to this restriction in target discharge since it was located below the Salt-Verde confluence and could be sized to regulate both upstream and local inflow to the desired target discharges. A summary of the design sizes is provided in table 24.

(2) SPF level of Protection. NRNB was redesigned to control the SPF to the target flows of 50, 100, 150, and 200 thousand cfs at the confluence while minimizing outlet sizes. This was accomplished thru use of the HEC-5R computer program for simulating reservoir operation as previously done during Stage II. However, the outlets were fixed at maximum capacity of one-half the downstream target for each flood control element (New Roosevelt and Cliff) and the flood pools sized accordingly. The resulting redesigns are also shown in table 24.

b. Project Condition Frequency Analysis.

(1) Salt River. There were two new or revised sets of designs addressed at Stage III as discussed above - 500-year and SPF level of protection to 50,000 100,000 150,000 and 200,000 cfs. The project condition discharge frequency curves for the 500-year designs were determined through use of balanced hydrographs as in Stage II. The redistribution of storage and revised outlets for the SPF designs were not expected to have any significant impact on the frequency of discharges at CP-40. The level of protection (therefore the extent to which flows could be controlled) and the target remained unchanged. Flows greater than the design flood would involve the spillways which would be unchanged. Therefore, project conditions frequency analyses for New Roosevelt-Cliff for the SPF designs were unchanged from Stage II. However, a more detailed investigation incorporating local flow into the analysis was undertaken and is described below.

Local Flow. To account for the probability of inflow below the proposed flood control reservoirs, a methodology was developed based on the four scenarios presented below which completely describe the possible combinations of reservoir releases (or spills) and local flow. The scenarios developed are mutually exclusive and describe the maximum events occurring within each water year. Most important, these scenarios allow for completion of the discharge frequency analysis for project conditions while directly utilizing the component frequency studies previously developed - local flow (Stage I) and upstream releases or project conditions (Stage II).

Case 1: $Q_D = Q_{u/s} + Q_L$, where Q_D = the downstream discharge
$Q_{u/s}$ = the upstream release
Q_L = the local flow

Case 2: $Q_{u/s} > Q_L$, therefore $Q_D = Q_{u/s}$

Case 3: $Q_{u/s} = Q_L$, therefore $Q_D = Q_{u/s} = Q_L$

Case 4: $Q_{u/s} < Q_L$, therefore $Q_D = Q_L$

In case 1 the upstream release, $Q_{u/s}$, and the local flow, Q_L, are concurrent, therefore additive. However, in cases 2 through 4, $Q_{u/s}$ and Q_L are not concurrent, therefore they do not combine, and the downstream peak, Q_D, is the greater of the two. The discharge frequency curves for the project conditions in Stage II were based upon the annual maximum release or spill, $Q_{u/s}$, plus coincident local flow, Q_L, which has been defined above as case 1. Since Stage II results included all annual maximum upstream releases or spills, and $Q_D = Q_{u/s}$ for both cases 2 and 3, then Stage II results apply to not only case 1, but also cases 2 and 3. In addition the local flow frequency analysis in Stage I was equivalent to case 4. The remaining objective, then, was to develop a combined set of frequency curves, which include not only upstream spills or releases combined with any coincident local inflow (cases 1 through 3), but also local flow alone (case 4).

The analysis to determine the frequency of upstream releases had been approached differently than to determine the frequency of local flow. The upstream release frequency curve (Stage II) was based on adjusting the existing conditions discharge frequency relationship to account for project sizes. The local flow discharge frequency relationship (Stage I) was developed by combining the Salt River and Verde River local flow frequency curves analytically (para. 8-04). The periods of record for existing conditons, local flow on the Verde River, and local flow on the Salt River are of different lengths. Use of the recorded streamflow data only would not provide an adequate representation of the project conditions frequency discharges because of the disparity in available record length. In addition the number of alternative projects addressed during Stage III required a generalized approach which could be modified with ease for each alternative. Because of this and the fact that local flow frequency for the Salt River below the Verde River had been developed analytically, the same approach was extended to determine the combined probability of discharges in the Salt River below the Verde River for upstream and local flow. As discussed above, the adjustment for combined probability involved only cases 1 and 4, i.e. adjustment of the Stage II upstream project condition frequency curve for instances when local flow exceeds upstream flow, but is not contemporaneous.

The analytical adjustment was achieved by examining two extremes for cases 1 and 4. First (referred to hereafter as "dependent" analysis) it was hypothesized that discharges with identical probabilities always occurred within the same water year. For instance, the 200-yr upstream flood, $Q_{u/s}$, and 200-yr local flood, Q_L, would occur in the same water year. Under these circumstances the annual maximum downstream flow, Q_D, would be the greater discharge. Therefore the combined curve would be composed of the greater of the upstream and local flows. This first extreme represents the minimum combined frequency curve. An example is shown on plate 16. The reason that this represents a combined minimum is that it is very unlikely such a sequence of events could occur, and any deviation from such a sequence would result in a series of annual maxima which is greater than or equal to the dependent series.

The other extreme, the combined maximum frequency curve, was generated by hypothesizing that the upstream events and local events were not linked by probability, i.e. they occurred "independently" of each other. In such a case the probability of a given event occurring downstream, e.g. 100,000 cfs, is the sum of the probability of $Q_{u/s}$ or Q_L occurring alone, since the probability of the event occurring locally, given the identical event occurring upstream, is zero (based on the independent hypothesis). An example of the maximum extreme is also shown on plate 16. This sequence represents a maximum because the probability of these events being unrelated, i.e. both occur, but at different times is very low. Not only does this assumption imply that both events occur independently, but also that the new series contains 2N events for N-years. Obviously some events upstream will supplant the local flow in the annual maximum series and vice versa. Also, as the severity of storms, and thus subsequent floods, increases over the Salt-Verde drainage, the storms become larger in areal extent, as well as intensity. Consequently, events with low probability of occurrence tend to occur during the same storm, both upstream and locally, and combine in some way (CASE 1). Any deviation between the hypothetical assumption, complete independence, and the more likely occurrence - either water year overlap or same flood combination of $Q_{u/s}$ and Q_L will result in a annual maximum discharge series and a discharge frequency relationship, which is less than the maximum.

Based upon these minimum and maximum frequency relationships, the more likely occurrence, i.e. a relationship between the extremes, was delineated to be the final with project frequency curve for each design considered (plate 16). The project conditions frequency relationships for NRNB alternatives were determined by combining the upstream release frequency relationships (Stage II) with the local flow frequency relationship (Stage I) at the Salt River below the Verde River (CP-40). The confluence site alternatives would be located near CP-40, and thus would control local flow at that location within their flood pool. Therefore, no adjustment was required to the discharge frequency curves for ORME at CP-40. However, an adjustment was made for discharge frequency relationships for ORME designs below Tempe Bridge (Indian Bend Wash) in an identical manner as to NRNB at CP-40. The resulting curves for Orme alternatives were then routed thru the Salt River to the Gila River using pre-established peak discharge relationships between various concentration points within the reach. Side inflow below Indian Bend Wash was considered insignificant due to the flood control structures constructed or being built in the Phoenix vicinity. The NRNB frequency curves were routed to the Gila River by combining the upstream release frequency relationships with the local flow frequency relationship at each point of interest. The upstream and local flows were routed separately because of the difference in respective volumes, which results in different degrees of attenuation. Results are displayed in table 26 and on plates 18a, 19a, and 20a.

(2) Gila River. Discharge frequency curves for project conditions for the Gila River were determined by adjusting the Gila River existing conditions frequency curves based on the difference in discharges between Salt River project conditions and existing conditions frequency curves. The approach taken was to consider the flow in the Gila River below the confluence with the Salt River (lower Gila, CP-1310) as the combination of inflow from the Salt River (CP-113) plus the Gila River above the Salt River (upper Gila):

$$Q_{\text{lower Gila}} = Q_{\text{Salt}} + Q_{\text{upper Gila}}$$

Then the actual project conditions discharges in the lower Gila, $Q'_{\text{lower Gila}}$, would equal the combination of project conditions discharge from the Salt, Q'_{Salt}, and flow from the upper Gila:

$$Q'_{\text{lower Gila}} = Q'_{\text{Salt}} + Q_{\text{upper Gila}}.$$

The with and without project equations were subtracted and solved for the $Q'_{\text{lower Gila}}$:

$$Q_{\text{lower Gila}} - Q'_{\text{lower Gila}} = Q_{\text{Salt}} - Q'_{\text{Salt}}, \text{ and}$$

$$Q'_{\text{lower Gila}} = Q_{\text{lower Gila}} - \Delta Q_{\text{Salt}},$$

where Q_{Salt} = the reduction in discharge in the Salt River above the Gila River (CP-113) under project conditions.

This equation served as the basis for the follow hypothesis - that the discharge for a given probability in the lower Gila under project conditions equals the discharge for that same probability in the lower Gila under existing conditions minus the reduction in discharge in the Salt River for project conditions for that probability:

$$Q'(Pr_i)_{\text{lower Gila}} = Q(Pr_i)_{\text{lower Gila}} - \Delta Q(Pr_i)_{\text{Salt}},$$

Pr_i = given probability.

This was done because it was evident from the available systematic and historic discharge records that most large flows in the lower Gila River resulted from Salt River floods. If the hypothesis proved accurate, the determination of with project discharge frequency relationships for the lower Gila River would be greatly simplified. The potential for error under this hypothesized situation increases as the Salt River target discharge decreases and the level of protection increases, due to the resulting decrease in most lower Gila project condition flows, including events with infrequent recurrence. Because of this, upper Gila flows which were not concurrent with Salt River discharges may become the more severe event for a given water year, or cause a shift in sequence (ranking) of annual maxima, thus yielding results which vary from those predicted through use of the simplified probability discharge equation above.

To test the applicability of the equation for determination of project condition probability discharges in the lower Gila, a worst case was examined, an SPF design to control the release to 50,000 cfs at the Salt-Verde confluence. The entire period-of-record annual maximum series for the Salt River at CP-113 and the lower Gila River at CP-1310 for existing conditions were modified for project conditions, and the results were compared to those generated by the proposed equation. The outcome was a nearly identical series of annual maxima, which corroborated use of the equation. Therefore, the proposed probability discharge equation was used to generate analytical project conditions discharge frequency relationships. As shown, a period-of-record systematic adjustment was unnecessary for even this worst case. Nevertheless, the largest of the systematic floods were also adjusted to provide additional data, and these results used in conjunction with analytical adjustments. An example of this procedure is shown on plate 17. The resulting project conditions discharge frequency curves were a combination of both the analytical and systematic adjustments, and are shown in table 26, and on plates 18b, 19b, and 20b.

8-06. SEASONAL FREQUENCY. In addition to an analysis of annual maximum discharges in the reach of the Gila River between the Salt River and Painted Rock Dam, an analysis of the probability of flooding in this reach on a seasonal basis was required. The overall objective of the seasonal frequency analysis was to determine the probability of inundation of crops and (or) croplands for various durations. The critical term is "inundation". To provide the information required to determine the duration of inundation, it was necessary to determine the magnitude of discharges which were equalled or exceeded for the durations of interest. The discharges that are equalled or exceeded for the given durations will be referred to as threshold duration discharges, or TDD. This type of data differs from ordinary duration data which presents the "average" discharge for the duration (plate 21), but does not present the threshold which determines the amount of inundation for each duration.

Note that the "average" discharge for each duration is always greater than the threshold discharge, unless the hydrograph is horizontal for that entire duration. Therefore, the results of this analysis are not volume-frequency relationships in the usual sense.

a. Seasons. For purpose of this study, the seasons were defined from an agricultural viewpoint as two "wet" seasons--December through April, and July through September--and two "dry" seasons--May through June, and October through November. The seasons were not rigid and could extend forward or backward across the boundaries, and the floods were analyzed on this basis.

(1) December through April. The data for this season were essentially the same as that generated for the peak discharge frequency relationships for the Gila River below the Salt River, since those floods were nearly always winter events due to spills from SRP reservoirs (occasionally augmented by spills from Coolidge and Waddell Dams). This set of existing conditions discharges was adjusted to exclude events which fell outside the Dec-April season. The remaining peak data were ordered and plotted on log-probability paper, using median plotting positions. Since the data still fit

the existing conditions annual frequency curve, the annual curve was used to represent the December-April season for return periods greater than five years.

(2) May through June. Data for this period were based on simulated Salt River discharge routed to Gila River for existing conditions and Gillespie record adjusted to exclude flow from SRP, Coolidge, and Waddell Dams. The combined results were ordered and plotted as before, and a smooth peak discharge frequency curve was fit to the data.

(3) July through September. Floods occurring in the summer "wet" season result from both thunderstorms and general summer storms. Data at Gillespie Dam were available since 1920; these data were screened to determine whether the streamflow was generated from controlled areas prior to reservoir construction and adjusted accordingly. There would have been no spills under existing conditions. The data adjusted for existing conditions were ordered and plotted, and a smooth frequency curve fit to the peak data.

(4) October Through November. Floods during this season typically result from late summer or early winter general storms. Reservoirs in the basin are ordinarily at their lowest seasonal level due to heavy summer demands, high evaporation rates, and low inflow; therefore, most, but not all, flow in the river stems from the areas not controlled by reservoirs, e.g., San Pedro River. Gillespie Dam record, adjusted to exclude any contribution from areas controlled under existing conditions, was combined with results of simulated SRP, Coolidge, and Waddell spills to produce a record in the Gila River below the Salt River. The peak curve was estimated in the same manner as the two preceding seasons.

b. Duration of Inundation. To determine the duration of inundation, it was necessary to determine the threshold duration discharge. To facilitate accomplishment of this task, it was decided to proceed in the following steps:

(1) establish ordinary volume frequency relationships for each season using the peak curves as guides; the required durations were peak, 1-day, 2-day, 3-day, and 6-day.

(2) generate balanced hydrographs for the n-year floods for each season using the volume-frequency data.

(3) determine TDD's for the 1-day, 2-day, 3-day, and 6-day durations from the balanced hydrographs.

(4) plot the n-year TDD's and fit smooth curves to the data, using the peak curves as guides.

c. Damaging Discharges. It had been determined that the non-damaging discharge in this reach of the Gila River was 60,000 cfs. Damaging discharges for various return periods up to 500 years are displayed in table 27. Plate 22 indicates the results for the Dec-April season.

8-07. SUMMARY OF DISCHARGE FREQUENCY ANALYSIS.

a. The methods used to determine discharge frequency relationships varied from location-to-location on the Salt, Verde, and Gila Rivers. Analytical procedures presented in the Water Resources Council Guidelines (reference 12.) were followed wherever possible. However, it was necessary to employ graphical methods at many locations due to two major factors, upstream control, and the number of zero discharge years. Expected probability adjustments were not made to the frequency curves because analytical techniques were not always used. The length of the streamflow record used in this study was 92 years; hence, there would be very little difference between expected and computed probability. No confidence limits were drawn for the graphical curves. Analytical techniques were used to develop the balanced hydrographs for the combined coincident inflow to Roosevelt and Horseshoe Dams, and confidence limits are shown in table 28. These limits cannot be extended to discharges downstream from the SRP reservoirs, however, because of the uncertainty of reservoir conditions. But actual short-term operation of the reservoirs during large flood events would not change the uncertainty, because the reservoirs have limited flood control capability.

b. Because of the long streamflow record, the completeness of the analysis, and the corraboration of final results with balanced hydrographs and natural flows, the frequency analyses in this report are considered reliable.

IX. SAFETY OF DAMS.

9.01. GENERAL. The safety-of-dams (SOD) issue arose during Stage II when both PMF (Corps of Engineers) and IDF (Burec) were determined to be considerably in excess of original design peaks and volumes for SRP reservoirs. The SOD investigation was conducted in series with CAWCS hydrology by BUREC under their dam safety program. Although it was apparent from the outset of the SOD study that results could impact on CAWCS hydrology, until recommendations were made, no measure of the impact could be made. In mid-Stage III the BUREC nominated two potential solutions to the dam safety issue:

(1) SOD_1 - Under this alternative the gated spillway at Roosevelt Dam (either a new or modified structure) would be modified such that it could safely pass 92,000 cfs at the top of the conservation pool. The gates would be operated to maintain the NWS until inflow exceeded 92,000 cfs, above which point the release would be maintained at 92,000 cfs. The existing three lower reservoirs on the Salt River would then operate such that outflow equalled inflow up to spillway capacity, if the water surface had reached or exceeded the NWS. Spillways for the Verde River Dams, Horseshoe and Bartlett, would be modified/enlarged to safely pass the IDF without overtopping.

(2) SOD_2 - This alternative involved suppression of the IDF on both the Salt River and the Verde River. The modification of Roosevelt and operation of the four Salt River reservoirs would be identical to that described for SOD_1. However, the Verde River modifications involved construction of a new dam, Cliff, to replace Horseshoe and suppress IDF releases so that the spillway capacity of the existing downstream structure, Bartlett Dam, would not be exceeded. The new dam would have a conservation pool equivalent to that of its predecessor, and a perched spillway. Releases would be made thru a gated outlet, when the stored water reached the NWS, to maintain that elevation until inflow exceeded outlet capacity. Flow in excess of the surcharge pool would be released over an emergency spillway. Bartlett Dam would release outflow equal to inflow above the existing NWS.

The proposed SOD solutions were then incorporated into CAWCS hydrology to determine their effect upon discharge frequency. The SOD_1 and SOD_2 alternatives were analyzed on a stand-alone basis, as well as in combination with CAWS flood control alternatives. These combined analyses were performed for SOD as first-added and last-added components. The procedure can be summarized as follows:

First Added- The effect of SOD on existing conditions frequency analaysis is evaluated as a stand-alone alternative, and then evaluated in combination with CAWCS flood control.

Last Added - The effect of CAWCS flood control was evaluated singly, and then in combination with SOD spillway fixes.

Thus there are four possible adjustments to the existing conditions frequency analysis, SOD, SOD + CAWCS, CAWCS, CAWCS+SOD. It is apparent that SOD+CAWCS is equivalent to CAWCS+SOD from the hydrologic viewpoint. CAWCS flood control was described in section 8. It remained, then, to evaluate SOD alone and in conjunction with CAWCS flood control. At the time SOD recommendations were made the only remaining CAWCS flood control plans were NRNB or ORME to control the SPF to 50,000 cfs at the Salt-Verde confluence, and Reregulation. How SOD alone, and SOD plus the remaining CAWCS alternatives, affect discharge frequency analysis is discussed below.

9.02. SOD_1.

a. 1st Added. The period-of-record inflows to the SRP reservoirs were examined to determine the effect of spillway modification and SOD operation. The conservation space for the six SRP reservoirs remained the same as for existing conditions. Therefore, the system spilled at the same times and in the same volumes. Also, because peak flows on the Verde River did not exceed the spillway capacity of Horseshoe or Bartlett Dams, the actual spillway operations for Verde under SOD_1 were unchanged. Therefore, the only adjustment made to outflows was due to suppression to 92,000 cfs at Roosevelt. The operation with the modified spillway resulted in an increase in probability discharges from existing conditions between 10-yr and 150-yr recurrence periods due to the improved hydraulic characteristics of the new or modified Roosevelt spillway. Beyond this range, suppression of inflow to 92,000 cfs resulted in a decrease in probability discharges (plate 23). The actual probability analysis was performed in an identical manner to that used for existing conditions, based on the "adjusted" period-of-record annual maxima. A summary of frequency discharges for the SOD_1 fix is included in table 26.

b. Last Added. At the time that this approach was to be considered, CAWCS planners detemined that the SOD_1 alternative was inferior to SOD_2, and would not be studied further.

9-03. SOD_2.

a. 1st Added. Existing conditions results were modified in the same manner as under SOD_1. For this alternative, though, inflows at both New or modified Roosevelt and Cliff were suppressed. The cumulative effect of this suppression resulted in a reduction in probability discharges throughout the frequency range, especially for events with recurrence intervals greater than 10-years (table 26 and plate 24). A comparison between SOD_1 and SOD_2 is shown on plate 25.

b. Last Added.

(1) NRNB. The results of the frequency analysis described in section 8 for the New Roosevelt - Cliff combination controlling the SPF to 50,000 cfs at the confluence were readdressed for SOD_2. Since the changes only affect spillway flow, i.e. inflows greater than the design flood, SPF, the only portion of the frequency curve which had to be adjusted was the range above the design probability. As before, balanced hydrographs were employed since all recorded inflows were less than the design flood. Suppression of inflow at both New Roosevelt and Cliff reduced discharges at CP-40 for recurrence intervals greater than 100 years as shown on plate 26a. Local flow

was incorporated into the analysis using procedures identical to those in the project conditions analysis, and the results were routed to the Gila River below the Salt River (plate 26b). A summary of frequency discharges is included in table 26.

(2) ORME. For the downstream CAWCS flood control alternative (ORME), the SOD_2 fix resulted in a series adjustment to probability discharges. For this case, not only were outflows modified, but inflows as well. Period-of-record inflows to Orme were altered since it is downstream of the SOD_2 fix. The Orme flood control space and spillway characteristics then further reduced inflow. The degree of modification resulting from SOD_2 was determined by simulating the outflow from SRP reservoirs with SOD_2 for balanced hydrographs greater than the design flood, and performing a subsequent flood control operation upon these upstream releases at ORME. Downstream local flows were combined with ORME releases in the same manner as in section 8, and subsequently routed to the Gila River below the Salt River. The results are summarized in table 26 and compared on plates 27, and 28. In addition the NRNB and ORME alternatives with SOD_2 are compared on plate 29.

9-04. REREGULATION. SOD_1 and SOD_2 were also studied in conjunction with reregulation of SRP facilities to achieve control of the SPF. The results of the frequency analyses for SOD_1 and SOD_2 were identical since a flood control outlet with 95,000 cfs capacity was required at Cliff (SOD_2) to prevent surcharge into the SOD pool. The operation of this outlet effectively rescinded suppression at Cliff. (During reregulation design operations at either Horseshoe (SOD_1) or Cliff (SOD_2) it became necessary to pass a 95,000 cfs peak discharge, thereby making SOD_1 and SOD_2 equivalent when in a reregulation context.)

a. DESIGN SIZING.

(1) 90,000 cfs Target. The maximum control possible for SPF protection was desired. This was achieved by assuming Roosevelt Dam could store all the SPF component inflow (maximum available space about 1.4 million acre-feet which is greater than SPF component volume) and minimizing release from the Verde. The resulting discharge, 90,000 cfs at the Salt-Verde confluence, was then established as the minimum target discharge. Sizing was achieved by allocating various blocks of conservation space to flood control along with appropriate outlets, and simulating a flood control operation using HEC-5R. Several alternatives were possible, but an optimum size was decided upon by results of previous economic analyses. Results are included in table 25.

(2) 150,000 cfs Target. Another target discharge, 150,000 cfs, was suggested by CAWCS planners. Since this flow was greater than the minimum achievable, a solution was sought directly. An iterative approach, as in the latter phase of the 90,000 cfs target, was used. Design sizes for this target are also displayed in table 25.

b. DISCHARGE FREQUENCY. The final designs were incorporated into a computer simulation model with SOD_1 or SOD_2 fixes (they were equivalent due to the outlet change at Cliff, i.e. SOD_2, for reregulation of SPF). As during previous studies, the effect of Roosevelt suppression would only be evident for spillway flow, i.e. floods greater than the design. To evaluate this effect, Balanced Hydrographs of inflows greater than or equal to the 200-year flood were utilized in conjunction with the computer simulation model. Local inflow was again accounted for in the manner described under section eight and the results routed to the Gila River below the Salt River. Results are shown in table 26, and on plates 30 and 31.

X. UPSTREAM FLOOD CONTROL WITH REGULATORY STORAGE AT THE CONFLUENCE.

10-01. GENERAL. One of the remaining CAWCS flood control alternatives during the latter phase of Stage III was upstream protection (NRNB, or New Roosevelt-Cliff) along with a small confluence structure which would be built for regulatory storage of Central Arizona Project (CAP) water. The NRNB alternative was intended to control the SPF to a 50,000 cfs target at the Salt-Verde confluence. It was felt that the existence of a confluence structure could reduce local flow peaks as well as attenuate upstream flood control releases.

10-02. DISCHARGE FREQUENCY ANALYSIS. Releases for the entire range of probability from NRNB combined with local inflow to the confluence structure were routed over the proposed confluence dam spillway to determine peak releases. In addition non-coincident local flows were combined on a probability basis in a similar manner as described in para. 8-05. The Central Arizona Project (CAP) regulation pool was assumed to be full during these studies, although this situation is a worst case. A refined analysis would result in probability discharges which were less than or equal to those in table 26 and plate 32. An operation plan would be necessary, however, to make such refinements. In the same manner described in para. 8-05, releases from the regulatory structure were combined with local inflow at locations of interest along the Salt River and routed to the Gila River below the Salt River. The results of this analysis, meanwhile, provide a reasonably accurate representation of discharge frequency below a regulatory confluence structure with upstream flood control.

XI. FLOOD CONTROL OPERATION CRITERIA

11-01. GENERAL. Criteria by which the proposed CAWCS flood control reservoirs should be operated are presented herein. These criteria were developed based on the algorithm in the HEC-5 computer simulation model which was used to size the storage-outlet requirements for the proposed flood control systems. The objectives of these criteria are to prevent downstream flooding by limiting flow at the Salt-Verde confluence to a pre-established target discharge, and to simultaneously keep the flood control system in "balance". Balance in this context represents the state in which the relative flood control space available at each flood control dam is equal. The criteria discussed within generally apply to the dual system, NRNB, and the mono-system, ORME. In the case of the single flood control reservoir system, balance is neither a problem nor an objective; the only objective in this type of system is controlling total inflow, SRP releases plus local flow, to the target discharge.

Preliminary flood control operation criteria were established for operation during an actual flood event and for yearly fluctuation in dedicated flood control storage allocations. A detailed analysis of seasonal flood control requirements is being undertaken for proposed projects within the ongoing 1982 CAWCS hydrology.

11-02. FLOOD EVENT OPERATION.

a. Total Release from a Reservoir or Reservoir System.

1. If the water surface (WS) > normal water surface (NWS) or bottom of flood pool, then

   flood control release (FCR_i) = 0.

2. If WS $\geq$ NWS, and

   total reservoir inflow (ΣI_i)* + local flow (Q_L) at the target location $\leq$ target discharge (Q_T), then

   total flow at the target location = $\Sigma I_i + Q_L$ in which

   $\Sigma FCR_i = \Sigma I_i$ for NRNB, or

   $FCR = \Sigma I_i + Q_L$ for ORME

3. If WS $\geq$ NWS, and

   ΣI_i* + Q_L > Q_T, then

   $\Sigma FCR_i = Q_T - Q_L$ for NRNB, or

   $FCR = Q_T$ for ORME.

b. Specific Outflow from a Reservoir System (NRNB)

1. Combined flood control release = total flood control release,

$$FCR_1 + FCR_2 = \Sigma FCR_i$$

*If all reservoirs do not have WS NWS, then I_i is only the inflow to the reservoirs in which this is true.

2. Case a.2. above,

$$FCR_i = \text{Individual Inflow, } I_i^*$$

3. Case a.3. above,

Individual flood control releases are made from a dual flood control reservoir system based on the individual flood control space available, the total flood control space available, and the total flood control system release allowed at the time period considered.

The total flood control release from the system is determined, as outlined in a.3 above, based on total reservoir inflow plus local flow, to insure that the target discharge is not exceeded at the target control point. The sum of the individual flood control releases is equal to the total flood control release.

After determination of the total flood control release, that amount is apportioned between the reserovirs based on the relative proportion of individual available flood control space to individual total flood control space, so that the reservoir with less relative available space makes the greater release. An attempt is made to balance the relative available flood control space between the reservoirs.

Example. At the end of time period 10 the water surface at both reservoirs 1 and 2 is above the NWS. Inflows are 50,000 and 30,000 cfs to the respective reservoirs such that total reservoir inflow equals 80,000 cfs. Local flow at the target location has been determined to be 10,000 cfs. Since the total reservoir inflow plus local flow equals 90,000, which exceeds the target discharge of 50,000 cfs, then the total flood control release equals the target discharge minus the local flow, or 40,000 cfs.

The total available flood control space in the hypothetical system when empty is 750,000 ac-ft. Reservoir 1 can hold 500,000 ac-ft, while reservoir 2 can store 250,000 ac-ft in their respective flood control pools. At the end of period 10 only 500,000 ac-ft is available in the entire system (500,000/750,000=66.7% Available). An attempt is then made to apportion flood control releases so that this percentage of available space is "balanced" between the reservoirs.

*Ibid

Reservoir 1 (maximum capacity = 500,000 ac-ft) has 400,000 ac-ft available, while reservoir 2 (maximum capacity = 250,000 ac-ft) has only 100,000 ac-ft of available space. The relative amounts of available flood control storage are then 80% and 40% respectively for reservoirs 1 and 2. Since the total relative available space is 67%, reservoir 2 (40% available) releases more water than reservoir 1 (80% available) in an attempt to achieve a proportional balance (67%) in each. The releases from reservoirs 1 and 2 are prorated according to reservoir size and relative space; reservoir 1 release is equal to the inverse proportion of relative space (40%/80%) times the inverse proportion of total space (250,000 ac-ft/500,000 ac-ft) times the release from reservoir 2, so that the combined releases ($FCR_1 + FCR_2$) equal the predetermined 40,000 cfs. Therefore, release from reservoir 2 equals 4 times the release for reservoir 1, i.e. reservoir 1 release equals 8,000 cfs and reservoir 2 release equals 32,000 cfs. However, the releases would be constrained by outlet capacity, so that if the maximum outlet capacity at reservoir 2 were only 25,000 cfs, the releases from reservoirs 1 and 2 would be 15,000 and 25,000 cfs respectively. In addition smoothness of operation to prevent excessive gate changes might cause deviation from computed releases. The operation prescribed is based on hourly adjustments to reservoir release.

If

FCR_i = release from reservoir $_i$,

ΣFCR_i = total release,

SA_i = space available at reservoir $_i$,

ΣSA_i = total space available,

$SMAX_i$ = flood pool at reservoir $_i$,

$\Sigma SMAX_i$ = total flood pool,

$\% \ SA_i$ = % space available at reservoir $_i$,

$\% \ \Sigma SA_i$ = total % space available, where

$\% \ \Sigma SA_i = \dfrac{\Sigma SA_i}{\Sigma SMAX_i}$, and

$\% \ SA_i = \dfrac{SA_i}{SMAX_i}$, then

$$FCR_1 = \frac{\% SA_2}{\% SA_1} \times \frac{SMAX_2}{SMAX_1} \times FCR_2.$$

For this example,

$FCR_1 + FCR_2 = 40{,}000$ cfs, and

$FCR_1 = \frac{40}{80} \times \frac{250{,}000}{500{,}000} \times FCR_2$. Then,

$FCR_1 = \frac{1}{4} \times FCR_2 = 8{,}000$ cfs and

$$FCR_2 = 32{,}000 \text{ cfs.}$$

11-03. YEARLY FLOOD CONTROL OPERATION.

The purpose of the yearly flood control operation is to maintain maximum flood control storage during the critical flood producing months of December, January, February, and March, and to gain additional water storage during the remaining spring and summer runoff period. In other words, yearly flood control operations are established such that the storage space set aside for flood control can be used for more than one purpose.

The yearly operation of the flood control pool would vary depending on the time of year and on design criteria. Maximum flood control space would be maintained during the months of December, January, February, and March and additional conservation space would be available during the months of April through September. The transition period for recovering flood control space was selected as 1 October to 1 December. Flood control space (minus 200,000 ac-ft/100,000 on both Salt and Verde proposed reservoirs) would be converted to water conservation space beginning 1 April and extending through September. These transition months were chosen because only one November spill and three April spills would have occurred under existing conditions during the period-of-record. To eliminate shared storage during these months would eliminate any shared pool benefit.

During the October to December transition period, reservoir water levels would be lowered in order to achieve maximum flood control space. During the April to October transition period, flood control releases would be decreased or stopped, thus permitting water levels to rise in the reservoir to the maximum elevation for that date. This type of operation would allow for additonal water storage during the 31 March to 1 December period.

BIBLIOGRAPHY

1. Letter from Salt River Project, dated 14 June 1979.

2. Interim Report on Survey for Flood Control, Gila and Salt Rivers, Gillespie Dam to McDowell Dam Site, Arizona, Corps of Engineers, 4 December 1957.

3. Los Angeles District Flood Hydrograph Package, Hydrologic Engineering Section, Los Angeles District, Corps of Engineers.

4. HEC-5 Simulation of Flood Control and Conservation Systems, Hydrologic Engineering Center, Davis, Ca.

5. Ground Water Recharge, Appendix 3, Phoenix Urban Study, Feb. 1979.

6. Hydrometeorological Report (HMR) No. 49, Probable Maximum Precipitation Estimates, Colorado and Great Basin Drainages, U.S. Department of Commerce, NOAA, U.S. Department of Army, Corps of Engineers, Sept. 1977.

7. Hydrometeorological Report (HMR) No. 36, Interim Report--Probable Maximum Precipitation in California, National Weather Service, Hydrometeorological Branch, October 1969.

8. Letter from Hydrometeorological Branch, National Weather Service, 24 April 1979.

9. Factors Affecting Sediment Yield and Measures for the Reduction of Erosion and Sediment Yield, Pacific Southwest Inter-Agency Committee, Oct. 1968.

10. Summary of Reservoir Sediment Deposition Surveys Made in the United States through 1965, Misc. Publication No. 1143, Agricultural Research Service, USDA, May 1969.

11. Letter from Salt River Project, dated 20 Dec 1978.

12. Guidelines for Determining Floodflow Frequencies, Bulletin 17A, Water Resources Council, November 1977.

13. Design Memorandum No. 1, General Design Memoranda, Phases I, II, Indian Bend Wash, Los Angeles District, U.S. Army Corps of Engineers, October 1973 and May 1975.

14. 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.

15. NOAA Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume VIII-Arizona, U.S. Department of Commerce, 1973.

TABLE 1

STRUCTURES WHICH AFFECT RUNOFF

Dam	Stream	Effective D.A. (sq. mi.)	Reservoir Capacity: Conservation storage at top of gates (ac-ft)	Reservoir Capacity: At top of dam (ac-ft)	Year of closure
Roosevelt	Salt R.	5,830	1,382,000	1,555,000	1911
Horse Mesa	Salt R.	5,935	245,000	264,000	1927
Mormon Flat	Salt R.	6,095	58,000	64,000	1926
Stewart Mtn.	Salt R.	6,221	70,000	73,000	1930
Horseshoe	Verde R.	5,657	131,000	182,000	1946
Bartlett	Verde R.	5,852	178,000	199,000	1939
Coolidge	Gila R.	12,886	1,066,000	1,360,000	1928
Waddell	Agua Fria	1,459	158,000	176,000	1927

Other structures that control runoff: (Insignificant effect on Salt R and/or Gila River discharges)

Whitlow Ranch Dam

Cave Buttes Dam

Tat Momolikot Dam

Central Arizona Project

Arizona Canal Diversion Channel

Indian Bend Wash

Paradise Valley Detention Dike

TABLE 2

PROJECT CONDITIONS, ELEMENTS, AND COMBINATIONS

	Elements	Combinations
1. Structural		
	New Roosevelt = NR	NR + NH
	New Horseshoe = NH	NR + NB
	New Bartlett = NB	
	Cliff = NB	
	Confluence = ORME	
2. Reregulation		
	Roosevelt = R	R + B
	Horseshoe = H	R + B + H
	Bartlett = B	

Note: Flood control design at Cliff = Flood control design at New Bartlett.

TABLE 3

REREGULATION

WATER SURFACE ELEVATIONS ASSOCIATED

WITH VARIOUS AMOUNTS OF FLOOD CONTROL SPACE

Dam	Space Dedicated (Ac-ft)	Elevation (ft)	Feature
Horseshoe	0	2026	NWS*
	63,000	2000	Spillway Crest
	126,000	1957	Maximum F.C.
Bartlett	0	1798	NWS*
	106,000	1748	Spillway Crest
	133,000	1726	
	173,000	1665	Maximum F.C.
Roosevelt	0	2136	NWS*
	140,000	2127	
	270,000	2120	Spillway Crest
	320,000	2117	
	370,000	2113	
	470,000	2107	
	510,000	2104	

*NWS-Normal Water Surface

TABLE 4

COMPARISON OF FLOODS OF RECORD

SIMULATED VS. NATURAL RESULTS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

Water Year	Month	Simulated existing conditions flow (cfs)	Flow that would have occurred without reservoirs (cfs)
1891	Feb	271,000	300,000[a]
1905	Apr	113,000	115,000[b]
1906	Nov	134,000	220,000[b]
1916	Jan	145,000	164,000[b]
1920	Feb	138,000	155,000[b]
1927	Feb	82,000	123,000[b]
1932	Feb	86,000	117,000[b]
1938	Mar	77,000	115,000[a]
1941	Mar	132,000	170,000[a]
1966	Dec	47,000	85,000[a,c]
1978	Mar	119,000	260,000[a]
1979	Dec	157,000	235,000[b]
1980	Feb	201,000	241,000[b]

a USGS

b COE

c This value is for the 31 Dec. peak; the peak which would have occurred without reservoirs for the previous flood, 23 Dec, was 117,000 cfs.

TABLE 5

SAMPLE EFFECTIVE RAINFALL COMPUTATION FOR SPF

6-Hr Period	Incremental incident rainfall (in.)	Accumulative incidental rainfall (in.)	% Accum. incidental rainfall	.35 x % Accum. incidental rainfall	Accumulative effective rainfall (in.)	Incremental effective rainfall (in.)
1	0	0	0	0	0	0
2	0	0	0	0	0	0
3	0.09	.09	3.169	1.109	.001	.001
4	0	.09	3.169	1.109	.001	0
5	0.62	.71	25.000	8.750	.062	.061
6	0.19	.90	31.690	11.092	.100	.038
7	0.38	1.28	45.070	15.775	.202	.102
8	0.05	1.33	46.831	16.391	.218	.016
9	0	1.33	46.831	16.391	.218	0
10	0.43	1.76	61.972	21.690	.382	.164
11	0	1.76	61.972	21.690	.382	0
12	0	1.76	61.972	21.690	.382	0
13	0.09	1.85	65.141	22.799	.422	.040
14	0.24	2.09	73.592	25.757	.538	0
15	0	2.09	73.592	25.757	.538	0
16	0	2.09	73.592	25.757	.538	0
17	0.28	2.37	83.451	29.208	.692	.154
18	0.19	2.56	90.141	31.549	.808	.115
19	0.14	2.70	95.070	33.275	.898	.091
20	0	2.70	95.070	33.275	.898	0
21	0	2.70	95.070	33.275	.898	0
22	0	2.70	95.070	33.275	.898	0
23	0.14	2.84	100.000	35.000	.994	.096
	2.84					

TABLE 6

UNIT GRAPH PARAMETERS

GILA RIVER BASIN

Subarea No.	D.A. (sq. mi.)	L (mi.)	Lca (mi.)	SLOPE (ft./mi.)	Bn value	S-Graph
201	2,830	160	92	35	0.047	Over 1,500 sq. mi.
202	4,060	203	98	20	0.047	Over 1,500 sq. mi.
203	2,280	120	45	55	0.047	Over 1,500 sq. mi.
204	3,730	146	34	45	0.047	Over 1,500 sq. mi.
205	5,300	176	76	20	0.047	Over 1,500 sq. mi.
206	5,222	158	57	25	0.054	Over 1,500 sq. mi.
207	1,780	52	23	30	0.053	Over 1,500 sq. mi.
208	1,492	65	34	15	0.062	Under 1,500 sq. mi.
209	Area above Whitlow Ranch Dam					
210	Negligible contributing area--SCS structures control flow					
211	1,459	73	39	65	0.045	Phoenix Mtn.
212	Area above McMicken Dam (assumed to be rebuilt)					
213	Area above New River Dam (assumed to be constructed)					
214	Area above Adobe Dam (assumed to be completed)					
215	540	52	27	20	0.031	Phoenix Valley
216	760	62	32	40	0.047	Phoenix Valley
217	1,450	105	47	65	0.047	Under 1,500 sq. mi.
218	1,990	107	37	40	0.047	Over 1,500 sq. mi.
219	1,195	54	24	45	0.047	Under 1,500 sq. mi.

TABLE 7

UNIT GRAPH PARAMETERS

SALT RIVER BASIN

Subarea No.	D.A. (sq. mi.)	L (mi.)	Lca (mi.)	SLOPE (ft./mi.)	Bn value	S-Graph
21	4,344	140	56	50	0.048	Over 1,500 sq. mi.
22	1,486	63	19	95	0.048	Phoenix Mtn.
7	105	13	2	235	0.047	Phoenix Mtn.
8	160	20	11	175	0.047	Phoenix Mtn.
6	126	16	7	260	0.047	Phoenix Mtn.
5	59	6	2	110	0.047	Phoenix Mtn.
1	5,483	176	65	25	0.047	Over 1,500 sq. mi.
31	174	18	5	145	0.047	Phoenix Mtn.
3	194	19	6	120	0.047	Phoenix Mtn.
41	251	21	15	10	0.047	Phoenix Mtn.
42	211	40	20	110	0.047	Phoenix Mtn.
9	190	18	9	10	0.030	Phoenix Valley
10	48	8	4	5	0.030	Phoenix Valley
11	82	8	5	5	0.030	Phoenix Valley
12	49	6	4	5	0.030	Phoenix Valley

Total D.A. = 12,962 sq. mi.

TABLE 8

PULS ROUTING (DT = 1 HR)

SALT RIVER PROJECT SYSTEM

		Storage (ac.-ft.) in Channel Reaches			
		Stewart Mtn to Granite Reef CP 4-CP 8	Tangle Ck to Horseshoe CP 50-CP 5	Horseshoe to Bartlett CP 5-CP 6	Bartlett to Granite Reef CP 6-CP 8
Discharge (cfs)	Reach Length	12.0 mi.	5.0 mi.	20.5 mi.	26.9 mi.
0		0	0	0	0
1,000		405	200	845	1,210
5,000		1,350	700	2,800	3,540
10,000		2,270	1,200	4,600	5,700
20,000		3,850	1,900	7,700	9,780
30,000		5,240	2,400	9,930	13,000
40,000		6,550	3,030	12,400	16,300
50,000		7,860	3,500	14,000	18,900
60,000		9,020	3,900	16,000	22,200
70,000		10,190	4,300	17,700	24,800
80,000		11,200	4,750	19,400	27,700
90,000		12,400	5,100	20,900	30,300
100,000		13,500	5,460	22,400	32,600
150,000		18,900	6,970	28,600	45,700
200,000		23,300	8,340	33,800	58,700
300,000		38,600	12,500	50,700	105,700
400,000		68,500	17,600	73,000	172,000

TABLE 9

PULS ROUTING (DT=1 HR)
LOWER SALT RIVER (BELOW VERDE RIVER CONFLUENCE)

Discharge (cfs)	Reach Length	Storage (ac-ft) and Percolation Loss (cfs) in Channel Reaches				
		Granite Reef to Gilbert Road	Gilbert Road to Tempe Bridge	Tempe Bridge to Central Avenue	Central Avenue to 67th Avenue	67th Avenue to above Gila River Confluence
		CP8-CP109	CP109-CP110	CP110-CP11	CP111-CP112	CP112-CP113
		6.4 mi.	16.3 mi.	7.7 mi.	7.7 mi.	4.6 mi.
0[1]	Storage	3,500	4,900	4,100	4,200	2,500
	Percolation	445	1,180	750	680	320
50,000	Storage	11,100	24,500	13,200	9,400	6,900
	Percolation	445	1,180	750	680	320
100,000	Storage	15,300	38,000	26,600	14,400	10,600
	Percolation	445	1,180	750	680	320
150,000	Storage	19,100	50,500	34,400	39,000	14,200
	Percolation	540	1,750	870	1,440	580
200,000	Storage	23,500	61,500	40,100	46,900	17,600
	Percolation	860	2,010	970	1,790	720
250,000	Storage	28,200	71,400	49,300	55,300	21,100
	Percolation	925	2,170	1,140	2,000	850
300,000	Storage	37,800	80,600	60,900	58,600	24,500
	Percolation	690[2]	2,340	1,310	2,230	950
9 400,000	Storage	69,000	100,000[2]	90,000[2]	66,000[2]	31,500[2]
	Percolation	755[2]	2,510[2]	1,480[2]	2,460[2]	1,050[2]

[1] Model interpolates between 0 and 50,000 cfs. Storage/percolation values for a discharge = 0 are used to account for high initial infiltration as well as gravel pits with available storage below channel invert.

[2] Extrapolated value.

TABLE 10

PULS ROUTING (DT=6 HR)

UPPER GILA RIVER (FROM SAN FRANCISCO RIVER CONFLUENCE TO COOLIDGE DAM)

		Storage (ac.-ft.) and Percolation Loss (cfs) in Channel Reaches		
		San Francisco River Confl to San Simon Ck.	San Simon Ck. Confl to Midway to Coolidge Dam	From Midway to Coolidge Dam
		CP1202 - CP1223	CP1223 - CP12041	CP12041 - CP1204
Discharge (cfs)	Reach Length	35.2 mi.	28.9 mi.	29.5 mi.
0(1)	Storage	0	0	0
	Percolation	0	0	0
10,000	Storage	3,420	4,040	4,190
	Percolation	1,000	1,400	990
50,000	Storage	17,100	20,200	20,900
	Percolation	1,000	1,400	990
100,000	Storage	34,200	40,400	41,900
	Percolation	1,000	1,400	990
150,000	Storage	51,400	60,600	62,800
	Percolation	1,000	1,400	990
200,000	Storage	67,900	80,800	83,800
	Percolation	1,000	1,400	990

(1)No storage/percolation for zero discharge since river is perrenial within these reaches.

TABLE 10 (Continued)

PULS ROUTING (DT=6 HR)
UPPER GILA RIVER (FROM COOLIDGE DAM TO SALT R. CONFL.)

		Storage (ac-ft) and Percolation Loss in Channel Reaches				
		Coolidge Dam to Midway to Buttes	From Mid-way to Buttes	Buttes to Midway to Santa Cruz River Confl	From Midway to Santa Cruz River Confl	Santa Cruz River Confl to Salt River Confl
		CP1204 - CP12051	CP12051 - CP1205	CP1205 - CP12101	CP12101 - CP1210	CP1210 - CP1310
Discharge (cfs)	Reach Length	24.7 mi.	24.7 mi.	45.0 mi.	45.0 mi.	12.1 mi.
0[1]	Storage	0	0	0	0	0
	Percolation	112	455	1,480	2,940	3,540
20,000	Storage	7,000	6,700	20,400	16,500	14,000
	Percolation	112	455	1,480	2,940	3,540
60,000	Storage	15,800	20,300	61,100	49,900	41,800
	Percolation	126	465	1,740	3,020	3,540
100,000	Storage	23,300	33,100	113,500	76,300	69,500
	Percolation	133	475	2,690	3,110	3,550
300,000	Storage	62,000	100,000	360,000	228,000	220,000
	Percolation	177	490	3,350	3,260	3,830

[1] Non-zero percolation used in these reaches to account for high initial infiltration since flow is intermittent. No storage at zero discharge.

TABLE 11

PULS ROUTING (DT=6 HR)
LOWER GILA RIVER (BELOW CONFL WITH SALT RIVER)

		Storage (ac-ft) and Percolation Loss (cfs) in Channel Reaches				
		Salt River Confl to Waterman W.	Waterman W. to Hassayampa River	Hassayampa River to Gillespie	Gillespie to Midway to Painted Rock	From Midway to Painted Rock
		CP113 - CP1216	CP1216 - CP1217	CP1217 - CP1218	CP1218 - CP12191	CP12191 - CP1219
Discharge (cfs)	Reach Length	15.8 mi.	26.1 mi.	8.5 mi.	15.3 mi.	15.3 mi.
0(1)	Storage	0	0	0	0	0
	Percolation	660	830	330	1,000	1,290
50,000	Storage	38,000	42,300	8,750	35,000	42,600
	Percolation	660	830	330	1,000	1,290
150,000	Storage	84,900	86,900	19,400	79,300	95,600
	Percolation	1,060	1,550	475	1,400	1,800
200,000	Storage	104,000	106,000	23,800	96,100	116,000
	Percolation	1,200	1,750	530	1,510	1,940
320,000	Storage	148,000	144,000	34,200	131,000	159,000
	Percolation	1,490	2,050	650	1,710	2,220

(1) Non-zero percolation used in these reaches to account for high initial infiltration since flow is intermittent. No storage at zero discharges.

TABLE 12

STANDARD PROJECT STORM RAINFALL

SALT RIVER BASIN

Subarea No.	Salt-Verde Confluence SPS Total Incident Rainfall (in.)	Salt-Verde Confluence SPS Total Effective Rainfall (in.)	Painted Rock SPS Total Incident Rainfall (in.)	Painted Rock SPS Total Effective Rainfall (in.)
21	6.86	2.40	6.80	2.38
22	8.00	2.80	7.40	2.59
7	9.06	3.17	7.90	2.77
8	6.53	2.29	6.70	2.35
6	6.06	2.12	4.60	1.61
5	4.20	1.47	2.50	0.88
1	5.68	1.99	5.00	1.75
31	5.34	1.87	8.10	2.84
3	5.54	1.94	6.50	2.28
41	4.94	1.73	4.20	1.47
42	7.76	2.72	5.40	1.89
9	3.00	1.05	1.90	0.67
10	2.60	0.91	1.70	0.60
11	2.50	0.88	1.60	0.56
12	2.40	0.84	1.60	0.56

TABLE 13

STANDARD PROJECT STORM RAINFALL

GILA RIVER BASIN

Subarea No.	Painted Rock SPS Total Incident Rainfall (in.)	Total Effective Rainfall (in.)
201	3.17	1.10
202	2.18	0.76
203	2.60	0.52
204	4.69	1.41
205	3.50	0.88
206	4.33	0.95
207	2.47	0.49
208	2.20	0.44
211	5.10	1.53
215	2.00	0.40
216	1.15	0.23
217	3.65	0.91
218	2.55	0.51
219	0.68	0.14

TABLE 14

STANDARD PROJECT FLOOD

PEAK DISCHARGES

EXISTING CONDITIONS

Location	Peak Discharge, cfs
Salt River at:	
CP 40 - Below confl. w/Verde River	295,000
CP 109 - Gilbert Road	292,000
CP 110 - Tempe Bridge	289,000
CP 111 - Central Ave.	285,000
CP 112 - 67th Ave.	283,000
CP 113 - Above confl. w/Gila River	281,000
Gila River at:	
CP 1310 - Below confl. w/Salt River	312,000
CP 1216 - Below confl. w/Waterman W.	315,000
CP 1217 - Below confl. w/Hassayampa River	309,000
CP 1218 - Gillespie Dam	311,000
CP 12191- Midway From Gillespie to Painted Rock	304,000
CP 1219 - Painted Rock Dam	298,000

TABLE 15

STANDARD PROJECT FLOOD PEAK DISCHARGE

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

WITH PROJECT

Design flood	Element/ combination	Target discharges, cfs 200,000	150,000	100,000	50,000
100-YR	NR	289,000	205,000	NA	NA
	NH	296,000	248,000	NA	NA
	NB	290,000	272,000	NA	NA
	NR+NB	275,000	260,000	260,000	165,000
	NR+NH	257,000	252,000	270,000	156,000
	ORME	290,000	275,000	240,000	203,000
	R	255,000	NA	NA	NA
	R+B	NR	197,000	NA	NA
	R+H+B	NR	NR	210,000 (1)	160,000
	R+H+B	NR	NR	233,000 (2)	160,000
50-YR	NR	N/A	318,000	NA	NA
	NH	N/A	264,000	260,000	NA
	NB	N/A	272,000	270,000	NA
	NR+NB	N/A	299,000	299,000	285,000
	NR+NH	N/A	299,000	298,000	285,000
	ORME	N/A	290,000	290,000	290,000
	R+H+B	N/A	270,000	270,000	260,000

NOTE: SPF discharge equals target discharge for SPF design
- (1) With flood control outlets
- (2) With extra flood control storage
- NA - Control of the design flood (100-year or 50-year) to the target discharge was not achieved.
- NR - Not required since reregulation with fewer elements was able to control the design flood to the target discharge.
- N/A - Not applicable since 50-yr flood is only 175,000 cfs.

500-year designs were not analyzed for ability to control SPF, since SPF is less than design flood (500-yr.)

Note:
- NR = New Roosevelt
- NH = New Horseshoe
- NB = New Bartlett
- NB = Cliff
- ORME = Confluence
- R = Roosevelt
- H = Horseshoe
- B = Bartlett

TABLE 16

PROJECT CONDITIONS

ELEMENTS AND COMBINATIONS

WHICH ACHIEVED DESIGN OBJECTIVE

Design flood	Target discharge, cfs 200,000	150,000	100,000	50,000
500-Yr.*				
SPF	NR NH NB NR+NB NR+NH ORME R+H+B	NB NR+NB NR+NH ORME R+H+B	NR+NB NR+NH ORME	NR+NB NR+NH ORME
100-YR	NR NH NB NR+NB NR+NH ORME R	NR NH NB NR+NB NR+NH ORME R+B	NR+NB NR+NH ORME R+H+B	NR+NB NR+NH ORME R+H+B
50-YR	EXISTING CONDITIONS IS 175,000 CFS	NR NH NB NR+NB NR+NH ORME R+H+B	NH NB NR+NB NR+NH ORME R+H+B	NR+NB NR+NH ORME R+H+B

Note:

NR = New Roosevelt
NH = New Horseshoe
NB = New Bartlett
NB = Cliff
ORME = Confluence
R = Roosevelt
H = Horseshoe
B = Bartlett
*All 500-yr designs included only NR+NB (Cliff), and ORME

TABLE 17

PROBABLE MAXIMUM PRECIPITATION

WINTER

(TIME INTERVAL=1 HR)

ABOVE HORSESHOE DAM

.04	.04	.05	.05	.05	.05	.05	.05	.05	.05
.05	.05	.05	.05	.05	.06	.06	.07	.07	.08
.08	.10	.10	.10	.10	.10	.10	.10	.10	.10
.10	.10	.10	.10	.10	.10	.10	.10	.10	.15
.15	.15	.15	.15	.15	.15	.15	.15	.15	.15
.20	.20	.20	.20	.25	.25	.30	.30	.35	.35
.40	.50	.50	.60	.50	.40	.35	.30	.25	.20
.12	.08								

TOTAL DEPTH = 11.4 INCHES

ABOVE ROOSEVELT DAM

.05	.05	.05	.05	.05	.05	.06	.06	.06	.06
.06	.07	.07	.07	.08	.08	.08	.08	.08	.09
.10	.10	.10	.10	.10	.10	.10	.10	.10	.12
.13	.15	.15	.15	.15	.15	.15	.15	.15	.15
.15	.15	.15	.20	.20	.20	.20	.20	.25	.25
.25	.30	.30	.30	.30	.40	.40	.40	.50	.50
.60	.60	.70	.70	.60	.60	.50	.35	.30	.25
.25	.15								

TOTAL DEPTH = 15.0 INCHES

ABOVE CONFLUENCE OF SALT AND VERDE RIVERS

.02	.03	.03	.04	.04	.04	.04	.04	.05	.05
.06	.06	.06	.06	.07	.08	.07	.06	.06	.06
.07	.07	.07	.07	.07	.08	.08	.08	.09	.10
.10	.10	.10	.10	.10	.10	.11	.13	.14	.16
.14	.12	.11	.11	.12	.12	.12	.12	.12	.13
.13	.13	.13	.15	.19	.24	.29	.31	.33	.34
.35	.39	.47	.41	.36	.32	.29	.26	.23	.19
.14	.09								

TOTAL DEPTH = 8.8 INCHES

TABLE 18

PROBABLE MAXIMUM PRECIPITATION

SUMMER

(TIME INTERVAL=1 HR)

ABOVE HORSESHOE DAM

.03	.03	.03	.03	.04	.04	.04	.04	.05	.05
.06	.06	.07	.08	.09	.09	.07	.06	.06	.05
.04	.05	.07	.07	.10	.10	.10	.10	.10	.10
.10	.10	.10	.11	.11	.12	.14	.15	.16	.17
.19	.19	.19	.17	.12	.07	.10	.11	.13	.16
.20	.21	.23	.25	.29	.30	.32	.32	.32	.35
.44	.50	.70	.90	.60	.46	.33	.28	.22	.21
.20	.18								

TOTAL DEPTH = 12.4 INCHES

ABOVE ROOSEVELT DAM

.05	.05	.05	.05	.05	.05	.05	.05	.05	.05
.06	.06	.08	.09	.11	.13	.10	.09	.09	.08
.06	.07	.09	.09	.13	.14	.14	.14	.15	.15
.15	.15	.15	.16	.16	.18	.18	.20	.22	.21
.20	.19	.16	.16	.16	.17	.17	.18	.21	.23
.26	.26	.27	.27	.34	.39	.46	.51	.52	.58
.60	.69	1.01	1.35	.78	.67	.50	.38	.33	.29
.25	.25								

TOTAL DEPTH = 17.1 INCHES

ABOVE CONFLUENCE OF SALT AND VERDE RIVERS

.03	.04	.05	.06	.06	.06	.06	.06	.07	.07
.07	.07	.07	.07	.08	.07	.06	.05	.06	.06
.07	.07	.07	.07	.08	.08	.09	.09	.08	.08
.08	.08	.08	.08	.08	.10	.14	.18	.22	.19
.15	.12	.10	.10	.10	.11	.13	.16	.17	.17
.18	.19	.19	.20	.22	.25	.29	.32	.35	.37
.41	.49	.70	.55	.45	.40	.36	.32	.28	.24
.19	.11								

TOTAL DEPTH = 11.6 INCHES

TABLE 19

PROBABLE MAXIMUM FLOOD

PEAK DISCHARGE AND VOLUME ESTIMATES

	Inflow Peak (cfs)	Total Inflow Volume (ac-ft)
Horseshoe Dam	670,000	1,540,000
Roosevelt Dam	1,000,000	2,100,000
Confluence of Salt and Verde Rivers	925,000	3,000,000

TABLE 20

SEDIMENT SURVEYS AND RESULTS

Dam	Effective drainage area (sq. mi.)	Date of survey	Period between surveys (yrs.)	Storage capacity (ac-ft)	Difference between surveys (ac-ft)	Average annual sediment accumulation for period (ac-ft/sq mi/yr)	Sediment inflow (ac-ft/yr)
Horseshoe	5,618	Oct 1950	-	142,830	-	-	-
		Nov 1963	13.1	139,238	3,592	0.049	275
		Oct 1978	14.9	131,427	7,811	0.093	522
	(Total per:	1950-1978)	28.0	-	11,403	0.072	404
Bartlett	194	Nov 1950	-	179,548	-	-	
		Jan 1964	13.2	178,488	1,060	0.414	80
		Jun 1977	13.5	178,185	303	0.116	23
	(Total per:	1950-1977)	26.7	-	1,360	0.263	51
Roosevelt	5,760	May 1909	-	1,522,200	-	-	
		Dec 1914	5.7	1,495,460	26,740	0.814	4,689
		Oct 1916	1.8	1,460,150	35,310	3.406	19,619
		Sep 1925	8.9	1,425,813	34,337	0.670	3,859
		Jan 1935	9.3	1,418,013	7,800	0.146	841
		Jan 1939	4.0	1,398,430	19,583	0.850	4,896
		Jan 1946	7.0	1,381,580	16,850	0.418	2,408
	(Total per:	1909-1946)	36.7	-	140,620	0.665	3,830
Waddell	1,444	Apr 1928	-	184,500	-	-	-
		Feb 1941	12.9	176,456	8,044	0.432	624
Coolidge	11,900	Nov 1928	-	1,267,447	-	-	-
		Feb 1935	6.3	1,233,335	34,112	0.455	5,414
		Jan 1937	1.9	1,231,350	1,985	0.088	1,047
		Jan 1947	10.0	1,209,953	21,397	0.180	2,142
	(Total per:	1928-1947)	18.2	-	57,494	0.265	3,154

TABLE 21

SEDIMENT PRODUCTION ESTIMATES

FOR VERDE RIVER ALTERNATIVES

Dam site	Equivalent drainage area (sq. mi.)	Average annual sediment yield (ac-ft/yr)	Sediment volume in 100-years (ac-ft)
New Horseshoe	1,000	650	65,000
Cliff (Horseshoe breached)	1,081	700	70,000
New Bartlett	195	130	13,000

TABLE 22

GILA RIVER BASIN

STREAMGAGE RECORD

Ref. No.	USGS No.	Location	Period of Record
1.	09468500	San Carlos River nr. Peridot	(1930-1979)
2.	09469000	San Carlos Reservoir Inflow	(1914-1975)
3.	09469500	Gila River below Coolidge Dam	(1899-1905)*, (1914-1979)
4.	09470000	Gila River at Winkelman	(1942-1979)
5.	09470500	San Pedro River at Palominas	(1930-1933), (1933-1941), (1950-1979)
6.	09471000	San Pedro River at Charleston	(1913-1934)*, (1935-1978)
7.	09471550	San Pedro River at Tombstone	(1967-1979)
8.	09471800	San Pedro River nr. Benson	(1966-1976)
9.	09472000	San Pedro River nr. Redington	(1943-1947), (1950-1978)
10.	09472500	San Pedro River nr. Mammoth	(1931-1941)
11.	09473400	San Pedro River nr. Winkelman	(1890), (1962-1966), (1966-1978)
12.	09474000	Gila River at Kelvin	(1911-1979)
13.	09479500	Gila River nr. Laveen	(1940-1946), (1948-1978)
14.	09482500	Santa Cruz River at Tucson	(1906-1907), (1913), (1915-1978)
15.	09465000	Santa Cruz River at Cortaro	(1940-1947), (1950-1978)
16.	09489000	Santa Cruz River nr. Laveen	(1940-1978)
17.	09497500	Salt River nr. Chrysotile	(1924-1979)
18.	09498500	Salt River nr. Roosevelt	(1913-1978)
19.	09498800	Tonto Creek nr. Gisela	(1964-1975)
20.	09499000	Tonto Creek above Gun Creek	(1940-1979)
21.	09499500	Tonto Creek nr. Roosevelt	(1914-1941)
22.	09500500	Salt River at Roosevelt	(1904-1908)
23.	09501000	Salt River at and below Roosevelt	(1910-1979)
24.	09502000	Salt River below Stewart Mtn.	(1930-1979)
25.	09502500	Salt River at McDowell	(1895-1910)*
26.	NA	Salt River at Granite Reef Dam	(1913-1938)
27.	NA	Salt River at Arizona Dam	(1888-1891), (1895)
28.	09508500	Verde River below Tangle Creek	(1945-1979)
29.	09509000	Verde River at Bartlett	(1938-1945)
30.	09510000	Verde River below Bartlett	(1888-1979)
31.	09510200	Sycamore Creek nr. Ft. McDowell	(1960-1979)
32.	09511300	Verde River nr. Scottsdale	(1961-1979)
33.	NA	Verde River nr. McDowell	(1889-1938)*
34.	09513970	Agua Fria River at Avondale	(1960-1967), (1967-1972), (1973-1978)
35.	09517500	Centennial Wash nr. Arlington	(1961-1978)
36.	09519500	Gila River below Gillespie	(1921-1979)

NOTE: *Intermittent record.

TABLE 23

DISCHARGE FREQUENCY VALUES

SALT RIVER AND GILA RIVER

EXISTING CONDITIONS

	Return period						
	500-yr	200-yr	100-yr	50-yr	20-yr	10-yr	5-yr
Salt River at:							
CP 40--Below confl w/Verde River	360,000	290,000	245,000	175,000	141,000	102,000	45,000
CP 109--Gilbert Road	345,000	285,000	230,000	170,000	139,000	100,000	44,000
CP 110--Tempe Bridge	330,000	275,000	215,000	160,000	135,000	93,000	40,000
CP 111--Central Avenue	325,000	265,000	200,000	155,000	130,000	91,000	39,000
CP 112--67th Avenue	315,000	255,000	190,000	150,000	126,000	90,000	38,000
CP 113--Above confl w/Gila River	310,000	250,000	185,000	145,000	125,000	85,000	36,000
Gila River at:							
CP 1310--Below confl w/Salt River	360,000	295,000	250,000	200,000	135,000	95,000	40,000
CP 1216--Below confl w/Waterman W.	350,000	290,000	245,000	195,000	133,000	88,000	39,000
CP 1217--Below confl w/Hassayampa River	340,000	280,000	240,000	190,000	129,000	82,000	38,000
CP 1218--Gillespie Dam	335,000	277,000	235,000	186,000	124,000	78,000	37,000
CP 12191--Midway from Gillespie to Painted Rock	330,000	272,000	230,000	180,000	120,000	75,000	36,000
CP 1219 --Painted Rock Dam	320,000	260,000	220,000	173,000	115,000	70,000	31,000

TABLE 24
PROJECT CONDITIONS
STRUCTURAL ALTERNATIVE DESIGNS

			New Roosevelt		New Horseshoe		New Bartlett		Orme	
Design Flood	Target Discharge (cfs)	Alternative	F.C. Space (ac-ft)	F.C. Outlet (cfs)	F.C. Space (ac-ft)	F.C. Outlet (cfs)	F.C. Space (ac-ft)	F.C Outlet (cfs)	F.C. Space (ac-ft)	F.C. Outlet (cfs)
50-Yr	50,000	Orme	-	-	-	-	-	-	470,000	50,000
		NR + NB	220,000	25,000	-	-	135,000	25,000	-	-
		NR + NH	220,000	25,000	145,000	25,000	-	-	-	-
50-Yr	100,000	Orme	-	-	-	-	-	-	210,000	100,000
		NH	-	-	190,000	30,000	-	-	-	-
		NB	-	-	-	-	225,000	23,000	-	-
		NR + NB	140,000	50,000	-	-	50,000	50,000	-	-
		NR + NH	140,000	50,000	60,000	50,000	-	-	-	-
50-Yr	150,000	Orme	-	-	-	-	-	-	47,000	150,000
		NR	100,000	80,000	-	-	-	-	-	-
		NH	-	-	78,000	60,000	-	-	-	-
		NB	-	-	-	-	69,000	58,000	-	-
		NR + NB	90,000	75,000	-	-	11,000	75,000	-	-
		NR + NH	90,000	75,000	20,000	75,000	-	-	-	-
50-Yr	200,000*	Not Required	-	-	-	-	-	-	-	-

*50-Yr flood for existing conditions is only 175,000 cfs.

TABLE 24 (CONT'D)
PROJECT CONDITIONS
STRUCTURAL ALTERNATIVE DESIGNS

sign ood	Target Discharge (cfs)	Alternative	New Roosevelt F.C. Space (ac-ft)	New Roosevelt F.C. Outlet (cfs)	New Horseshoe F.C. Space (ac-ft)	New Horseshoe F.C. Outlet (cfs)	New Bartlett F.C. Space (ac-ft)	New Bartlett F.C. Outlet (cfs)	Orme F.C. Space (ac-ft)	Orme F.C. Outlet (cfs)
0-Yr	50,000	Orme	-	-	-	-	-	-	750,000	50,000
		NR + NB	550,000	23,000	-	-	245,000	23,000	-	-
		NR + NH	550,000	23,000	255,000	23,000	-	-	-	-
0-Yr	100,000	Orme	-	-	-	-	-	-	420,000	100,000
		NR + NB	350,000	48,000	-	-	145,000	42,000	-	-
		NR + NH	355,000	50,000	145,000	50,000	-	-	-	-
0-Yr	150,000	Orme	-	-	-	-	-	-	190,000	150,000
		NR	325,000	70,000	-	-	-	-	-	-
		NH	-	-	207,000	43,000	-	-	-	-
		NB	-	-	-	-	205,000	38,000	-	-
		NR + NB	240,000	75,000	-	-	71,000	57,000	-	-
		NR + NH	247,000	71,000	74,000	65,000	-	-	-	-
0-Yr	200,000	Orme	-	-	-	-	-	-	46,000	200,000
		NR	154,000	105,000	-	-	-	-	-	-
		NH	-	-	52,000	58,000	-	-	-	-
		NB	-	-	-	-	52,000	75,000	-	-
		NR + NB	130,000	100,000	-	-	30,000	87,000	-	-
		NR + NH	154,000	90,000	57,000	85,000	-	-	-	-

TABLE 24 (CONT'D)
PROJECT CONDITION
STRUCTURAL ALTERNATIVE DESIGNS

Design Flood	Target Discharge (cfs)	Alternative	New Roosevelt F.C. Space (ac-ft)	New Roosevelt F.C. Outlet (cfs)	New Barlett F.C. Space (ac-ft)	New Barlett F.C. Outlet (cfs)	Orme F.C. Space (ac-ft)	Orme F.C. Outlet (cfs)
500-Yr	50,000	Orme					1,425,000	50,000
		NR + NB*	860,000	40,000	360,000	40,000		
500-Yr	100,000	Orme					995,000	100,000
		NR + NB	710,000	46,000	340,000	46,000		
500-Yr	150,000	Orme					610,000	150,000
		NR + NB	520,000	66,000	265,000	66,000		
500-Yr	200,000	Orme					370,000	200,000
		NR + NB	370,000	100,000	235,000	100,000		

*Due to the magnitude of local inflow (uncontrolled inflow below NR, NB), the 500-yr flood could not be controlled to the target discharge of 50,000 cfs. Actual design was therefore predicated upon the minimum achievable 500-yr downstream discharge at CP-40, 90,000 cfs.

TABLE 24 (CONT'D)
PROJECT CONDITION
STRUCTURAL ALTERNATIVE DESIGNS

esign ood	Target Discharge (cfs)	Alternative	New Roosevelt F.C. Space (ac-ft)	New Roosevelt F.C. Outlet (cfs)	New Horseshoe F.C. Space (ac-ft)	New Horseshoe F.C. Outlet (cfs)	New Bartlett F.C. Space (ac-ft)	New Bartlett F.C. Outlet (cfs)	Orme F.C. Space (ac-ft)	Orme F.C. Outlet (cfs)
PF	50,000	Orme	-	-	-	-	-	-	970,000	50,000
		NR + NB[a]	565,000	25,000	-	-	445,000	25,000	-	-
		NR + NH[b]	500,000	45,000	500,000	45,000	-	-	-	-
PF	100,000	Orme	-	-	-	-	-	-	560,000	100,000
		NR + NB[a]	365,000	50,000	-	-	285,000	50,000	-	-
		NR + NH[b]	340,000	90,000	310,000	90,000	-	-	-	-
PF	150,000	Orme	-	-	-	-	-	-	270,000	150,000
		NB	-	-	-	-	315,000	45,000	-	-
		NR + NB[a]	210,000	75,000	-	-	180,000	75,000	-	-
		NR + NH[b]	190,000	95,000	210,000	95,000	-	-	-	-
PF	200,000	Orme	-	-	-	-	-	-	105,000	200,000
		NR	230,00	112,000	-	-	-	-	-	-
		NH	-	-	140,000	88,000	-	-	-	-
		NB	-	-	-	-	115,000	100,000	-	-
		NR + NB[a]	125,000	100,000	-	-	100,000	100,000	-	-
		NR + NH[b]	130,000	100,000	110,000	100,000	-	-	-	-

[a]Optimized results during Stage III for cost
[b]Stage II results which maximized outlets

TABLE 25
PROJECT CONDITIONS
REREGULATION ALTERNATIVE DESIGNS

Design Flood	Target Discharge (cfs)	Elements	Designated F.C. Space (ac-ft)	F.C. Outlet (cfs)	Drawdown Outlet (cfs)
50-Yr	50,000	Horseshoe	126,000	-	10,000
		Bartlett	173,000	-	10,000
	100,000	Roosevelt	270,000	-	-
		Horseshoe	126,000	-	10,000
		Barlett	173,000	-	10,000
	150,000	Roosevelt	270,000	-	-
		Horseshoe	63,000	-	-
		Bartlett	106,000	-	-
100-Yr	50,000	Roosevelt	510,000	50,000	-
		Horseshoe	126,000	50,000	-
		Bartlett	173,000	50,000	-
	100,000	Roosevelt	320,000	45,000	-
		Horseshoe	63,000	-	-
		Bartlett	173,000	20,000	-
	100,000	Roosevelt	462,000	10,000	-
		Horseshoe	63,000	-	-
		Bartlett	173,000	10,000	-
	150,000	Roosevelt	320,000	45,000	-
		Horseshoe	-	-	-
		Bartlett	173,000	10,000	-
	200,000	Roosevelt	370,000	20,000	-
		Horseshoe	-	-	-
		Bartlett	-	-	-
SPF	50,000	UNABLE TO MEET THE TARGETS			
	90,000(1)	Roosevelt	495,000	30,000	-
		Cliff/Horseshoe	126,000	95,000	-
		Bartlett	173,000	70,000	-
	150,000	Roosevelt	520,000	-	10,000
		Horseshoe	63,000	-	
		Bartlett	133,000	75,000	-
	150,000(2)	Roosevelt	140,000	100,000	-
		Horseshoe	126,000	80,000	-
		Bartlett	173,000	50,000	-

TABLE 25 (CONT'D)

Design Flood	Target Discharge (cfs)	Elements	Designated F.C. Space (ac-ft)	F.C. Outlet (cfs)	Drawdown Outlet (cfs)
	150,000 (1)	Roosevelt	125,000	50,000	-
		Cliff/Horseshoe	126,000	95,000	-
		Bartlett	173,000	75,000	-
	200,000	Roosevelt	320,000	30,000	-
		Horseshoe	63,000	-	-
		Bartlett	173,000	30,000	-

Note: 50-yr design/200,000 cfs target is unnecessary run since 50-yr flood through the existing condition produces peak of 175,000 cfs.

(1) W/$SOD_{1,2}$, spillway modification @ Roosevelt, Cliff/Horseshoe to control IDF

(2) The existing gated spillway was upgraded to 100,000 cfs capability @ spillway crest.

TABLE 26-A1

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP-40)

	Alternatives	Design flood (yrs.)	Design target (cfs)	Peak Discharge, cfs — Frequency, yrs. 500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	360,000	290,000	245,000	175,000	141,000	102,000
2. Project Conditions									
a. Structural Elements:	SOD_1	N/A	N/A	297,000	270,000	249,000	220,000	160,000	100,000
New Roosevelt = NR	SOD_2	N/A	N/A	177,000	168,000	159,000	148,000	127,000	90,000
Cliff = NB									
Confluence = ORME	ORME	50	50,000	320,000	230,000	155,000	50,000	50,000	50,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			270,000	200,000	150,000	85,000	50,000	50,000
Regulatory Storage	ORME	50	100,000	345,000	265,000	205,000	100,000	100,000	100,000
at confluence = RS	NRNB			305,000	235,000	170,000	100,000	100,000	100,000
b. Reregulation Elements:	ORME	50	150,000	355,000	270,000	210,000	150,000	141,000	102,000
Roosevelt = R	NRNB			340,000	260,000	210,000	150,000	141,000	102,000
Horseshoe = H									
Cliff = C		50	200,000[a]						
Bartlett = B									

NOTE: N/A = not applicable

[a]Not required, existing conditions discharge = 175,000 cfs

TABLE 26-A2

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP-40)

	Alternatives	Design flood (yrs.)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	360,000	290,000	245,000	175,000	141,000	102,000
2. Project Conditions									
a. Structural Elements:	ORME	100	50,000	250,000	145,000	50,000	50,000	50,000	50,000
New Roosevelt = NR	NRNB			235,000	135,000	85,000	55,000	50,000	50,000
Cliff = NB									
Confluence = ORME	ORME	100	100,000	275,000	180,000	100,000	100,000	100,000	100,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			295,000	200,000	100,000	100,000	100,000	100,000
Regulatory Storage									
at confluence = RS	ORME	100	150,000	295,000	210,000	150,000	150,000	141,000	102,000
	NRNB			320,000	225,000	150,000	150,000	141,000	102,000
b. Reregulation Elements:									
Roosevelt = R	ORME	100	200,000	340,000	255,000	200,000	175,000	141,000	102,000
Horseshoe = H	NRNB			340,000	260,000	200,000	175,000	141,000	102,000
Cliff = C									
Bartlett = B									

NOTE: N/A = not applicable

TABLE 26-A3

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP-40)

		Design flood (yrs.)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	360,000	290,000	245,000	175,000	141,000	102,000
2. Project Conditions	Alternatives								
a. Structural Elements:	ORME	500	50,000	50,000	50,000	50,000	50,000	50,000	50,000
New Roosevelt = NR	NRNB			120,000	92,000	74,000	55,000	50,000	50,000
Cliff = NB									
Confluence = ORME	ORME	500	100,000	100,000	100,000	100,000	100,000	100,000	100,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			130,000	100,000	100,000	100,000	100,000	100,000
Regulatory Storage									
at confluence = RS	ORME	500	150,000	150,000	150,000	150,000	150,000	141,000	102,000
	NRNB			160,000	150,000	150,000	150,000	141,000	102,000
b. Reregulation Elements:									
Roosevelt = R	ORME	500	200,000	200,000	200,000	200,000	175,000	141,000	102,000
Horseshoe = H	NRNB			200,000	200,000	200,000	175,000	141,000	102,000
Cliff = C									
Bartlett = B									

NOTE: N/A = not applicable

TABLE 26-A4

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP-40)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	360,000	290,000	245,000	175,000	141,000	102,000
2. Project Conditions	Alternatives								
a. Structural Elements:	ORME	SPF	50,000	180,000	50,000	50,000	50,000	50,000	50,000
New Roosevelt = NR	ORME			135,000	50,000	50,000	50,000	50,000	50,000
Cliff = NB	(w/SOD_2)								
Confluence = ORME	NRNB			190,000	110,000	78,000	55,000	50,000	50,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			130,000	96,000	76,000	55,000	50,000	50,000
Regulatory Storage	(w/SOD_2)								
at confluence = RS	NRNB			145,000	105,000	80,000	55,000	41,000	41,000
	(w/RS)								
b. Reregulation Elements:									
Roosevelt = R	R+H/C+B	SPF	90,000	180,000	135,000	90,000	90,000	90,000	90,000
Horseshoe = H	(w/$SOD_{1,2}$)								
Cliff = C									
Bartlett = B	ORME	SPF	100,000	210,000	100,000	100,000	100,000	100,000	100,000
	NRNB			190,000	125,000	100,000	100,000	100,000	100,000
	ORME	SPF	150,000	255,000	175,000	150,000	150,000	141,000	102,000
	NRNB			270,000	190,000	150,000	150,000	141,000	102,000
	R+H/C+B			250,000	225,000	150,000	150,000	141,000	102,000
	(w/$SOD_{1,2}$)								
	ORME	SPF	200,000	275,000	205,000	200,000	175,000	141,000	102,000
	NRNB			310,000	230,000	200,000	175,000	141,000	102,000

NOTE: N/A = not applicable

TABLE 26-B1

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP-113)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	310,000	250,000	185,000	145,000	125,000	85,000
2. Project Conditions	Alternatives								
a. Structural Elements:	SOD_1	N/A	N/A	240,000	220,000	205,000	184,000	138,000	92,000
New Roosevelt = NR	SOD_2	N/A	N/A	150,000	145,000	140,000	130,000	110,000	82,000
Cliff = NB									
Confluence = ORME	ORME	50	50,000	258,000	191,000	134,000	56,000	45,000	45,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			230,000	175,000	125,000	56,000	45,000	45,000
Regulatory Storage									
at confluence = RS	ORME	50	100,000	275,000	220,000	170,000	90,000	90,000	85,000
	NRNB			245,000	195,000	150,000	90,000	90,000	85,000
b. Reregulation Elements:									
Roosevelt = R	ORME	50	150,000	280,000	225,000	175,000	130,000	125,000	85,000
Horseshoe = H	NRNB			270,000	215,000	175,000	130,000	125,000	85,000
Cliff = C									
Bartlett = B		50	200,000[a]						

NOTE: N/A = not applicable
[a]Not required, existing conditions discharge = 175,000 cfs

TABLE 26-B2

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP-113)

	Alternatives	Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	310,00	250,000	185,000	145,000	125,000	85,000
2. Project Conditions	Alternatives								
a. Structural Elements:	ORME	100	50,000	205,000	125,000	63,000	45,000	45,000	45,000
New Roosevelt = NR	NRNB			190,000	110,000	56,000	45,000	45,000	45,000
Cliff = NB									
Confluence = ORME	ORME	100	100,000	225,000	155,000	90,000	90,000	90,000	85,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			235,000	170,000	90,000	90,000	90,000	85,000
Regulatory Storage									
at confluence = RS	ORME	100	150,000	240,000	175,000	130,000	130,000	125,000	85,000
	NRNB			260,000	190,000	130,000	130,000	125,000	85,000
b. Reregulation Elements:									
Roosevelt = R	ORME	100	200,000	270,000	210,000	170,000	145,000	125,000	85,000
Horseshoe = H	NRNB			270,000	210,000	170,000	145,000	125,000	85,000
Cliff = C									
Bartlett = B									

NOTE: N/A = not applicable

TABLE 26-B3

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP-113)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	310,000	250,000	185,000	145,000	125,000	85,000
2. Project Conditions	Alternatives								
a. Structural Elements:	ORME	500	50,000	71,000	45,000	45,000	45,000	45,000	45,000
New Roosevelt = NR	NRNB			80,000	53,000	45,000	45,000	45,000	45,000
Cliff = NB									
Confluence = ORME	ORME	500	100,000	100,000	90,000	90,000	90,000	90,000	85,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			90,000	90,000	90,000	90,000	90,000	85,000
Regulatory Storage									
at confluence = RS	ORME	500	150,000	130,000	130,000	130,000	130,000	125,000	85,000
	NRNB			130,000	130,000	130,000	130,000	125,000	85,000
b. Reregulation Elements:									
Roosevelt = R	ORME	500	200,000	170,000	170,000	170,000	145,000	125,000	85,000
Horseshoe = H	NRNB			170,000	170,000	170,000	145,000	125,000	85,000
Cliff = C									
Bartlett = B									

NOTE: N/A = not applicable

TABLE 26-B4

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP-113)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
Existing Conditions		N/A	N/A	310,000	250,000	185,000	145,000	125,000	85,000
Project Conditions	<u>Alternatives</u>								
a. Structural Elements:	ORME	SPF	50,000	153,000	64,000	45,000	45,000	45,000	45,000
New Roosevelt = NR	ORME			115,000	64,000	45,000	45,000	45,000	45,000
Cliff = NB	(w/SOD_2)								
Confluence = ORME	NRNB			150,000	80,000	50,000	45,000	45,000	45,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			105,000	82,000	53,000	45,000	45,000	45,000
Regulatory Storage	(w/SOD_2)								
at confluence = RS	NRNB			110,000	65,000	42,000	36,000	36,000	36,000
	w/RS								
b. Reregulation Elements:	R+H/C+B	SPF	90,000	150,000	110,000	80,000	80,000	80,000	80,000
Roosevelt = R	(w/$SOD_{1,2}$)								
Horseshoe = H									
Cliff = C	ORME	SPF	100,000	175,000	96,000	90,000	90,000	90,000	85,000
Bartlett = B	NRNB			170,000	110,000	90,000	90,000	90,000	85,000
	ORME	SPF	150,000	210,000	150,000	130,000	130,000	125,000	85,000
	NRNB			220,000	160,000	130,000	130,000	125,000	85,000
	R+H/C+B			205,000	190,000	130,000	130,000	125,000	85,000
	(w/$SOD_{1,2}$)								
	ORME	SPF	200,000	225,000	175,000	170,000	145,000	125,000	85,000
	NRNB			250,000	190,000	170,000	145,000	125,000	85,000

: N/A = not applicable

TABLE 26-C1

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs Frequency, yrs. 500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	360,000	295,000	250,000	200,000	135,000	95,000
2. Project Conditions	Alternatives								
a. Structural Elements:	SOD_1	N/A	N/A	290,000	270,000	250,000	210,000	150,000	100,000
New Roosevelt = NR	SOD_2	N/A	N/A	205,000	200,000	185,000	160,000	119,000	80,000
Cliff = NB									
Confluence = ORME	ORME	50	50,000	315,000	235,000	180,000	120,000	46,000	44,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			295,000	230,000	175,000	120,000	46,000	44,000
Regulatory Storage									
at confluence = RS	ORME	50	100,000	325,000	265,000	220,000	165,000	90,000	88,000
	NRNB			310,000	240,000	195,000	145,000	88,000	88,000
b. Reregulation Elements:									
Roosevelt = R	ORME	50	150,000	330,000	268,000	225,000	180,000	125,000	95,000
Horseshoe = H	NRNB			320,000	260,000	220,000	175,000	135,000	95,000
Cliff = C									
Bartlett = B		50	200,000[a]						

NOTE: N/A = not applicable

[a]Not required, existing conditions discharge = 175,000 cfs

TABLE 26-C2

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

	Alternatives	Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	360,000	295,000	250,000	200,000	135,000	95,000
2. Project Conditions									
a. Structural Elements:	ORME	100	50,000	260,000	190,000	140,000	97,000	46,000	44,000
New Roosevelt = NR	NRNB			240,000	175,000	135,000	92,000	46,000	44,000
Cliff = NB									
Confluence = ORME	ORME	100	100,000	265,000	215,000	180,000	140,000	88,000	88,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			290,000	225,000	180,000	140,000	88,000	88,000
Regulatory Storage									
at confluence = RS	ORME	100	150,000	280,000	235,000	205,000	165,000	125,000	95,000
	NRNB			300,000	245,000	210,000	170,000	135,000	95,000
b. Reregulation Elements:									
Roosevelt = R	ORME	100	200,000	320,000	260,000	215,000	175,000	135,000	95,000
Horseshoe = H	NRNB			320,000	260,000	215,000	175,000	135,000	95,000
Cliff = C									
Bartlett = B									

NOTE: N/A = not applicable

TABLE 26-C3

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	360,000	295,000	250,000	200,000	135,000	95,000
2. Project Conditions	Alternatives								
a. Structural Elements:	ORME	500	50,000	130,000	110,000	88,000	68,000	44,000	44,000
New Roosevelt = NR	NRNB			140,000	120,000	100,000	75,000	44,000	44,000
Cliff = NB									
Confluence = ORME	ORME	500	100,000	165,000	150,000	135,000	115,000	88,000	88,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			165,000	145,000	130,000	110,000	88,000	88,000
Regulatory Storage									
at confluence = RS	ORME	500	150,000	205,000	190,000	175,000	150,000	125,000	95,000
	NRNB			225,000	195,000	175,000	150,000	125,000	95,000
b. Reregulation Elements:									
Roosevelt = R	ORME	500	200,000	265,000	230,000	195,000	165,000	135,000	95,000
Horseshoe = H	NRNB			280,000	230,000	195,000	165,000	135,000	95,000
Cliff = C									
Bartlett = B									

NOTE: N/A = not applicable

TABLE 26-C4

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing Conditions		N/A	N/A	360,000	295,000	250,000	200,000	135,000	95,000
2. Project Conditions	Alternatives								
a. Structural Elements:	ORME	SPF	50,000	210,000	150,000	110,000	85,000	45,000	45,000
New Roosevelt = NR	ORME			160,000	130,000	108,000	80,000	45,000	45,000
Cliff = NB	(w/SOD_2)								
Confluence = ORME	NRNB			200,000	150,000	115,000	82,000	45,000	45,000
Safety-of-Dams= SOD_1,SOD_2	NRNB			160,000	135,000	115,000	90,000	54,000	45,000
Regulatory Storage	w/SOD_2								
at confluence = RS	NRNB			170,000	130,000	105,000	80,000	45,000	36,000
	(w/RS)								
b. Reregulation Elements:	R+H/C+B	SPF	90,000	210,000	175,000	150,000	120,000	80,000	80,000
Roosevelt = R	(w/$SOD_{1,2}$)								
Horseshoe = H									
Cliff = C	ORME	SPF	100,000	220,000	180,000	150,000	120,000	88,000	88,000
Bartlett = B	NRNB			220,000	180,000	150,000	120,000	88,000	88,000
	ORME	SPF	150,000	250,000	215,000	185,000	155,000	125,000	95,000
	NRNB			270,000	225,000	190,000	155,000	125,000	95,000
	R+H/C+B			270,000	230,000	200,000	167,000	135,000	95,000
	(w/$SOD_{1,2}$)								
	ORME	SPF	200,000	290,000	240,000	205,000	165,000	135,000	95,000
	NRNB			305,000	245,000	205,000	165,000	135,000	95,000

NOTE: N/A = not applicable

TABLE 27

SEASONAL DISCHARGE FREQUENCY
Summary of Non-Damaging Flows, cfs

Season		Duration (1)				
		Peak	1-Day	2-Day	3-Day	6-Day
May through June	Q	60,000	*	*	*	*
	Pr.	.0028				
	Q	80,000				
	Pr.	.002				
July through September	Q	60,000	*	*	*	*
	Pr.	.008				
	Q	69,000				
	Pr.	.005				
	Q	90,000				
	Pr.	.002				
October through November	Q	60,000	*	*	*	*
	Pr.	.0054				
	Q	63,000				
	Pr.	.005				
	Q	150,000				
	Pr.	.002				
December through April	Q	60,000	60,000	60,000	60,000	*
	Pr.	.155	.105	.063	.033	
	Q	91,000	62,000	67,000	74,000	
	Pr.	.10	.10	.05	.02	
	Q	140,000	93,000	98,000	94,000	
	Pr.	.05	.05	.02	.01	
	Q	200,000	130,000	120,000	110,000	
	Pr.	.02	.02	.01	.005	
	Q	250,000	160,000	135,000	130,000	
	Pr.	.01	.01	.005	.002	
	Q	295,000	185,000	160,000		
	Pr.	.005	.005	.002		
	Q	360,000	220,000			
	Pr.	.002	.002			

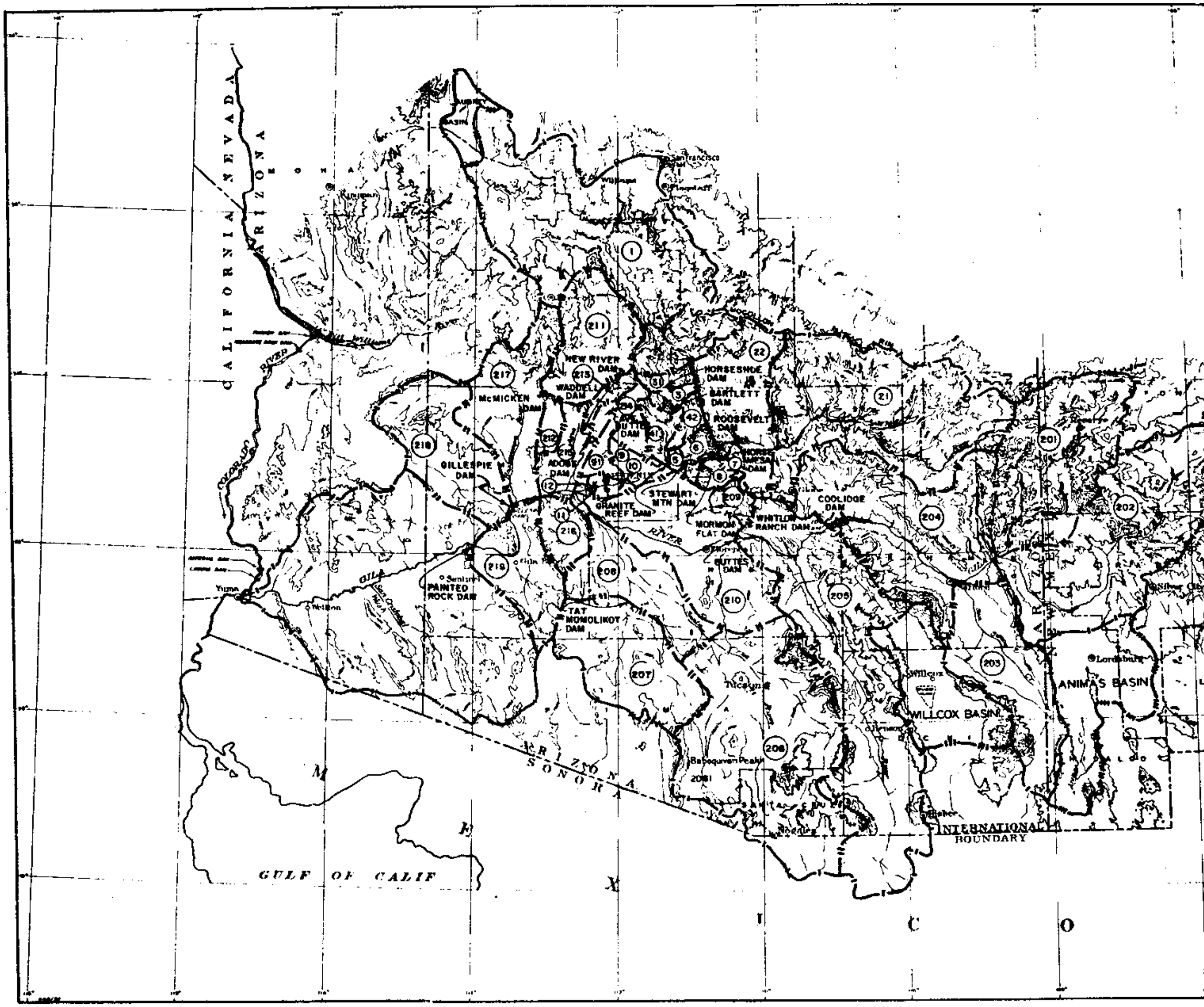
(1) Duration discharges represent threshold discharge i.e. discharges which are equalled or exceeded for given durations.

* All duration discharges 60,000 cfs for 500-yr return period (Pr=.002).
Return period = (1/Pr.)

TABLE 28
SALT RIVER AND VERDE RIVER
CONFIDENCE LIMITS

Pr[a] exceedance probability	Confidence limits (cfs) .05 limit	.95 limit	Salt River Peak Inflow to Roosevelt Dam (cfs)
.002	489,000.	218,000.	320,000
.005	391,000.	182,000.	260,000
.010	321,000.	155,000.	220,000
.020	255,000.	128,000.	175,000
.040	195,000.	103,000.	135,000
.100	124,000.	70,200.	90,000
.200	77,800.	47,100.	60,000
.500	29,400.	19,100.	23,500
.800	10,200.	6,200.	8,000
.900	5,640.	3,120.	4,300
.950	3,390.	1,690.	2,500
.990	1,220.	479.	800
			Verde River Peak Inflow to Horseshoe Dam (cfs)
.002	235,000.	107,000.	160,000
.005	213,000.	98,200.	145,000
.010	192,000.	90,200.	130,000
.020	168,000.	80,700.	115,000
.040	141,000.	69,600.	97,000
.100	99,700.	51,900.	71,000
.200	66,400.	36,500.	48,500
.500	23,900.	14,300.	18,000
.800	6,400.	3,590.	4,900
.900	2,860.	1,410.	2,100
.950	1,380.	583.	920
.990	293.	85.	170

a Return period = $\frac{1}{Pr}$

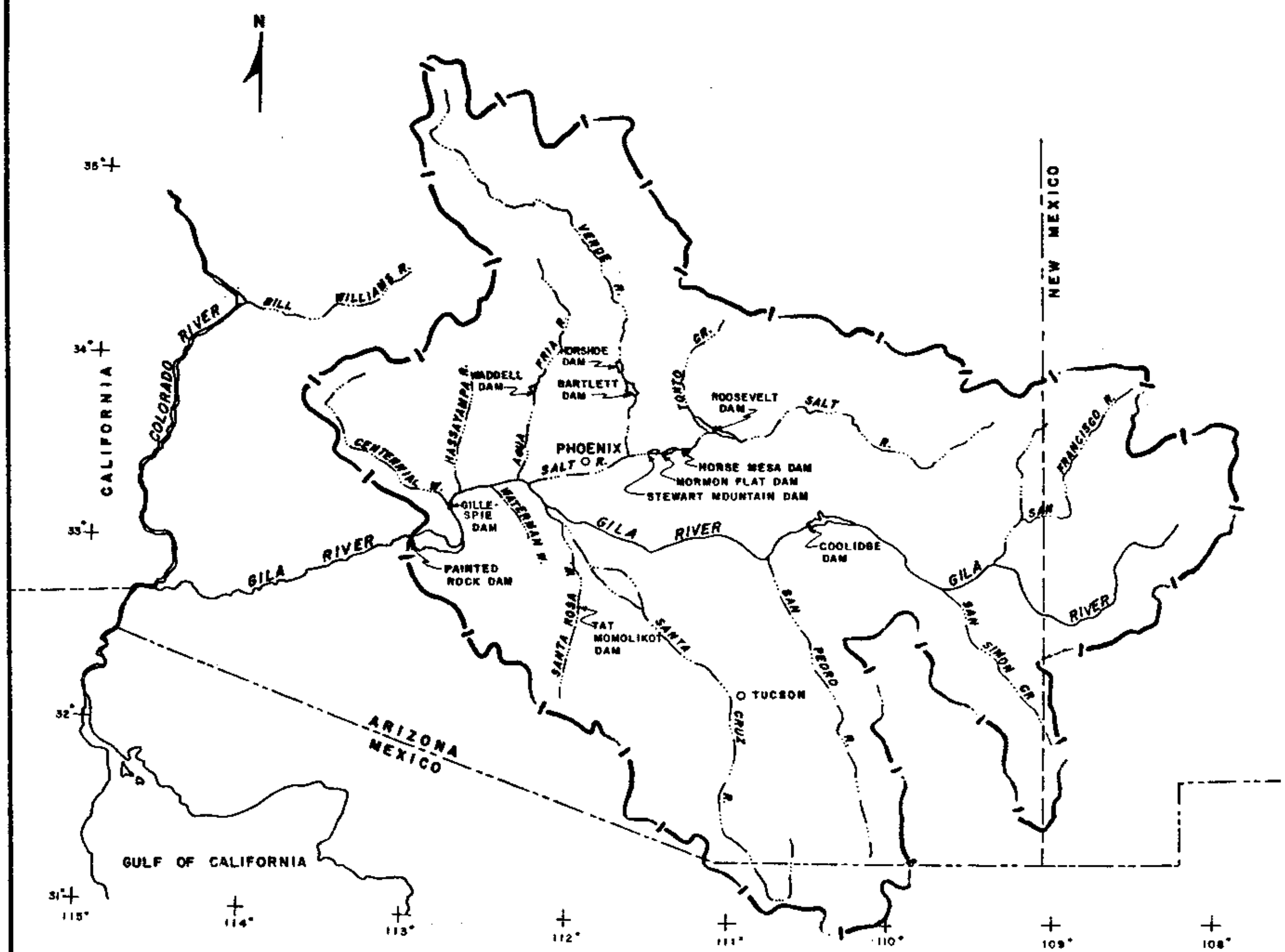


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DRAINAGE AREA MAP
GILA RIVER

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

PLATE 1

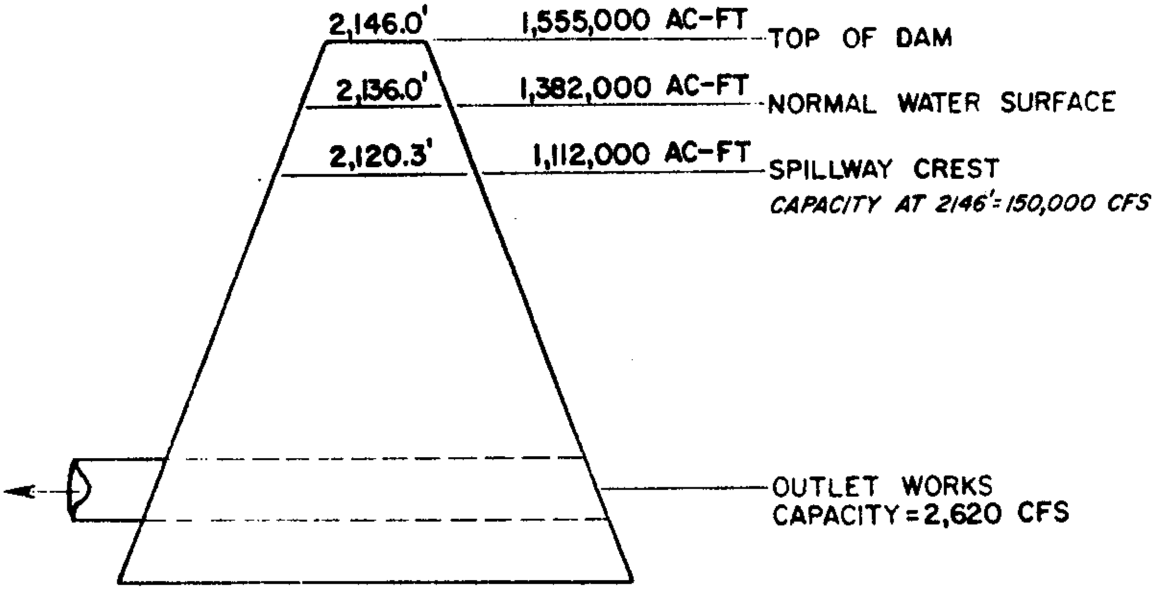


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

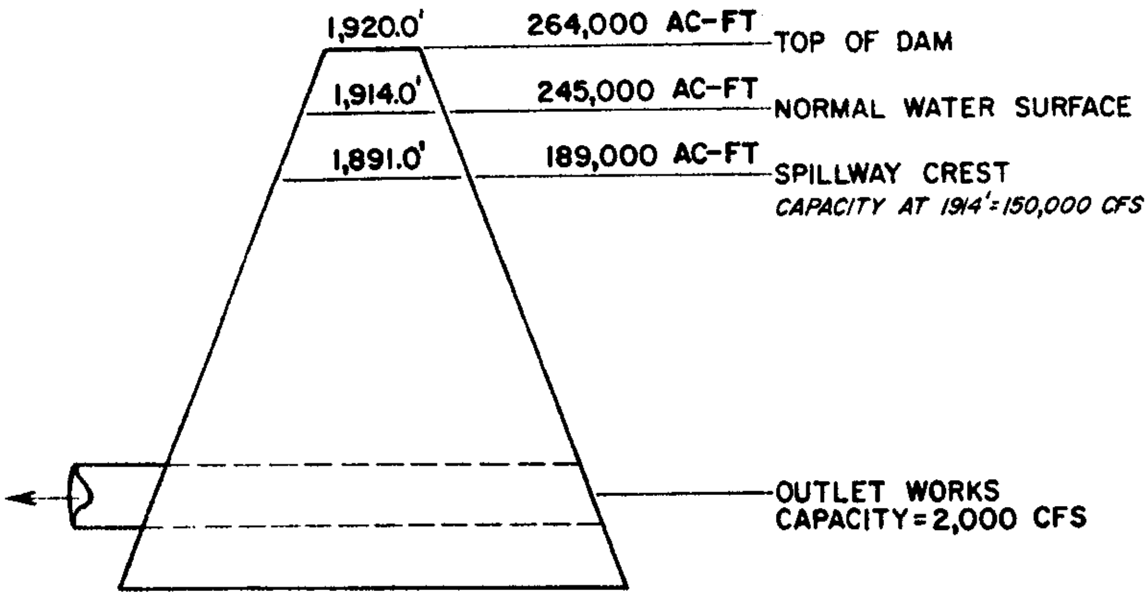
GILA RIVER AND TRIBUTARIES
WITH MAJOR STRUCTURES

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

PLATE 2



ROOSEVELT DAM



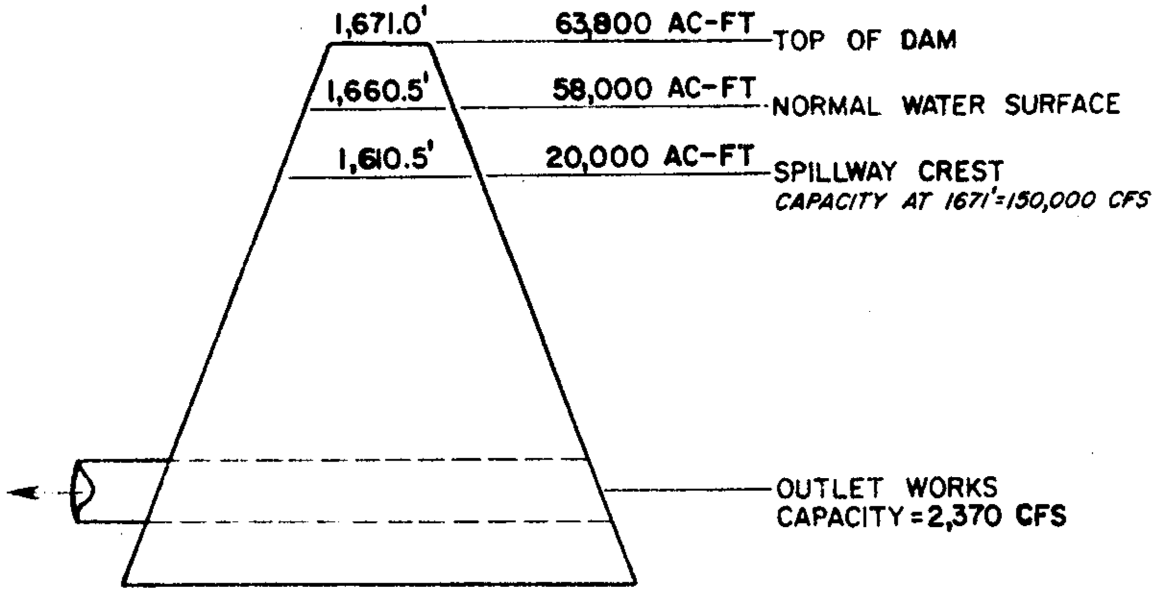
HORSE MESA DAM

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

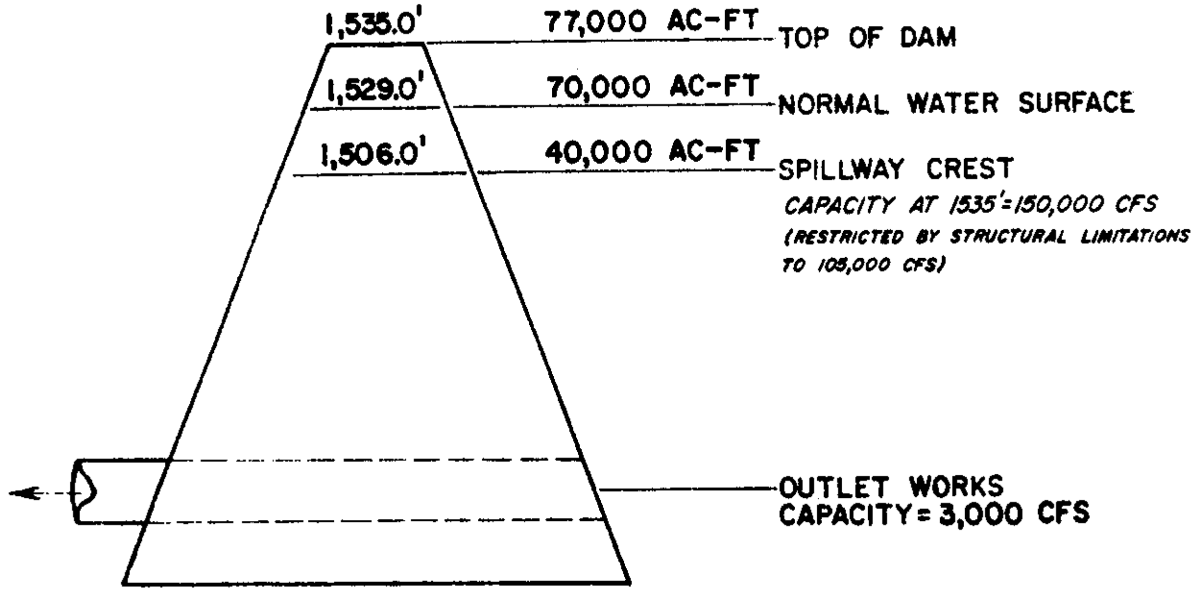
EMBANKMENT PROFILES

ROOSEVELT AND HORSE MESA

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



MORMON FLAT DAM



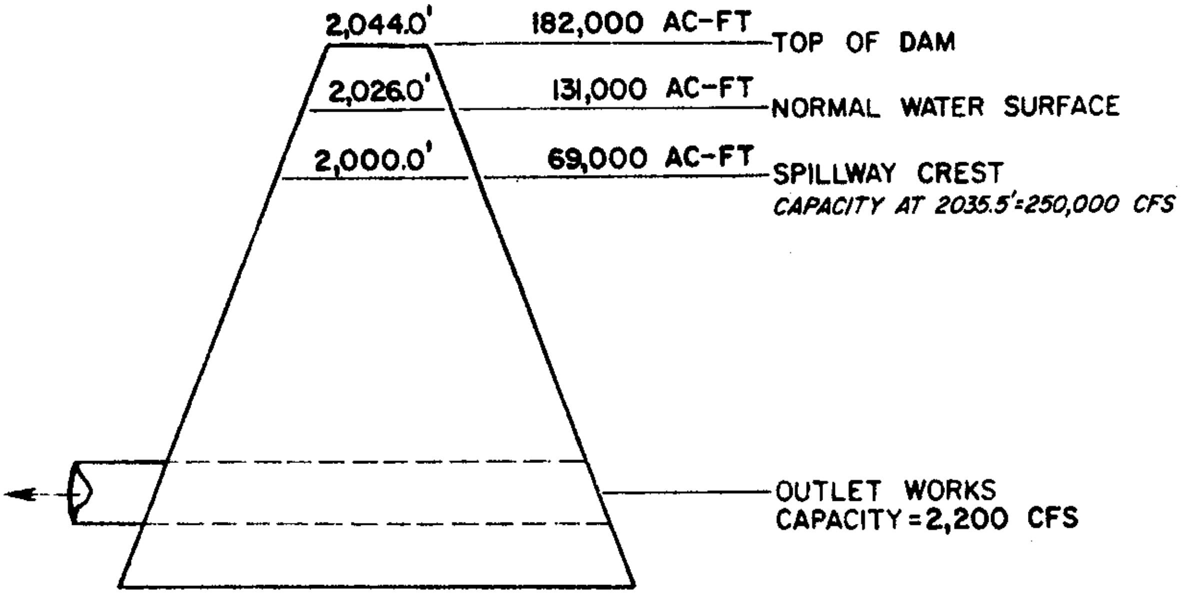
STEWART MOUNTAIN DAM

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

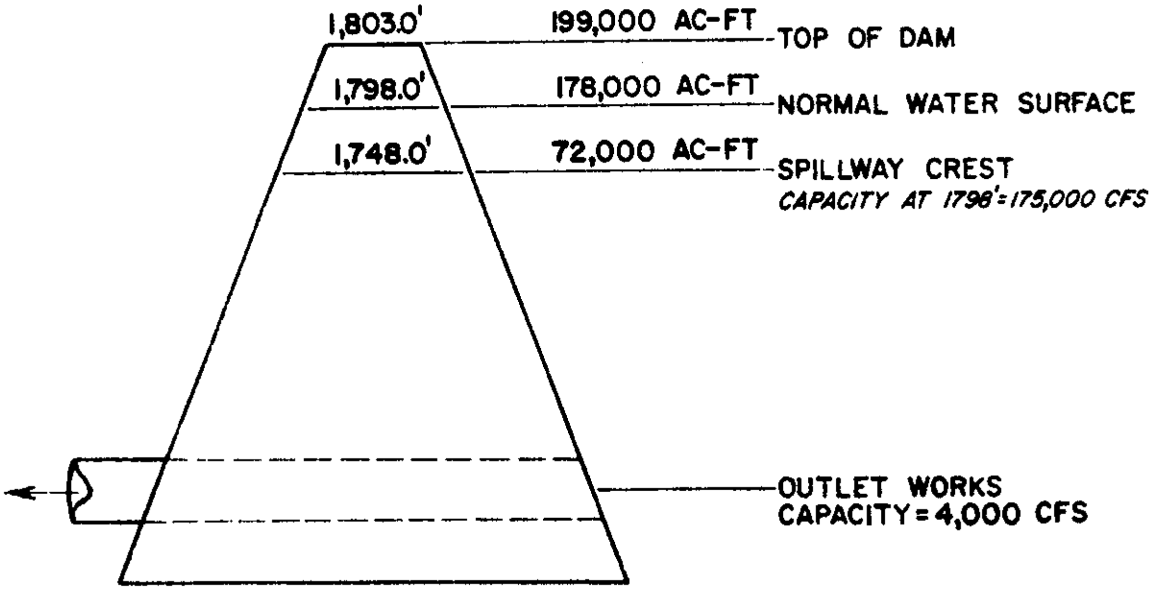
EMBANKMENT PROFILES

MORMON FLAT AND STEWART MTN.

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



HORSESHOE DAM



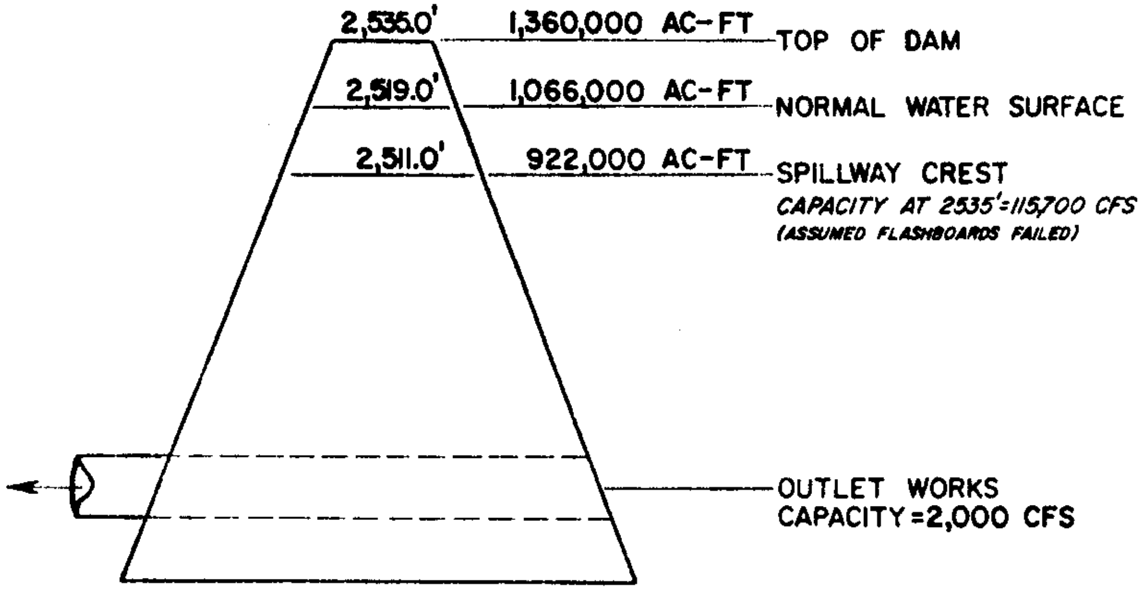
BARTLETT DAM

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

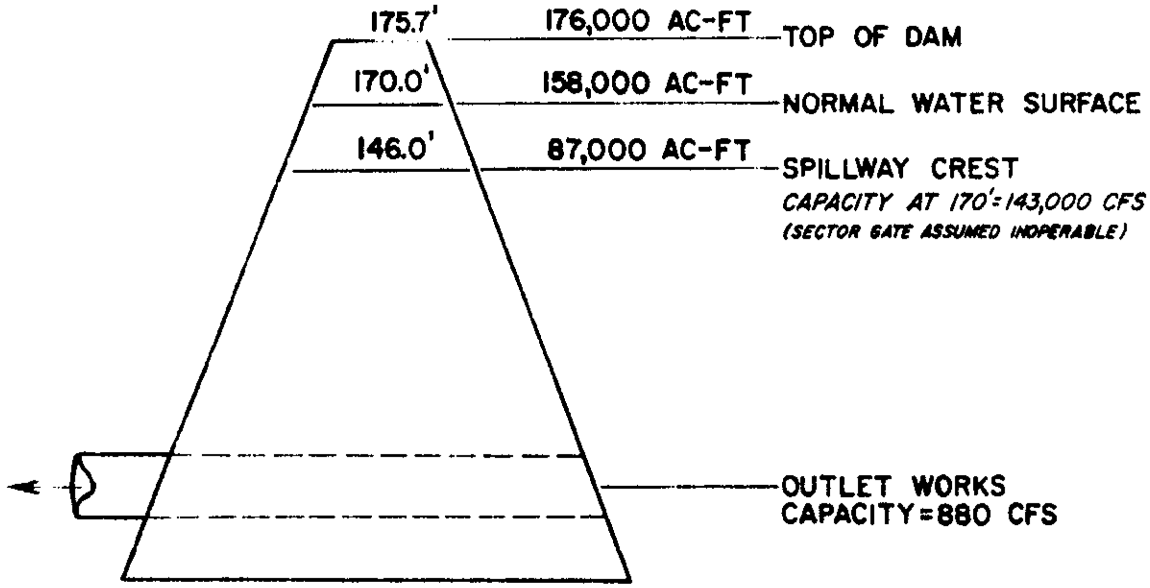
EMBANKMENT PROFILES

HORSESHOE AND BARTLETT

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



COOLIDGE DAM



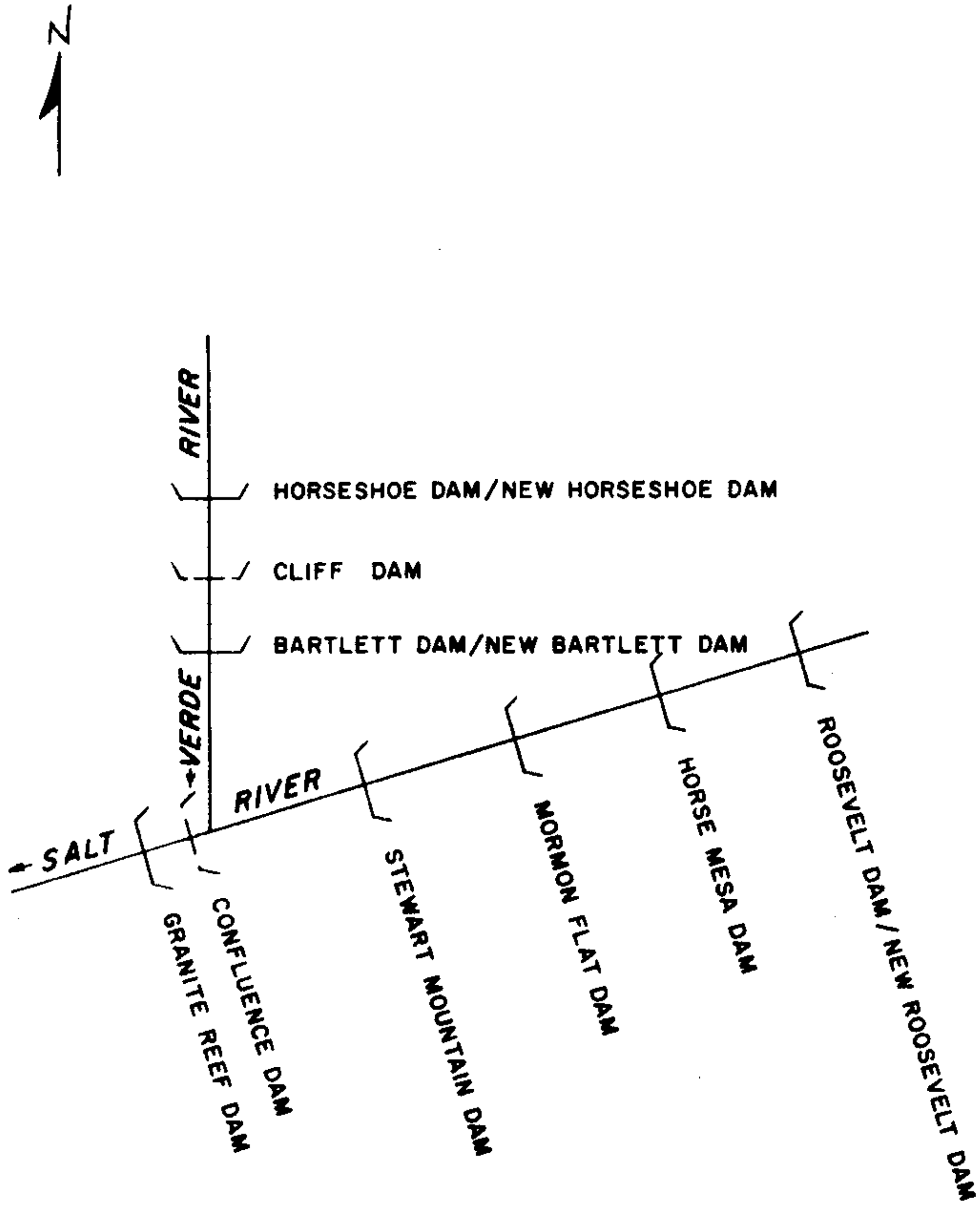
WADDELL DAM

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

EMBANKMENT PROFILES

COOLIDGE AND WADDELL

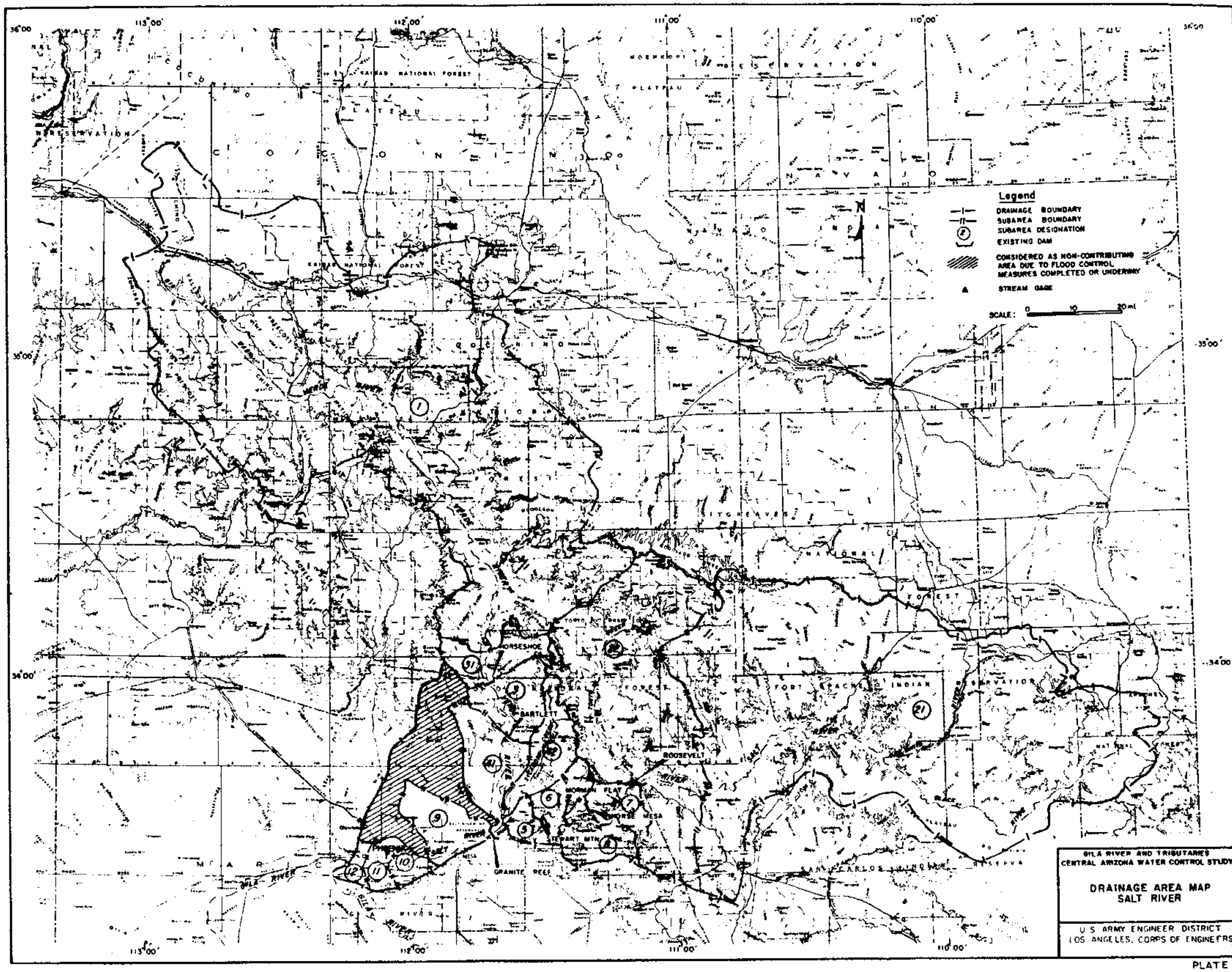
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS



GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

LOCATIONS OF PROJECT SITES

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



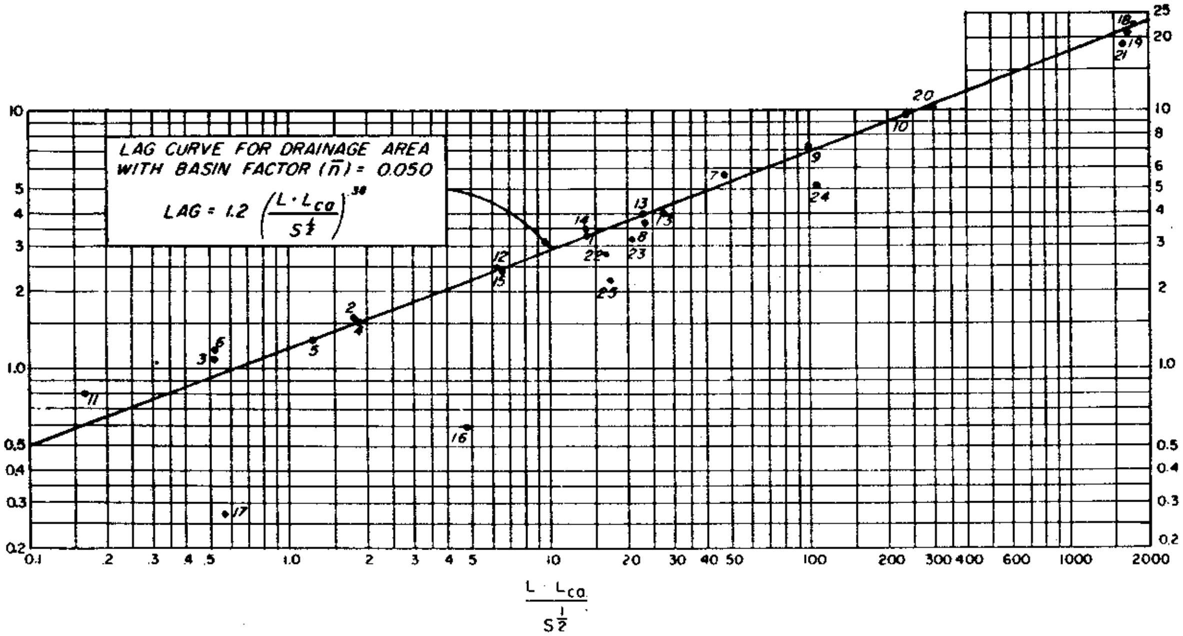
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DRAINAGE AREA MAP
SALT RIVER

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

PLATE 8

		CONTRIBUTING AREA	L	L_{ca}	S	LAG	ESTIMATED $\bar{n}$
		SQ. MI.	MILES	MILES	FT/MI.	HOURS	
1.	SAN GABRIEL RIVER AT SAN GABRIEL DAM, CALIF.	162.0	23.2	11.6	350	3.3	0.050
2.	WEST FORT SAN GABRIEL RIVER AT COGSWELL DAM, CALIF.	40.4	9.3	4.3	450	1.6	.050
3.	SAN ANITA CREEK AT SANTA ANITA DAM, CALIF.	10.8	5.8	2.5	690	1.1	.050
4.	SAN DIMAS CREEK AT SAN DIMAS DAM, CALIF.	16.2	8.6	4.8	440	1 5	.050
5.	EATON WASH AT EATON WASH DAM, CALIF.	9.5	7.3	4.4	600	1.3	.050
6.	SAN ANTONIO CREEK NEAR CLAREMONT, CALIF.	16.9	5.9	3.0	1,017	1 2	.055
7.	SANTA CLARA RIVER NEAR SAUGUS, CALIF.	355.0	36.0	15.8	140	5 6	.050
8.	TEMECULA CREEK AT PAUBA CANYON, CALIF.	168.0	26.0	11.3	150	3.7	.050
9.	SANTA MARGARITA RIVER NEAR FALLBROOK, CALIF.	645.0	46.0	22.0	105	7 3	.055
10.	SANTA MARGARITA RIVER AT YSIDORA, CALIF.	740.0	61.2	34.3	85	9.5	.055
11.	LIVE OAK CREEK AT LIVE OAK DAM, CALIF.	2.3	2.9	1.5	700	8	.070
12.	TUJUNGA CREEK AT BIG TUJUNGA DAM, CALIF.	81.4	15.1	7.3	290	2.5	.050
13.	MURRIETA CREEK AT TEMECULA, CALIF.	220.0	27.2	10.3	95	4.0	.050
14.	LOS ANGELES RIVER AT SEPULVEDA DAM, CALIF.	152.0	19.0	9.0	145	3.5	.050
15.	PACOIMA WASH AT PACOIMA DAM, CALIF.	27.8	15.0	8.0	315	2.4	.050
16.	ALHAMBRA WASH ABOVE SHORT STREET, CALIF.	14.0	9.5	4 6	85	.6	.015
17.	BROADWAY DRAIN ABOVE RAYMOND DIKE, CALIF.	2.5	3.4	1.7	100	.28	.015
18.	GILA RIVER AT CONNOR NO. 4 DAM SITE, ARIZ.	2840.0	131.0	71 0	29	21.5	.050
19.	SAN FRANCISCO RIVER AT JUNCTION WITH BLUE RIVER, ARIZ.	2000.0	130.0	74 0	32	20.6	.050
20.	BLUE RIVER NEAR CLIFTON, ARIZ.	790.0	77.0	37.0	65	10.3	.050
21.	SALT RIVER NEAR ROOSEVELT, ARIZ.	4310.0	160.0	66.0	45	18.6	.050
22.	NEW RIVER AT ROCK SPRINGS, ARIZ.	67.3	20.2	9.7	141	2.8	.044
23.	NEW RIVER AT NEW RIVER, ARIZ.	85.7	26.2	8 9	122	3.2	.042
24.	NEW RIVER AT BELL ROAD, ARIZ.	187.0	47.6	20.7	83	5.1	.035
25.	SKUNK CREEK NEAR PHOENIX, ARIZ.	64.6	17.6	9.9	102	2.3	.031



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

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

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

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

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

TERMINOLOGY

L = LENGTH OF LONGEST WATERCOURSE.

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

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

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

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

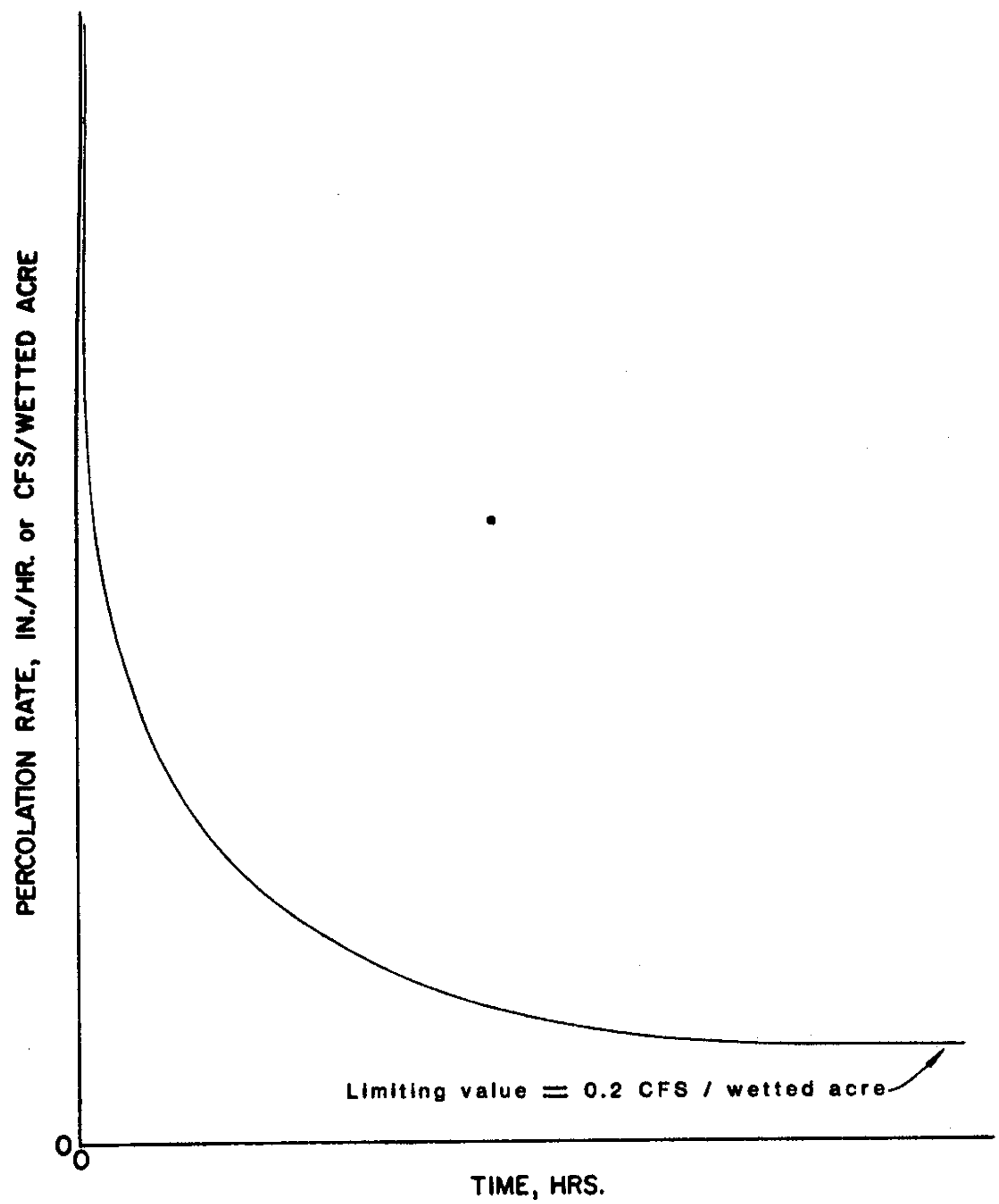
NOTE:
TO OBTAIN THE LAG (IN HOURS) FOR ANY AREA, MULTIPLY THE LAG OBTAINED FROM THE CURVE BY:

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

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

LAG RELATIONSHIPS

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

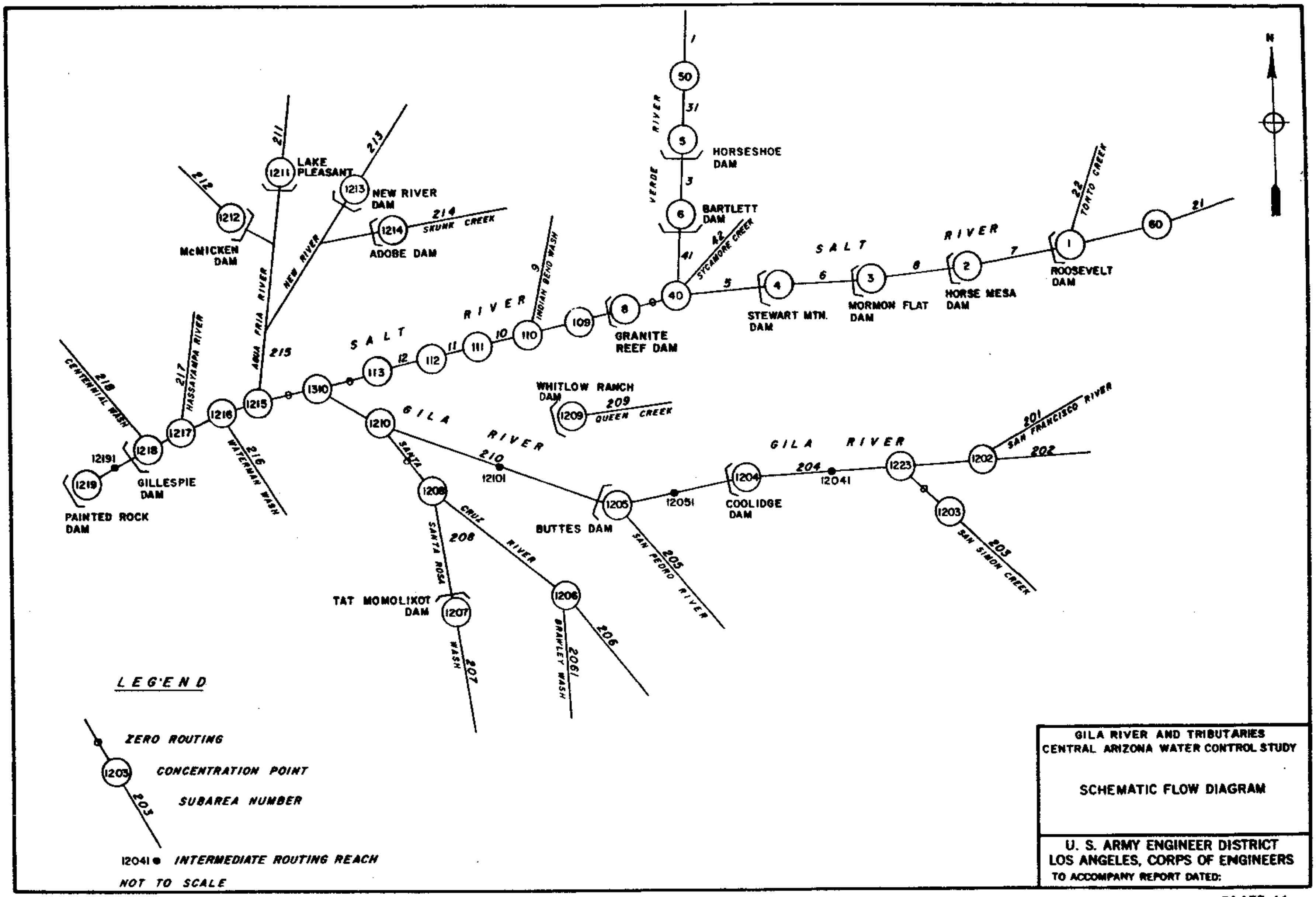


Note: 1in/hr ≈ 1cfs/wetted acre

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

HYPOTHETICAL PERCOLATION LOSS RATE CURVE

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

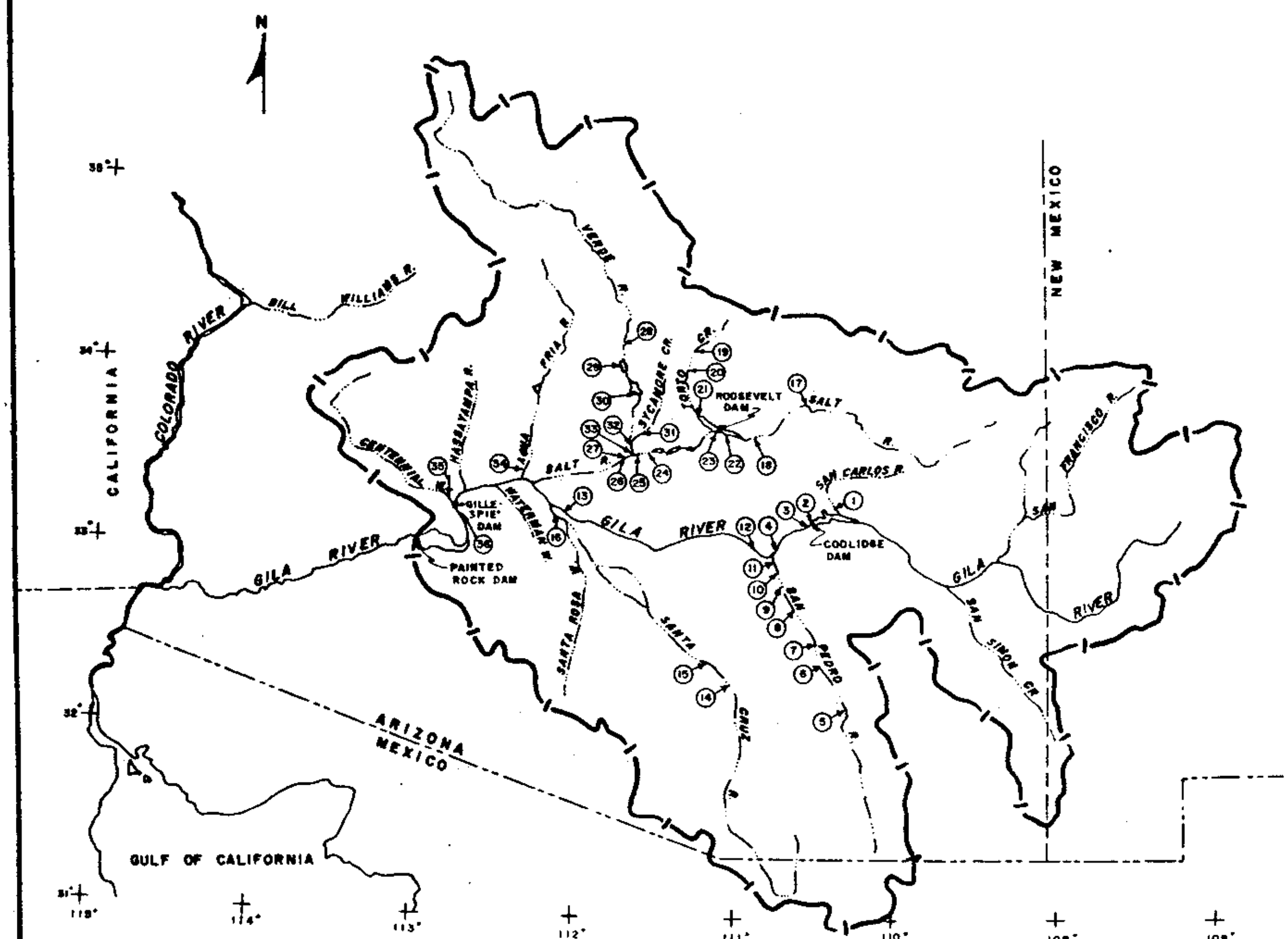


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

SCHEMATIC FLOW DIAGRAM

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

PLATE 11



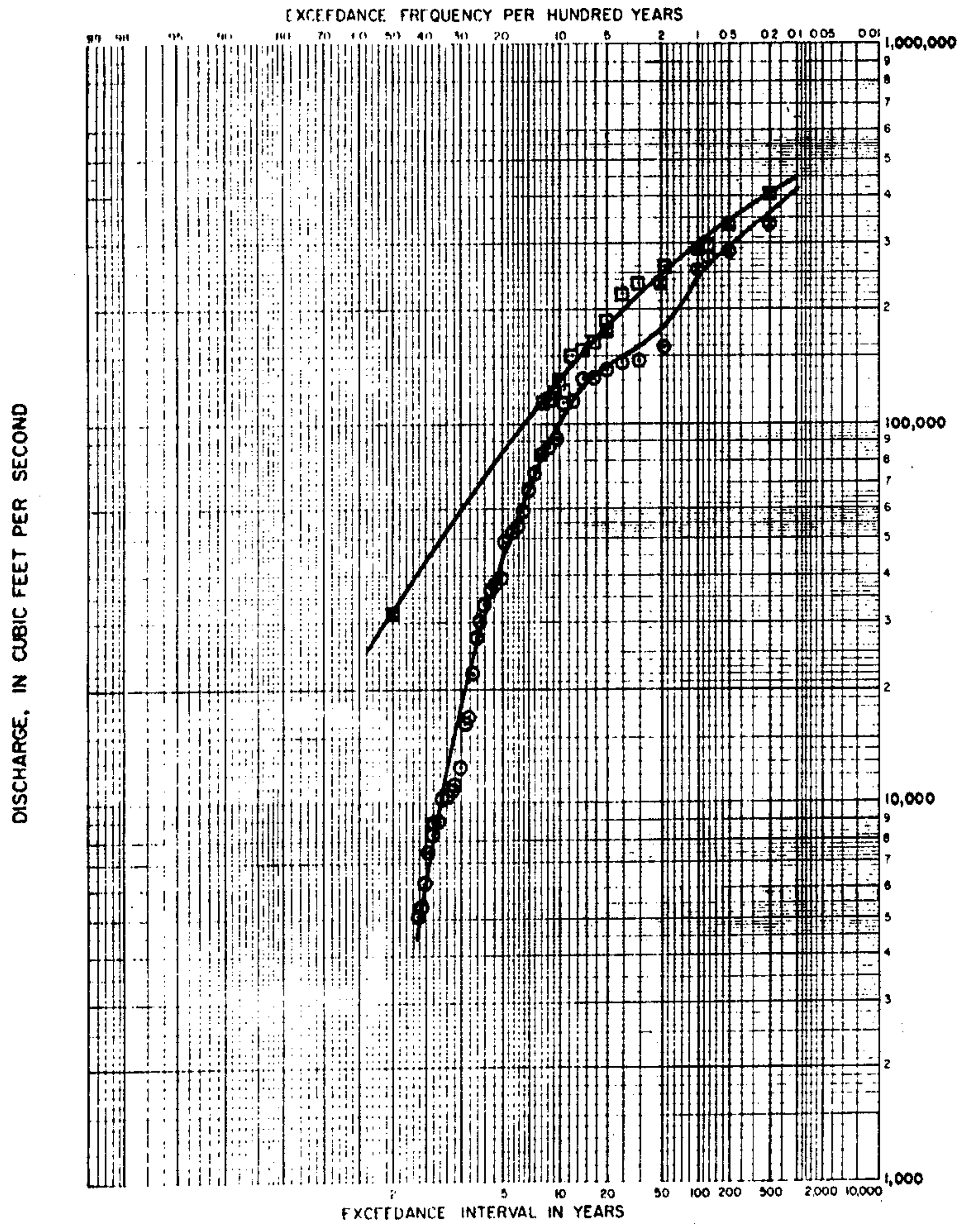
REF. NO.	LOCATION
1	SAN CARLOS RIVER NR. PERIDOT
2	SAN CARLOS RESERVOIR INFLOW
3	GILA RIVER BELOW COOLIDGE DAM
4	GILA RIVER AT WINKELMAN
5	SAN PEDRO RIVER AT PALOMINAS
6	SAN PEDRO RIVER AT CHARLESTON
7	SAN PEDRO RIVER AT TOMBSTONE
8	SAN PEDRO RIVER NR. BENSON
9	SAN PEDRO RIVER NR. REDINGTON
10	SAN PEDRO RIVER NR. MAMMOTH
11	SAN PEDRO RIVER NR. WINKELMAN
12	GILA RIVER AT KELVIN
13	GILA RIVER NR. LAVEEN
14	SANTA CRUZ RIVER AT TUCSON
15	SANTA CRUZ RIVER AT CORTARO
16	SANTA CRUZ RIVER NR. LAVEEN
17	SALT RIVER NR. CHRYSOTILE
18	SALT RIVER NR. ROOSEVELT
19	TONTO CREEK NR. GISELA
20	TONTO CREEK ABOVE GUN CREEK
21	TONTO CREEK NR. ROOSEVELT
22	SALT RIVER AT ROOSEVELT
23	SALT RIVER AT AND BELOW ROOSEVELT
24	SALT RIVER BELOW STEWART MTN.
25	SALT RIVER AT McDOWELL
26	SALT RIVER AT GRANITE REEF DAM
27	SALT RIVER AT ARIZONA DAM
28	VERDE RIVER BELOW TANGLE CREEK
29	VERDE RIVER AT BARTLETT
30	VERDE RIVER BELOW BARTLETT
31	SYCAMORE CREEK NR. FT. McDOWELL
32	VERDE RIVER NR. SCOTTSDALE
33	VERDE RIVER NR. McDOWELL
34	AGUA FRIA RIVER AT AVONDALE
35	CENTENNIAL WASH NR. ARLINGTON
36	GILA RIVER BELOW GILLESPIE

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

STREAMGAGE LOCATIONS

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

PLATE 12



LEGEND

A. MEDIAN PLOTTING POSITIONS FOR N=91 YRS USING PERIOD-OF-RECORD FLOWS INPUT TO HEC-5 SIMULATION MODEL.

- ⊙ REGULATED DISCHARGES FOR PRESENT CONDITIONS.
- ⊡ NATURAL DISCHARGES.

B. BALANCED HYDROGRAPH RESULTS USING HEC-5 SIMULATION MODEL.

- ⊗ REGULATED DISCHARGES FOR PRESENT CONDITIONS.
- ⊠ NATURAL DISCHARGES.

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

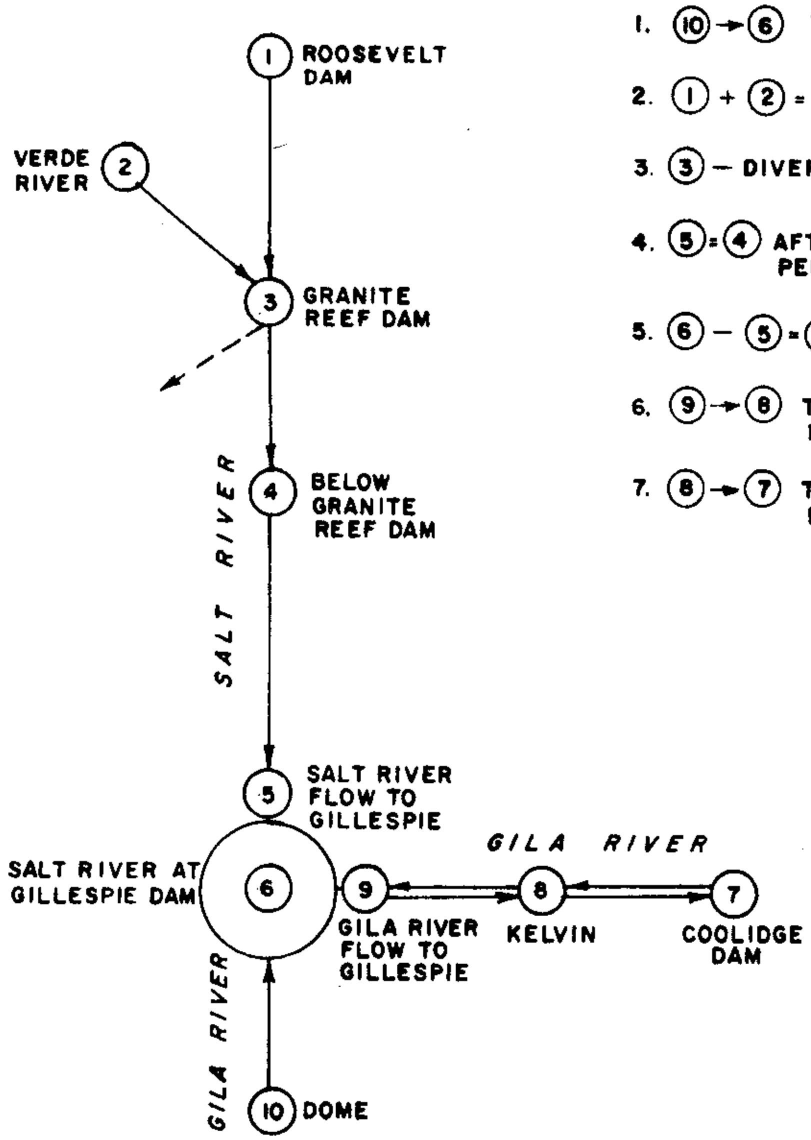
DISCHARGE – FREQUENCY
(EXISTING CONDITIONS)
SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER

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

DEFINITION: 6 = 5 + 9

CORRELATION STEP SEQUENCE

1. 10 → 6 TO FILL GAPS IN STREAM-FLOW RECORD AT "6"
2. 1 + 2 = 3
3. 3 − DIVERSION = 4
4. 5 = 4 AFTER ROUTING AND PERCOLATION LOSS
5. 6 − 5 = 9
6. 9 → 8 TO FILL GAPS IN STREAM-FLOW RECORD AT "8"
7. 8 → 7 TO FILL GAPS IN STREAM-FLOW RECORD AT "7"



LEGEND

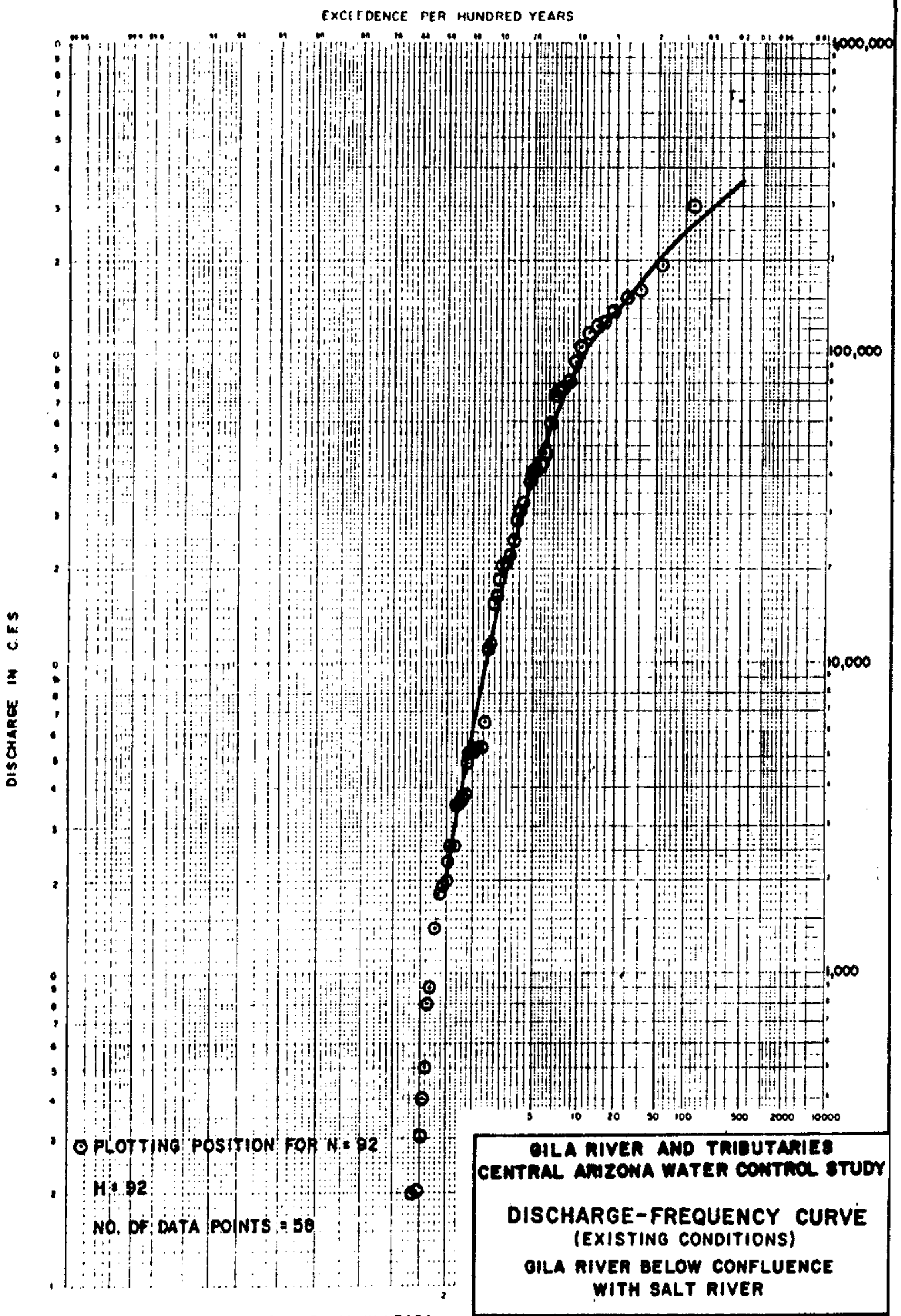
——— *INDICATES DIRECTION OF CORRELATION AND SUBSEQUENT ROUTING*

– – – *DIVERSION*

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

CORRELATION SEQUENCE FOR INFLOW TO COOLIDGE DAM
1903-1938

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

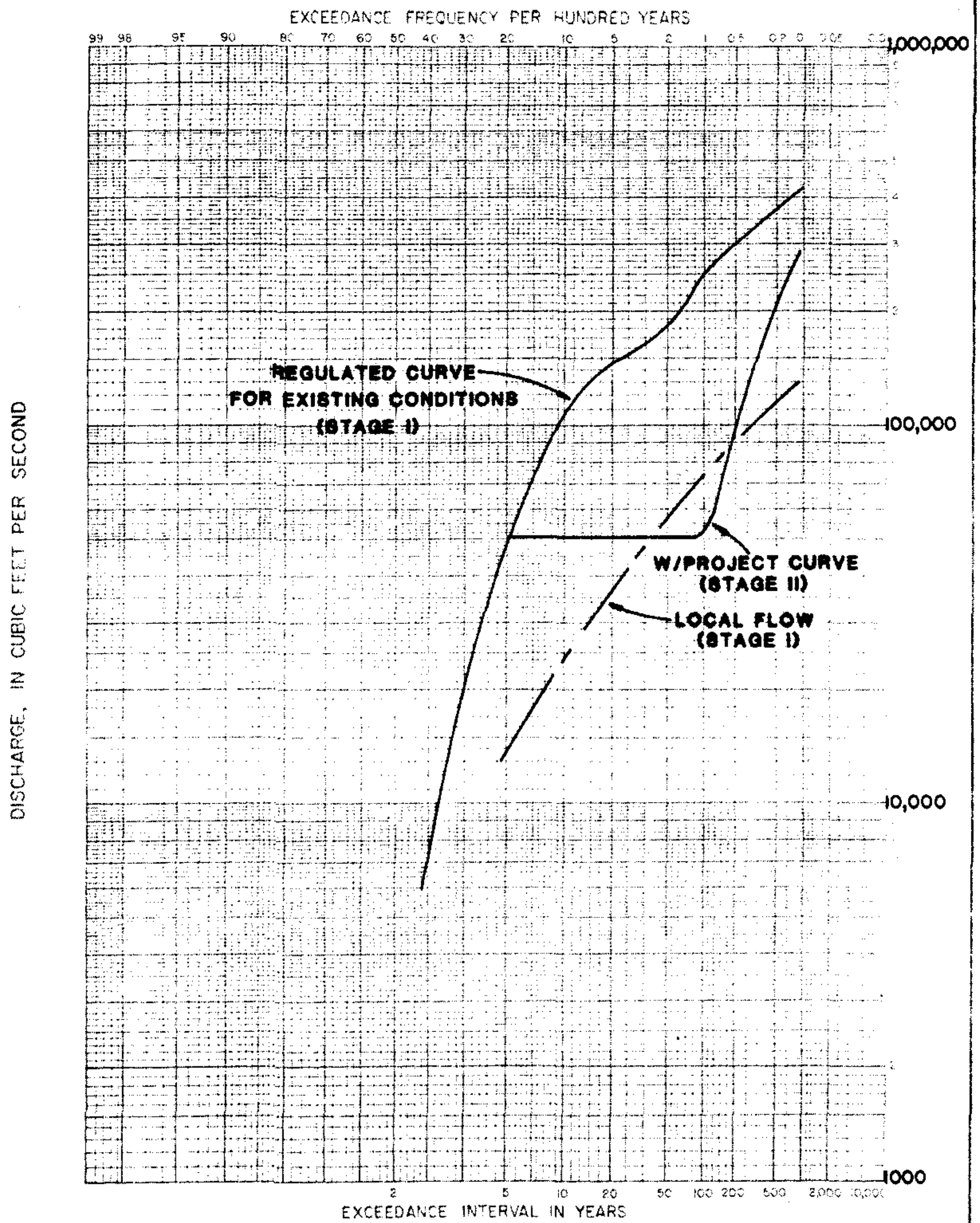


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVE
(EXISTING CONDITIONS)
GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER

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

PLATE 15



ALTERNATIVE = NRNB

DESIGN = SPF

TARGET = 50,000 CFS

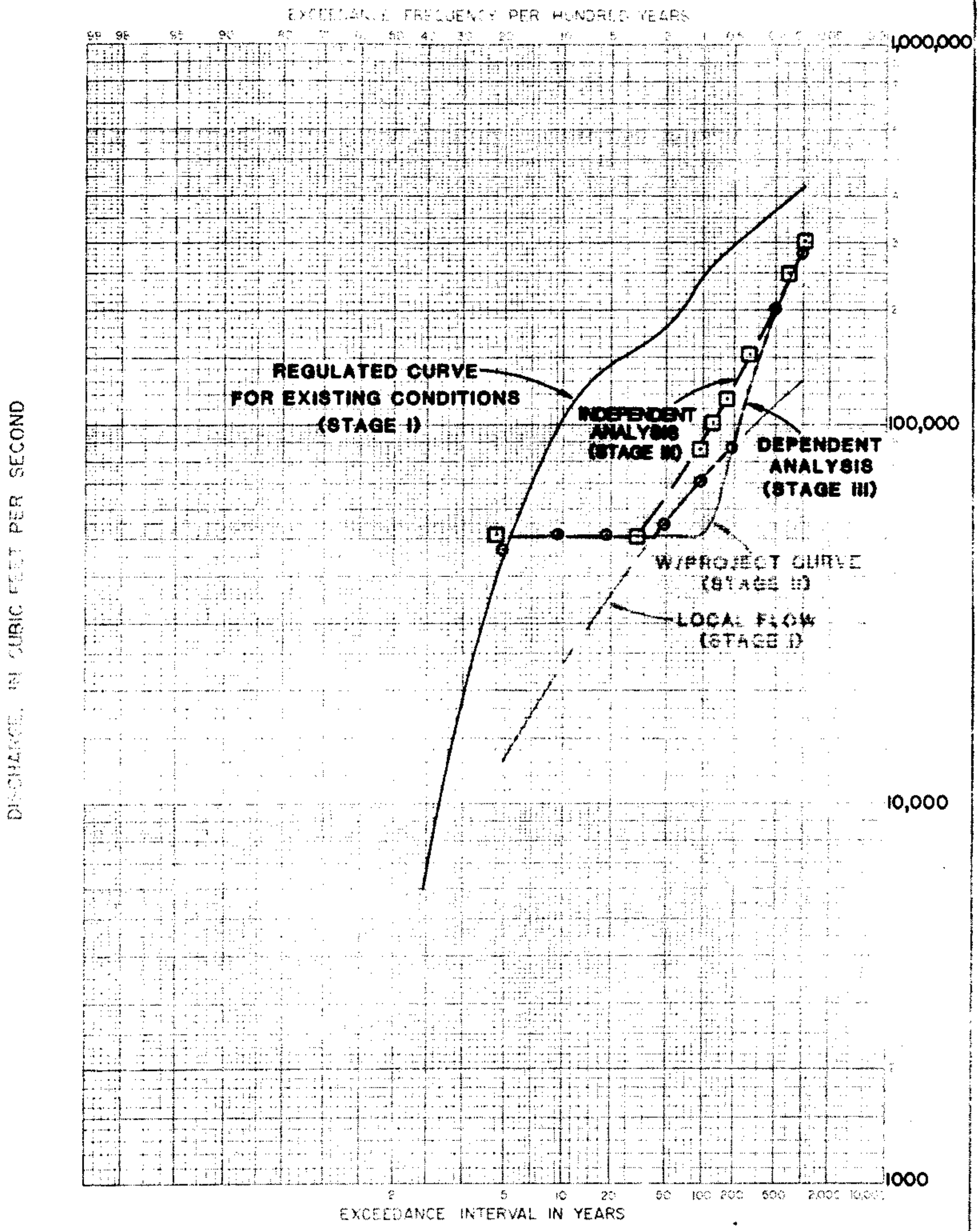
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DEVELOPMENT OF PROJECT CONDITIONS

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



ALTERNATIVE = NRNB

DESIGN = SPF

TARGET = 50,000 CFS

⊙ - DEPENDENT ANALYSIS

⊡ - INDEPENDENT ANALYSIS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

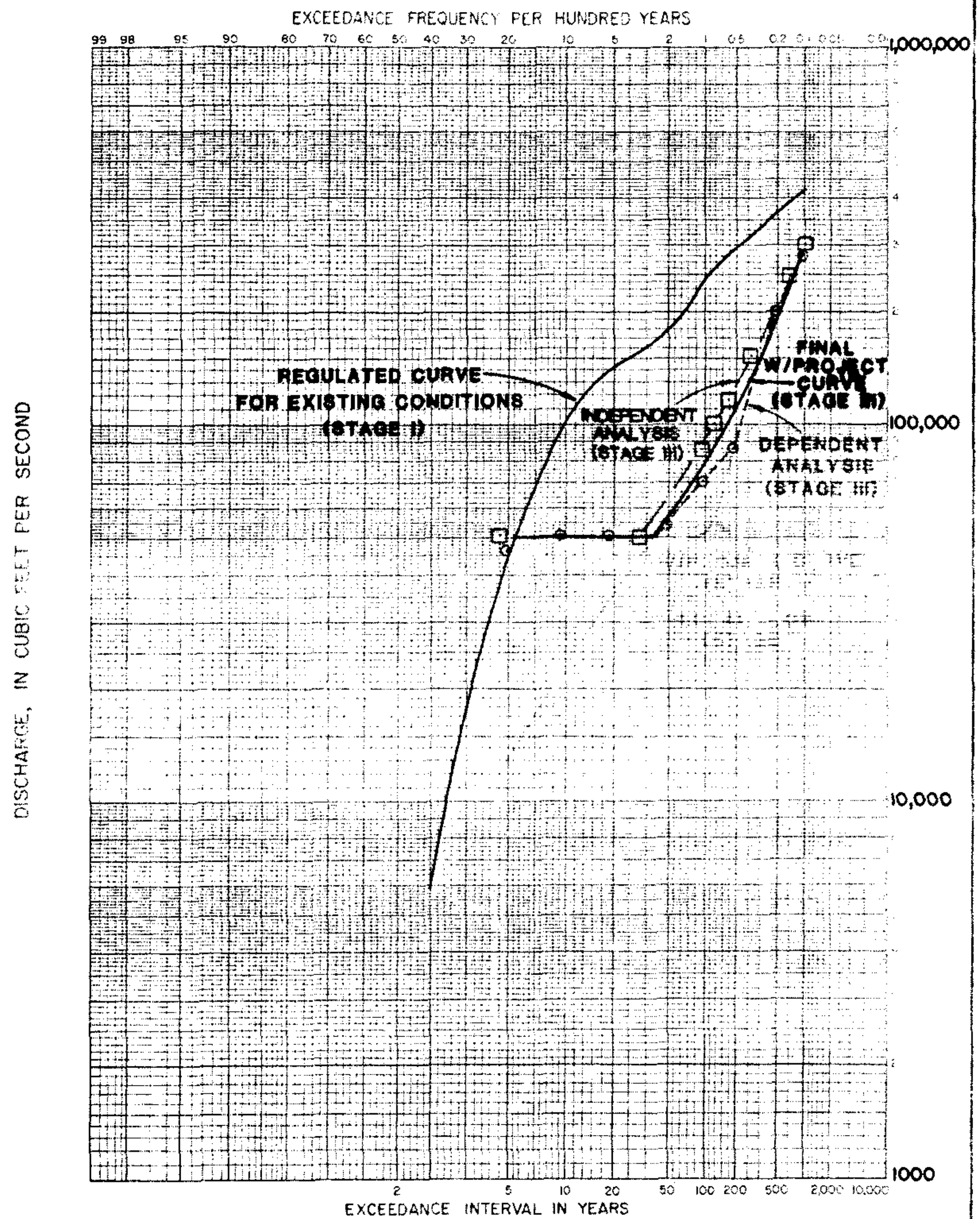
DEVELOPMENT OF PROJECT CONDITIONS

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 16b



ALTERNATIVE = NRNB

DESIGN = SPF

TARGET = 50,000 CFS

⊙ - DEPENDENT ANALYSIS

□ - INDEPENDENT ANALYSIS

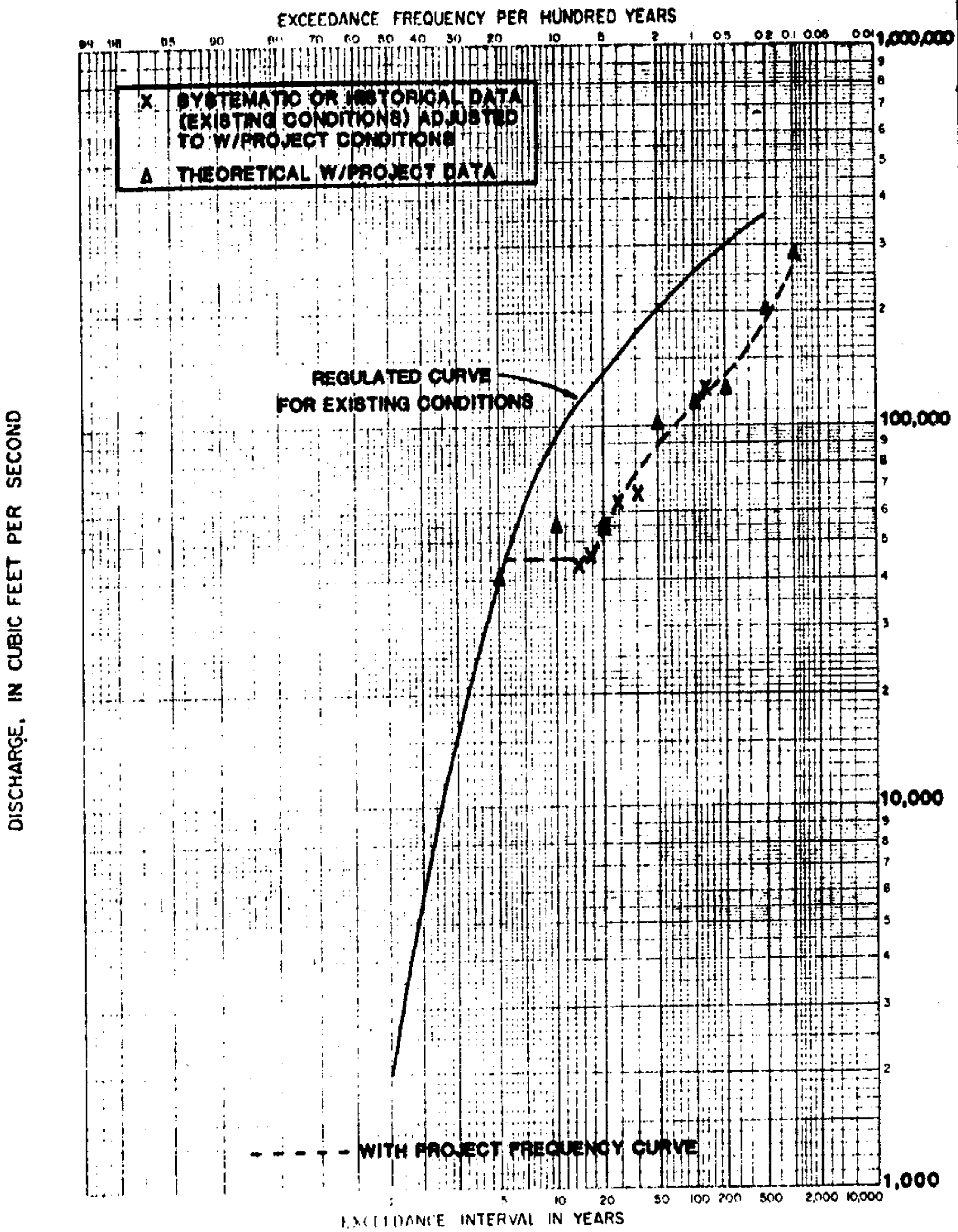
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DEVELOPMENT OF PROJECT CONDITIONS

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



ALTERNATIVE = NRNB

DESIGN = SPF

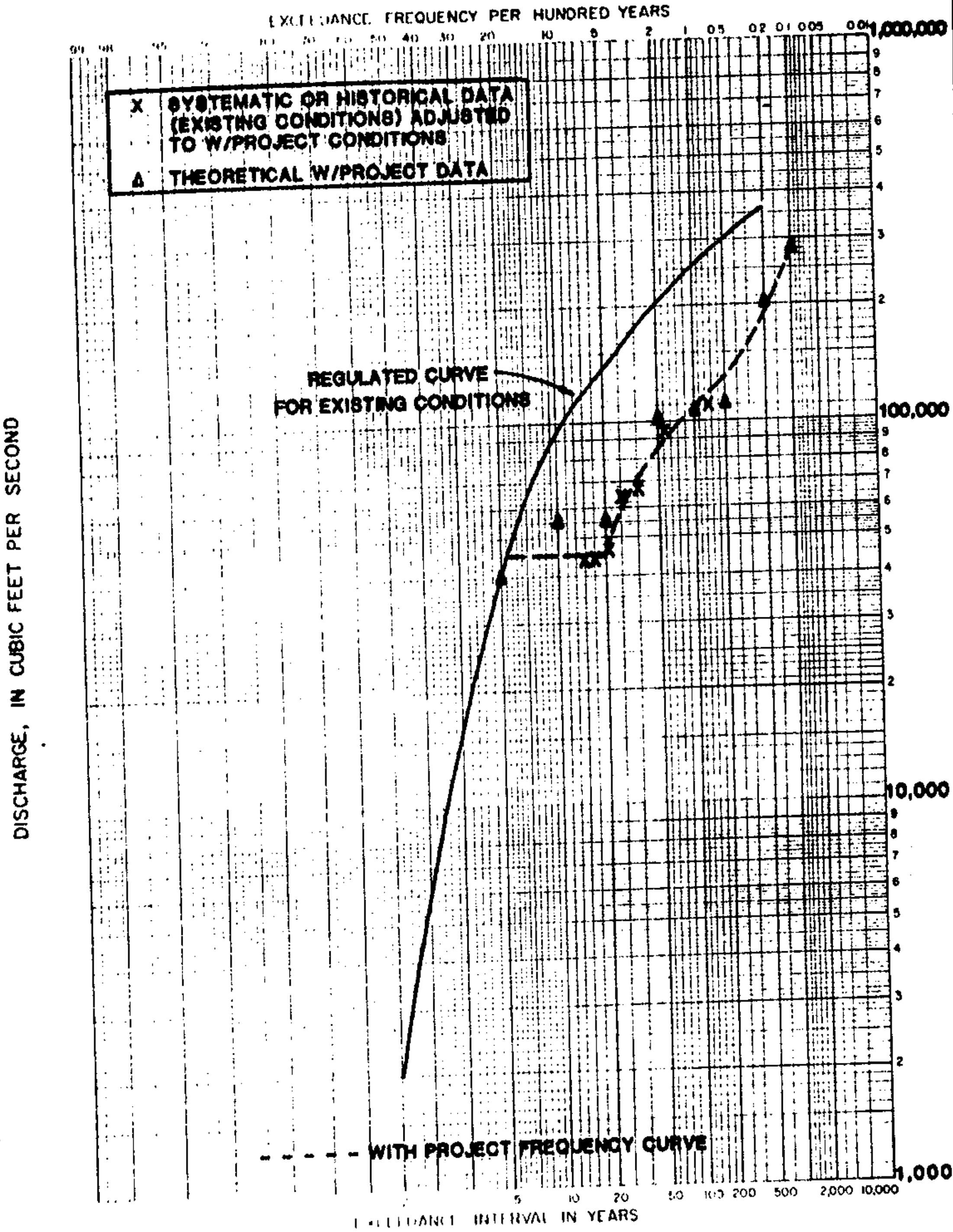
TARGET = 50,000 CFS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



ALTERNATIVE = ORME

DESIGN = SPF

TARGET = 50,000 CFS

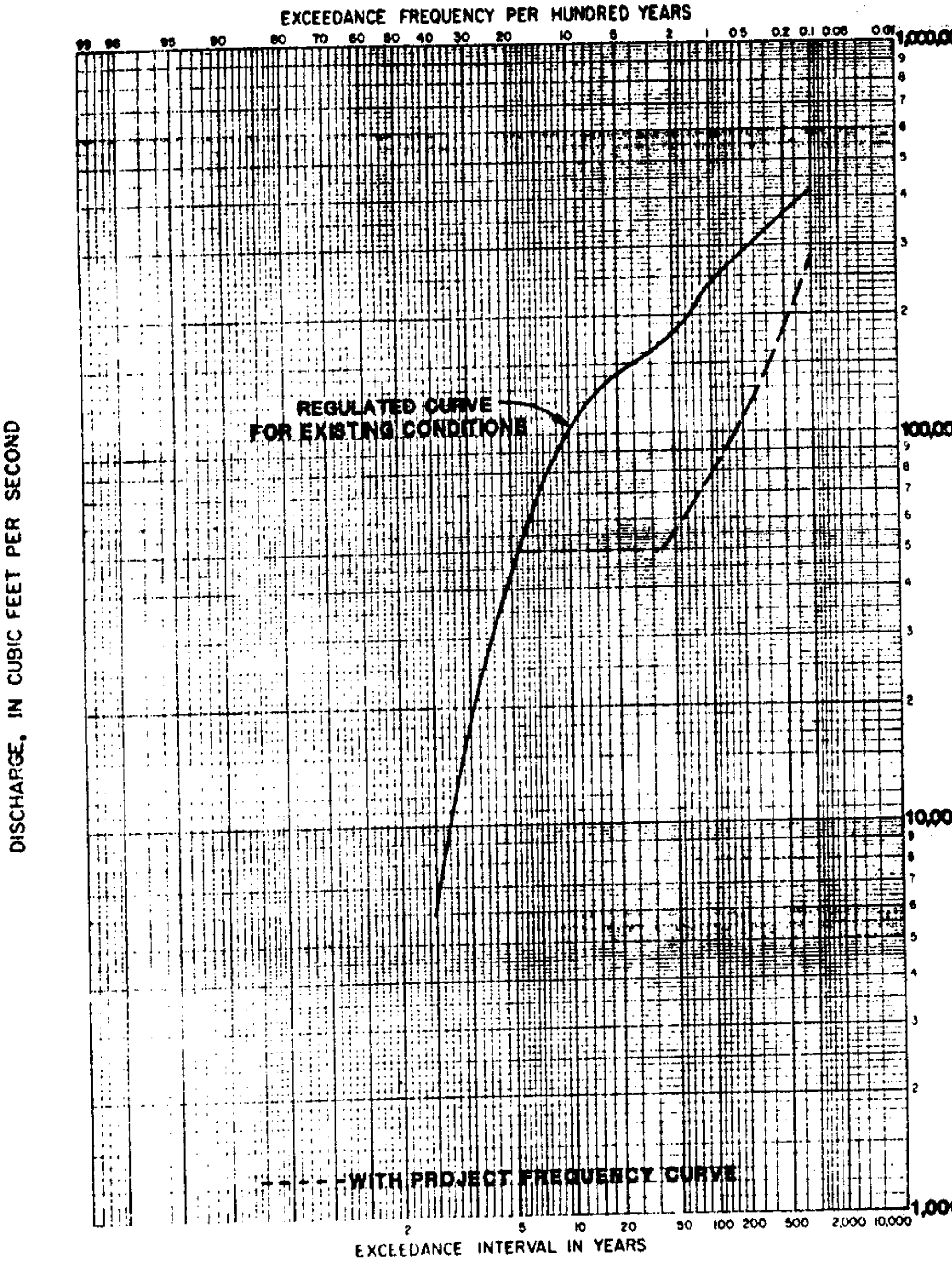
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER (CP-1310)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 17b



ALTERNATIVE = NRNB

DESIGN = SPF

TARGET = 50,000 CFS

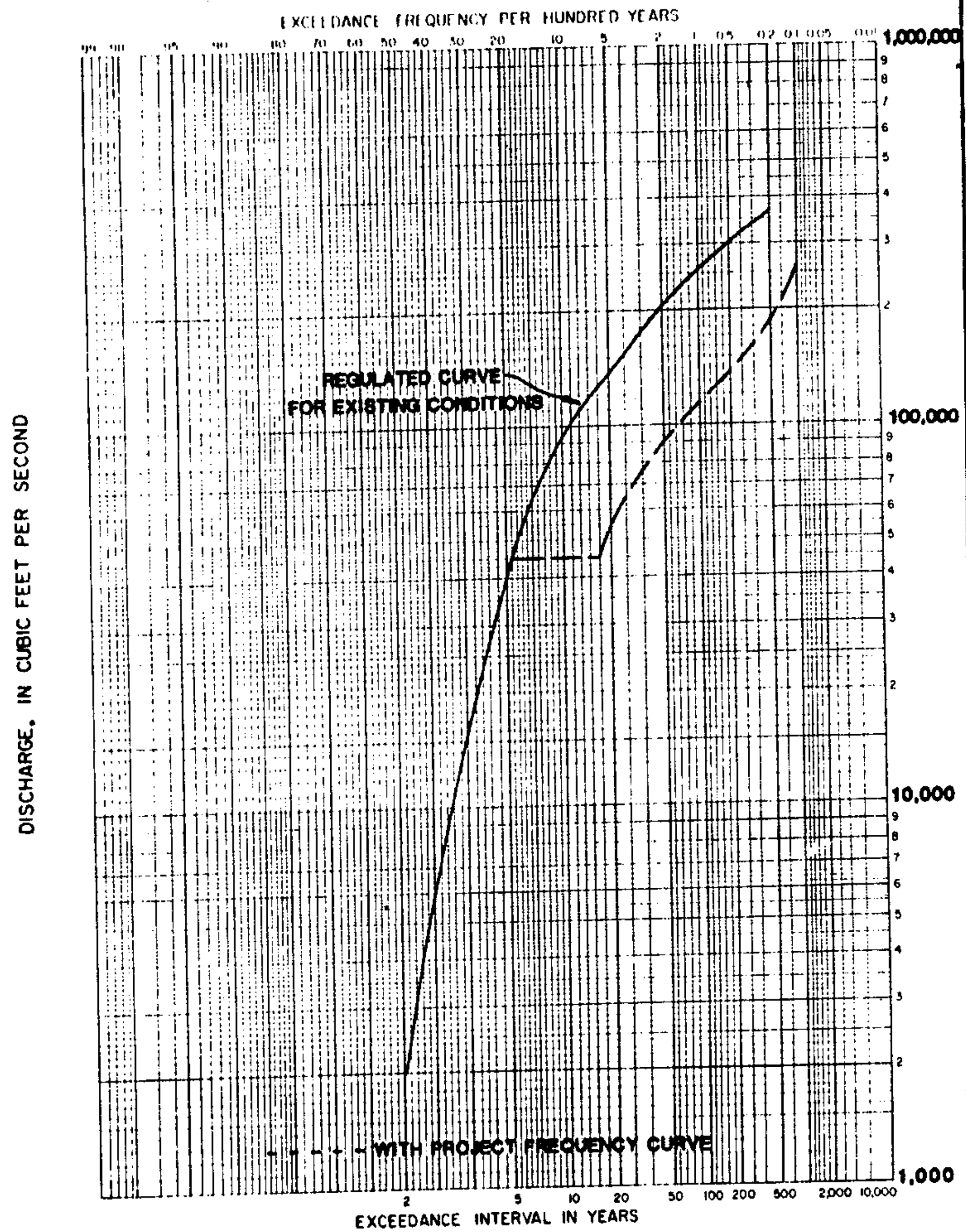
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH
VERDE RIVER (CP 40)

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

PLATE 18a



ALTERNATIVE = NRNB

DESIGN = SPF

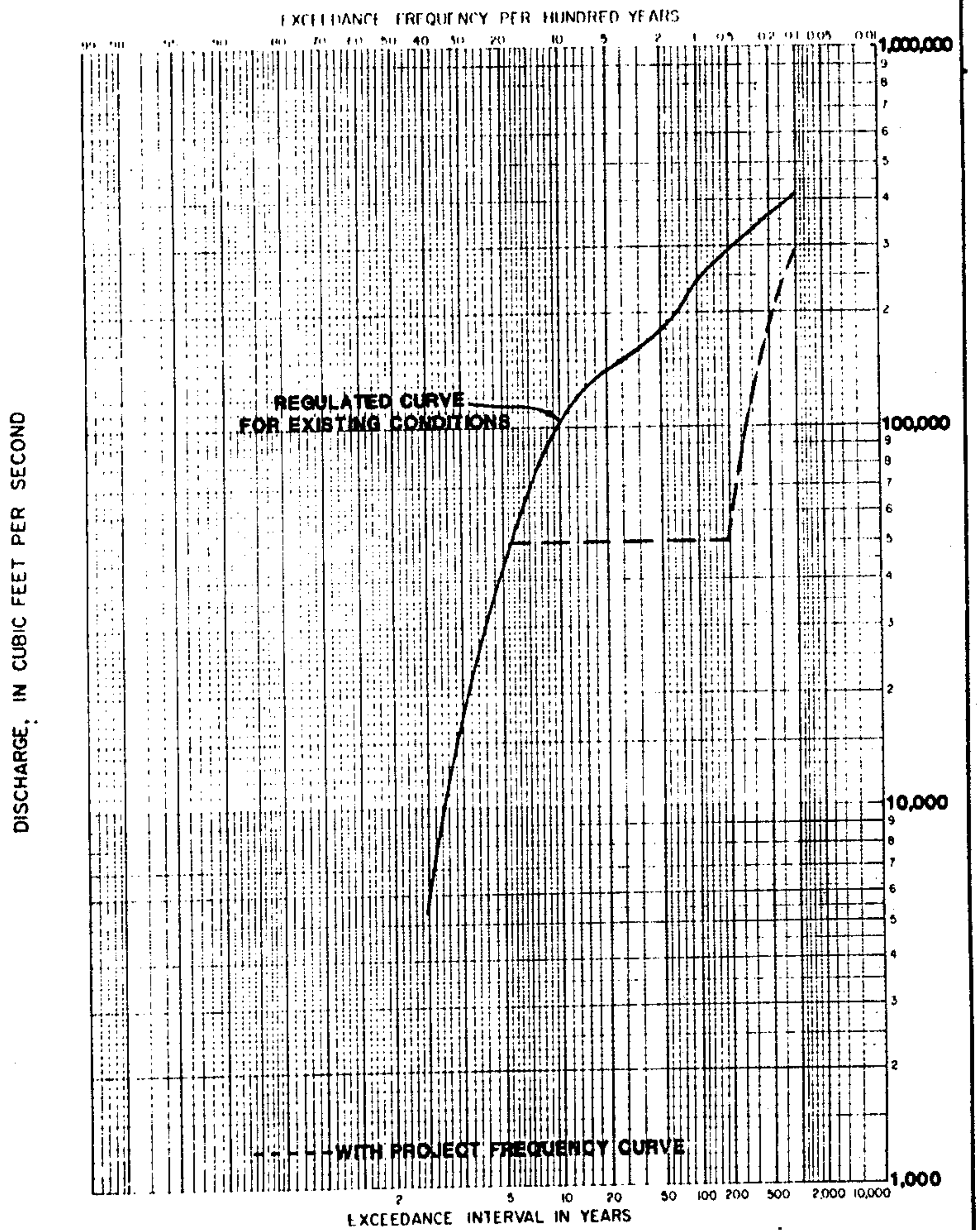
TARGET = 50,000 CFS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER (CP-1310)

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



ALTERNATIVE=ORME

DESIGN=SPF

TARGET=50,000 CFS

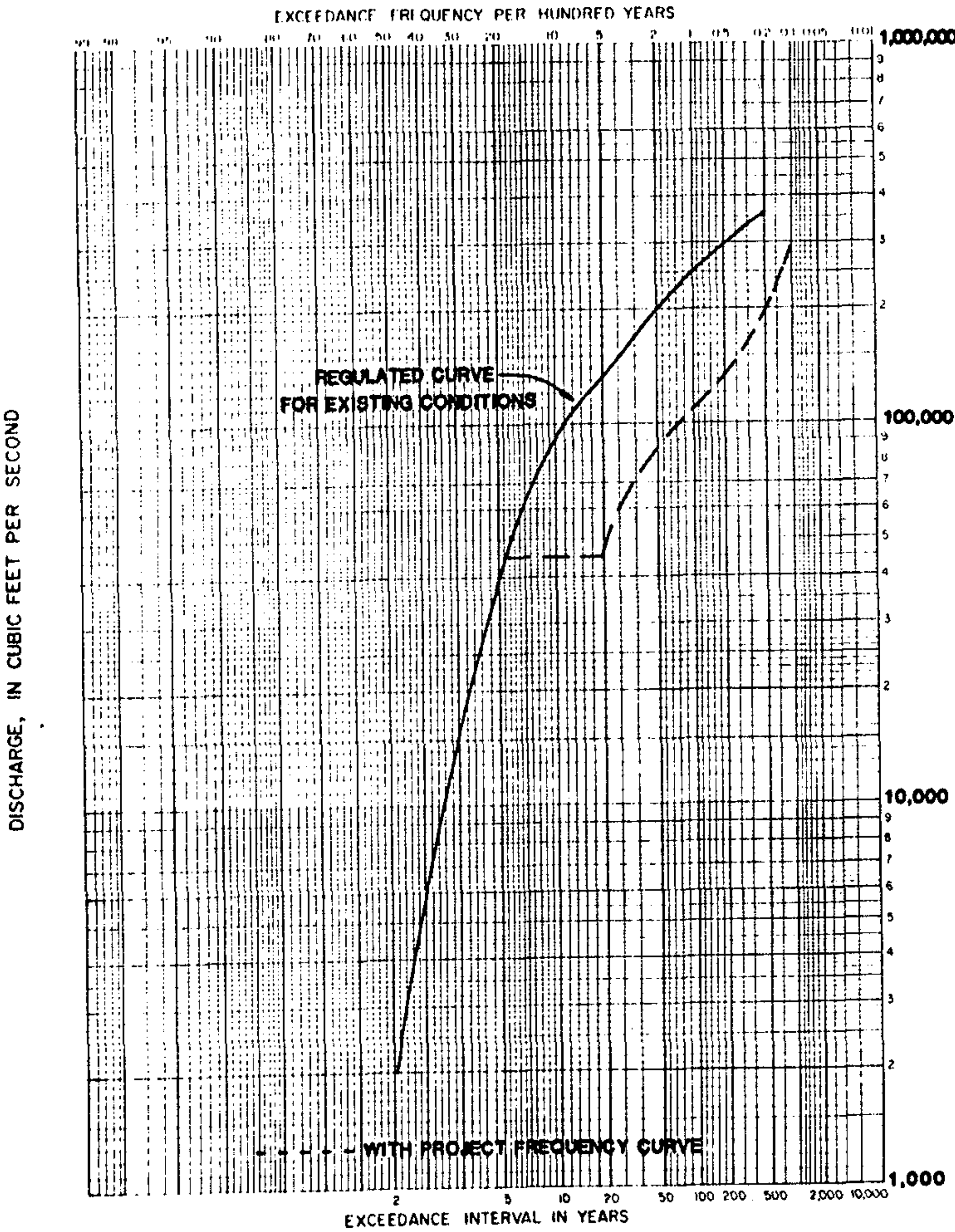
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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

PLATE 19a



ALTERNATIVE = ORME

DESIGN = SPF

TARGET = 50,000 CFS

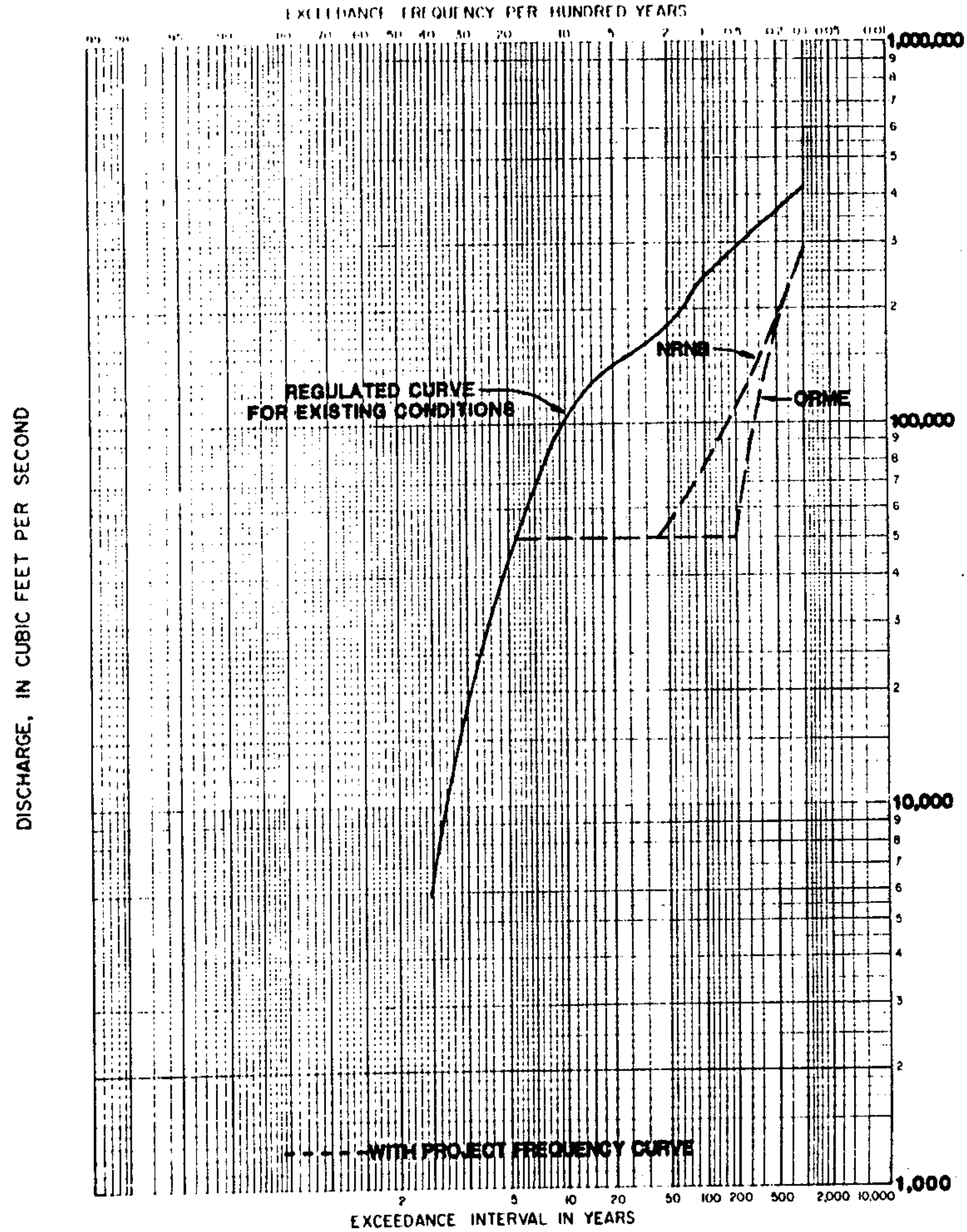
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER (CP-1310)

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

PLATE 19b



ALTERNATIVE = NRNB & ORME

DESIGN = SPF

TARGET = 50,000 CFS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

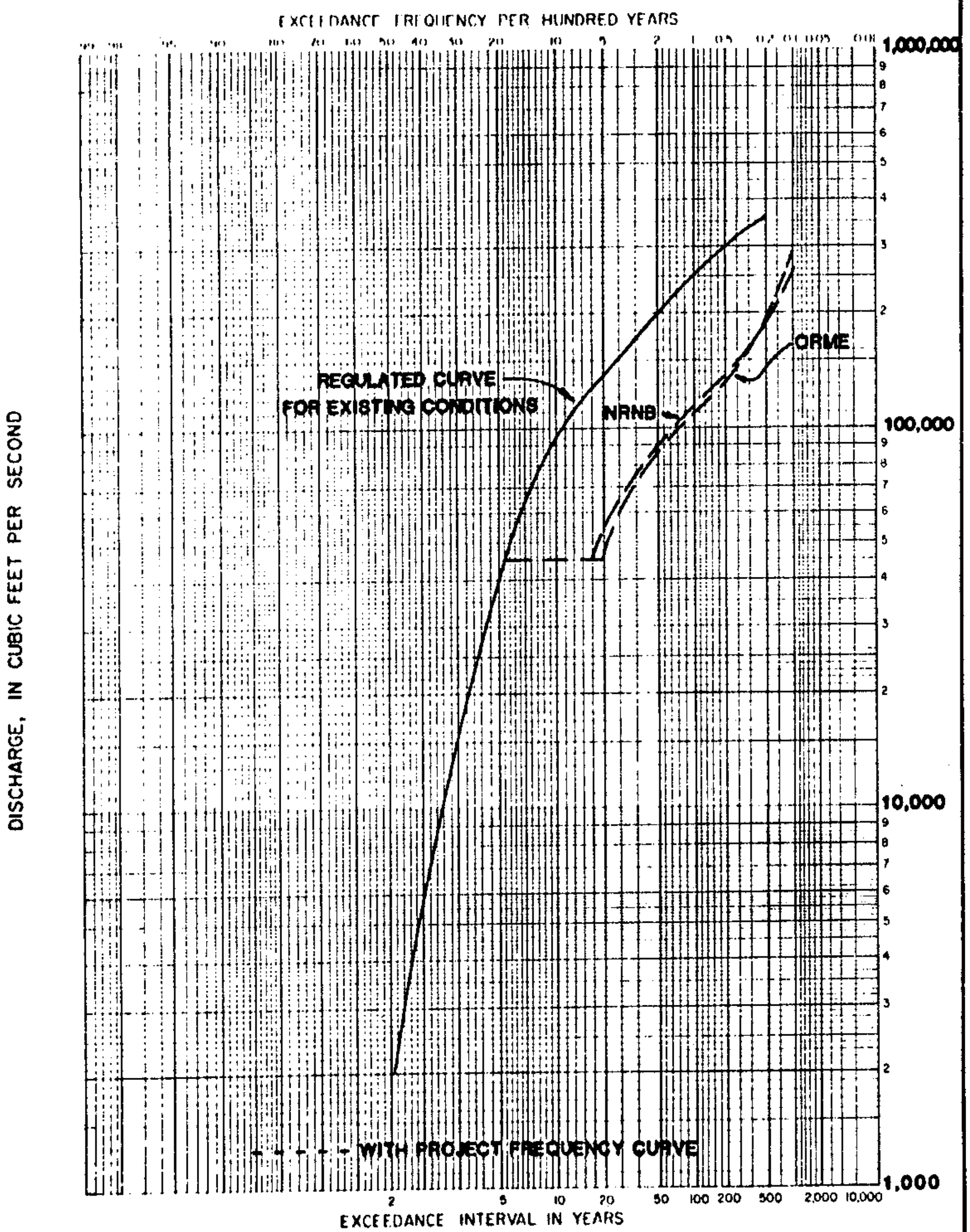
DISCHARGE-FREQUENCY CURVE

NRNB VS ORME

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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

PLATE 20a



ALTERNATIVE = NRNB & ORME

DESIGN = SPF

TARGET = 50,000 CFS

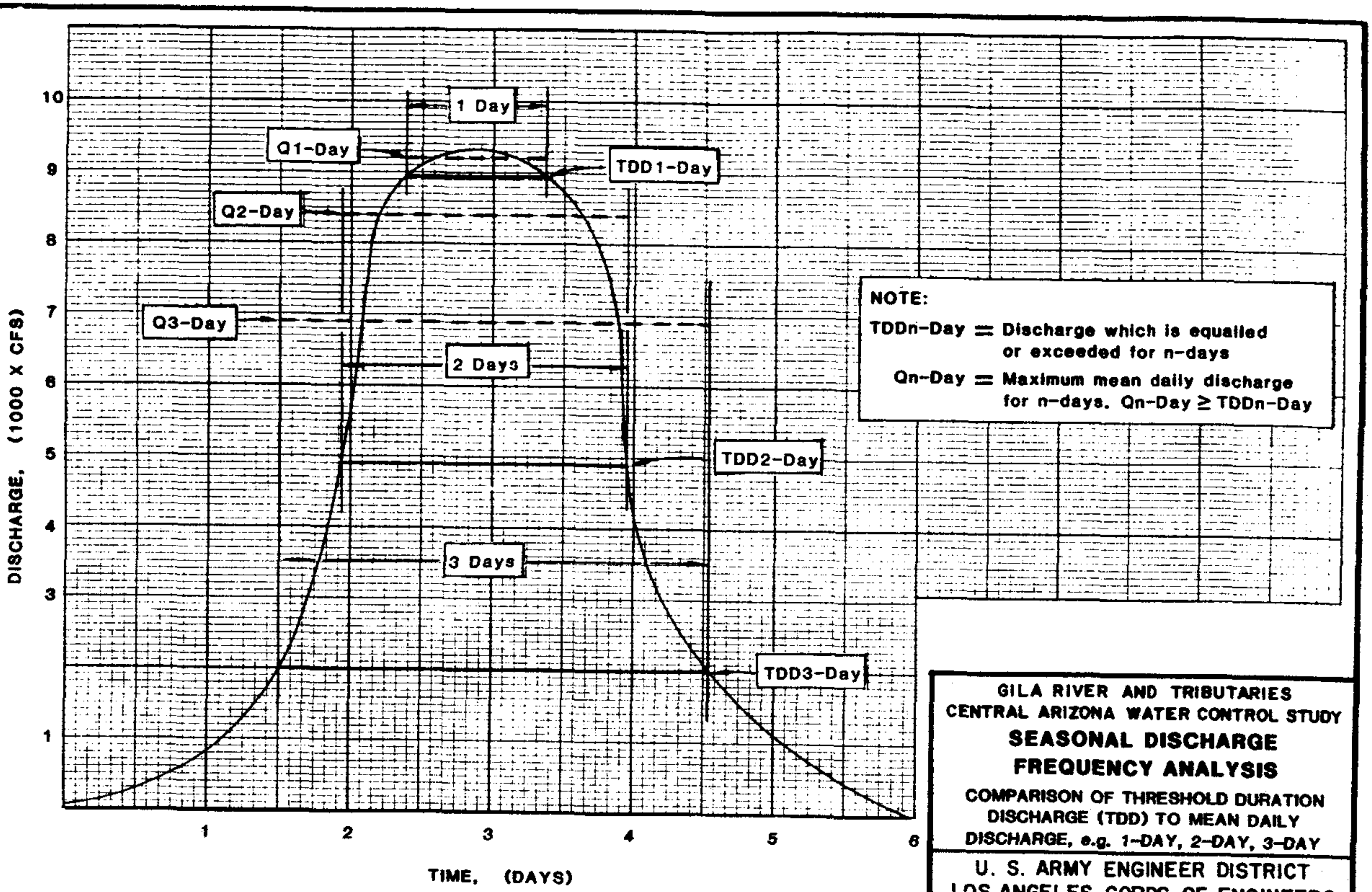
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

NRNB VS ORME

GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER (CP-1310)

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

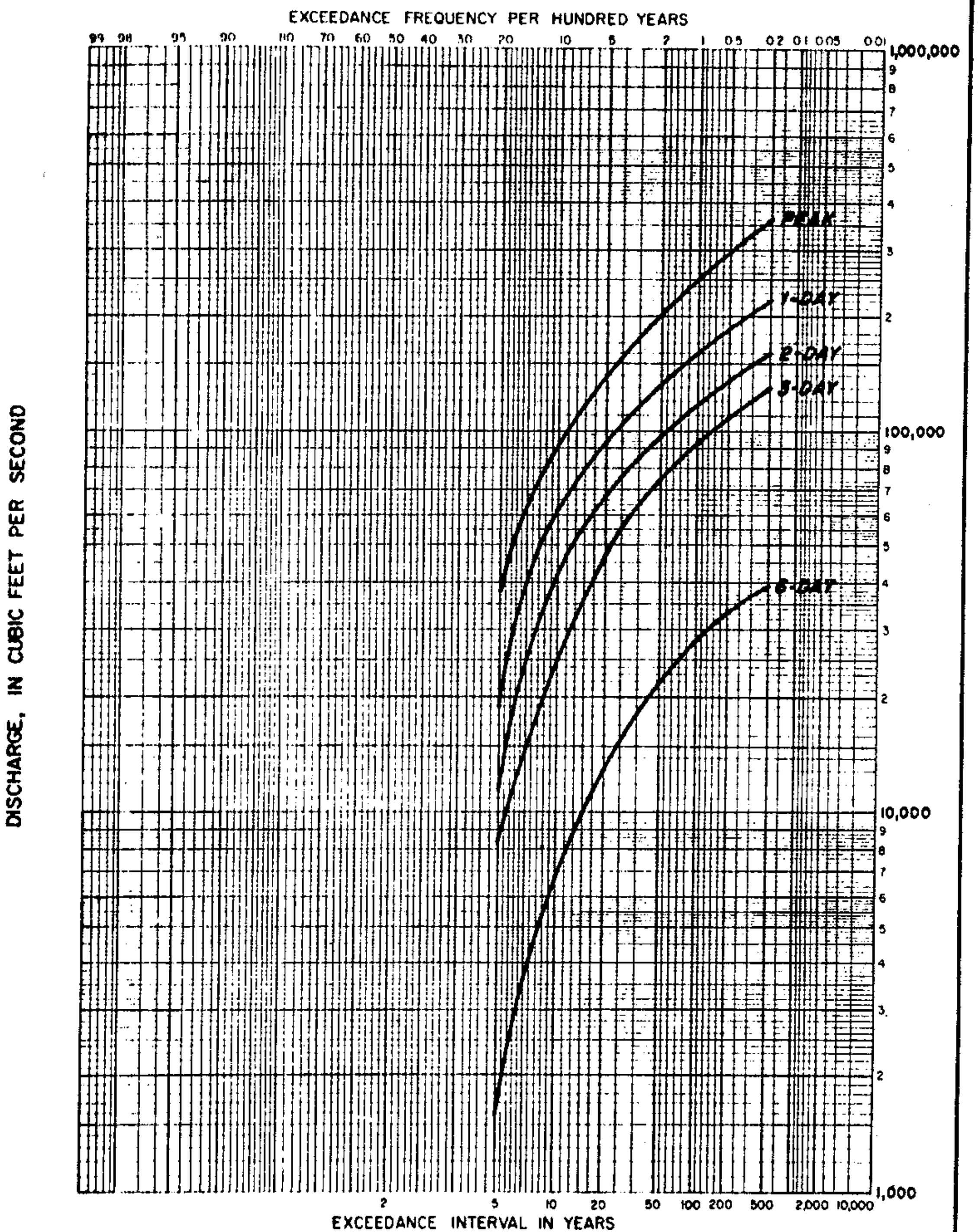


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

SEASONAL DISCHARGE FREQUENCY ANALYSIS

COMPARISON OF THRESHOLD DURATION DISCHARGE (TDD) TO MEAN DAILY DISCHARGE, e.g. 1-DAY, 2-DAY, 3-DAY

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



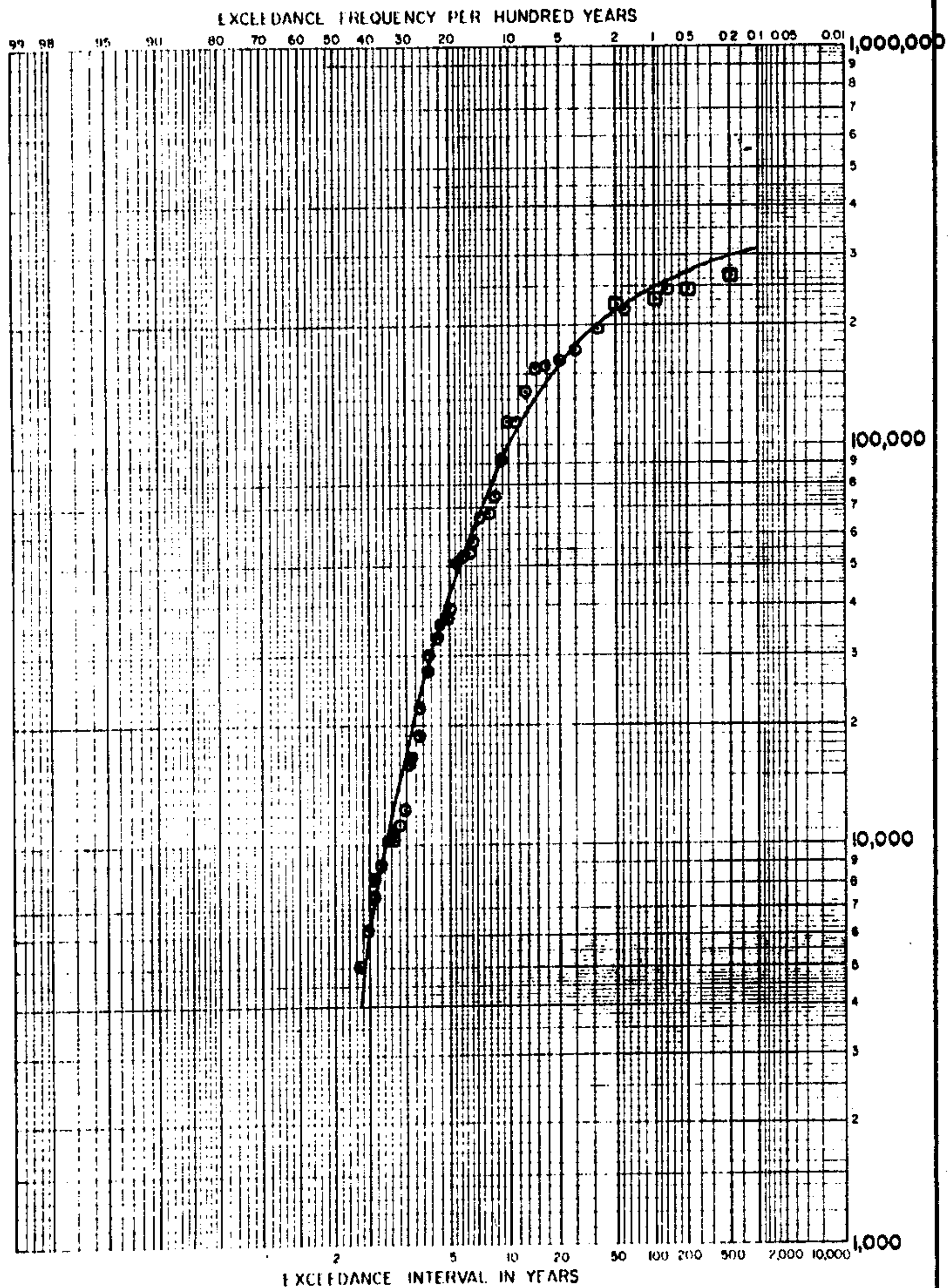
NOTE: DURATIONS REPRESENT THRESHOLD DISCHARGE, I.E. DISCHARGES WHICH ARE EQUALLED OR EXCEEDED FOR GIVEN DURATION.

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

SEASONAL DISCHARGE FREQUENCY CURVES

DECEMBER THRU APRIL
GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP 1310)

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



⊙ PERIOD-OF-RECORD DISCHARGES (W/S.O.D.)[1]

⊡ BALANCED HYDROGRAPHS (W/S.O.D.)[1]

[1] FIX AT ROOSEVELT, HORSESHOE AND BARTLETT

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

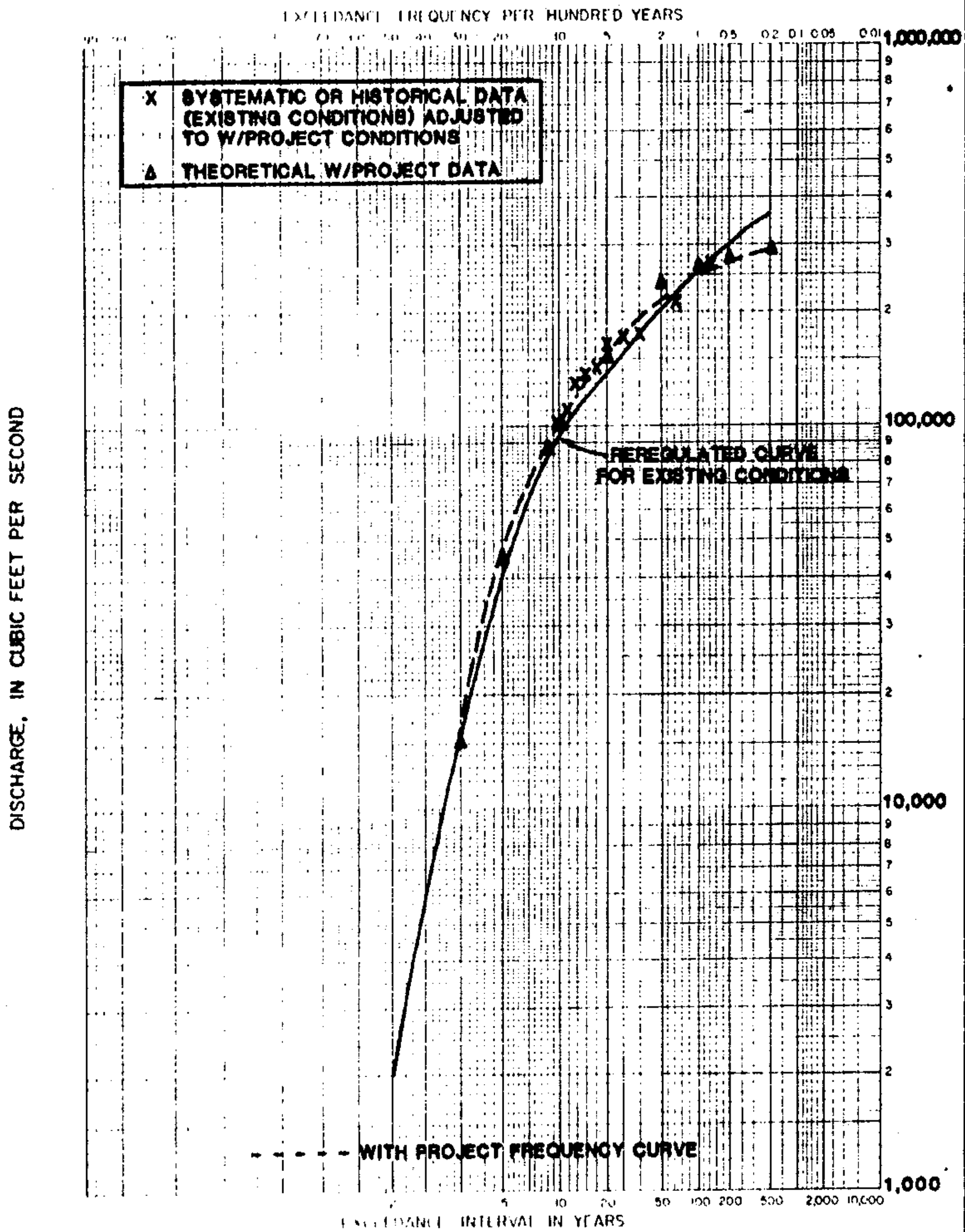
DISCHARGE-FREQUENCY CURVE

S.O.D.[1] 1ST ADDED W/O FLOOD CONTROL

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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

PLATE 23a



ALTERNATIVE = SOD 1

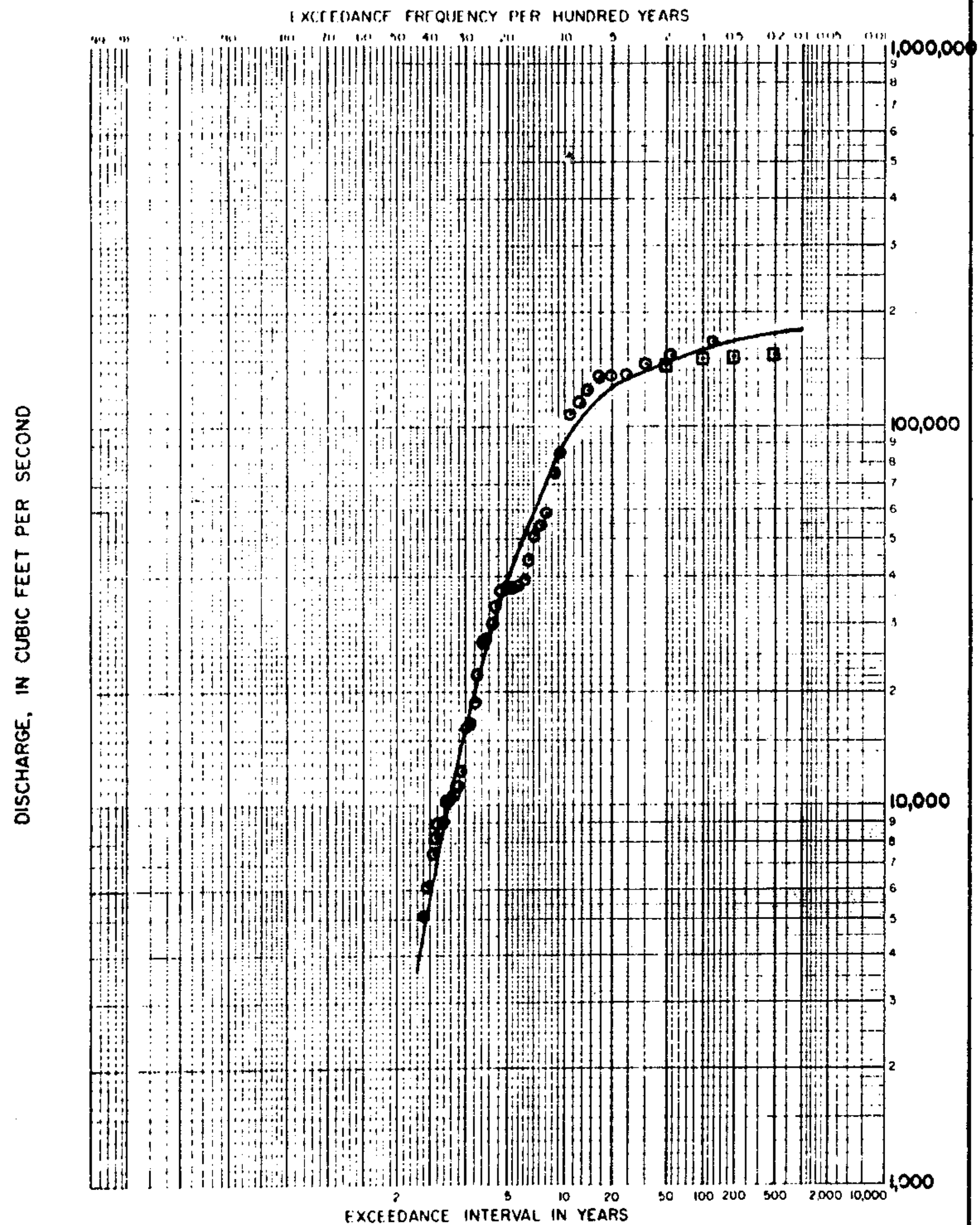
1 FIX AT ROOSEVELT, HORSESHOE, AND BARTLETT

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER (CP-1310)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



⊙ PERIOD-OF-RECORD DISCHARGES (W/S.O.D.)₂

⊡ BALANCED HYDROGRAPHS (W/S.O.D.)₂

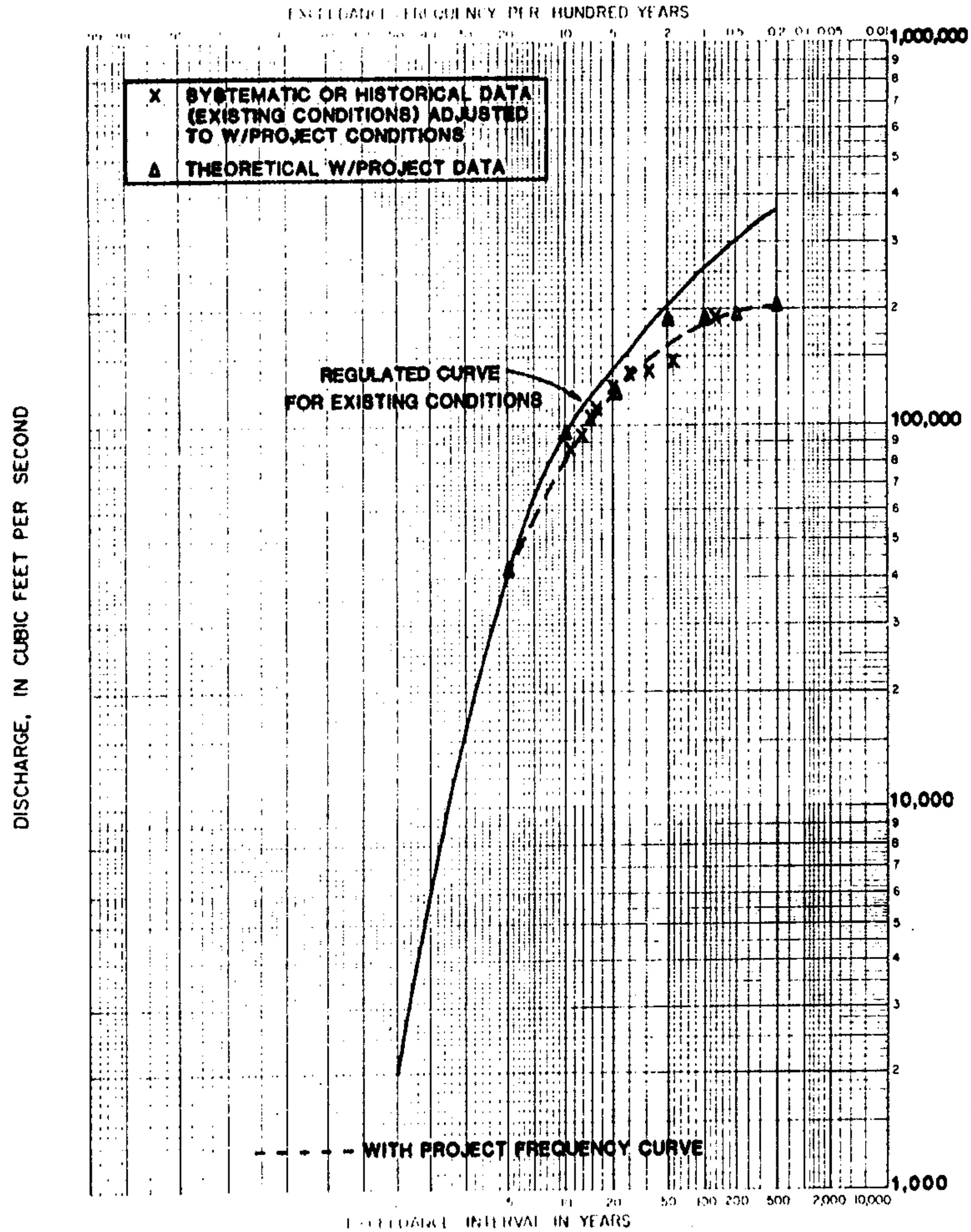
₂ FIX AT ROOSEVELT AND CLIFF

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVE
S.O.D.₂ 1ST ADDED W/O FLOOD CONTROL
SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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

PLATE 24a



ALTERNATIVE = SOD 2

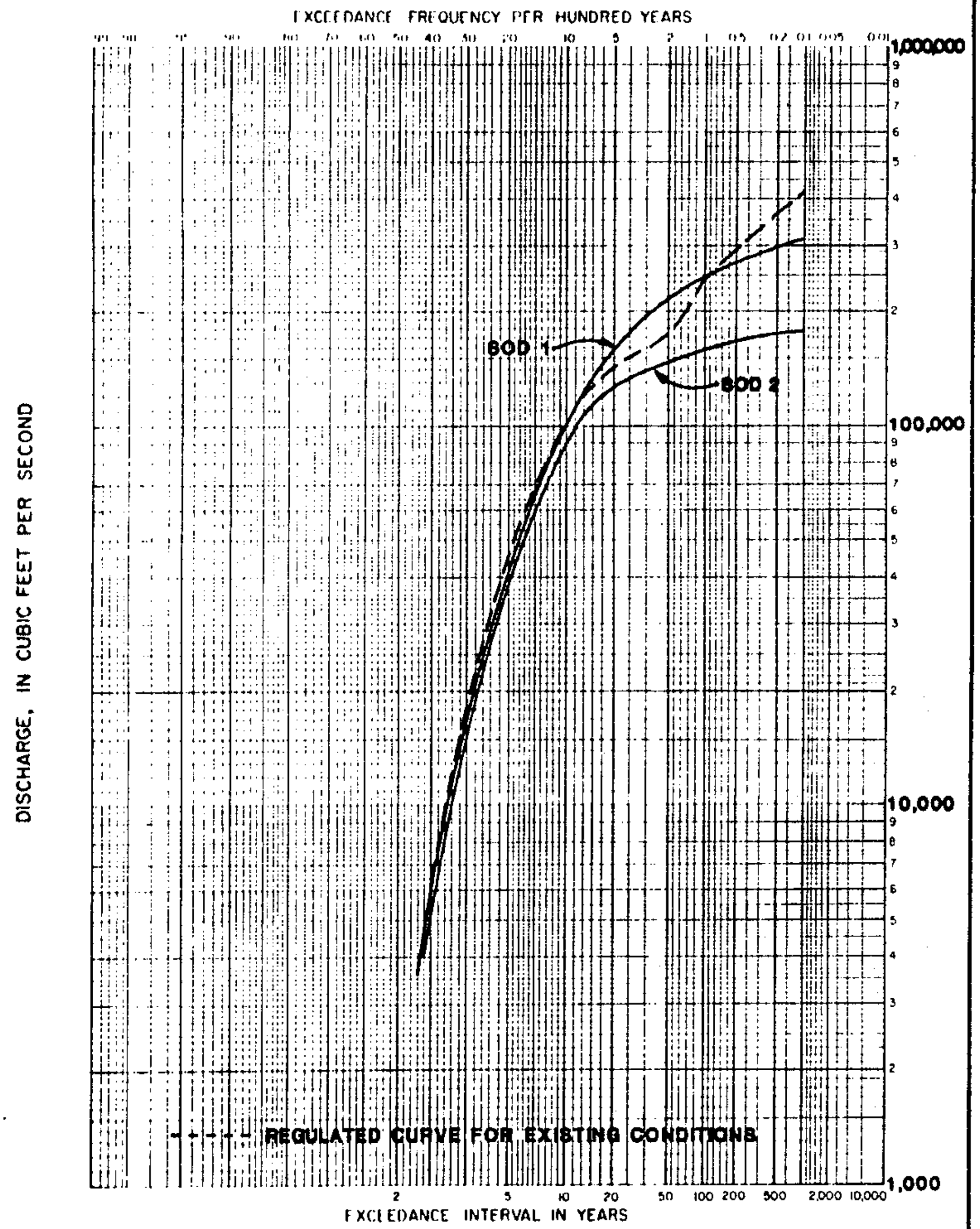
2 FIX AT ROOSEVELT AND CLIFF

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER (CP-1310)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVE
SOD 1 VS SOD 2 W/O FLOOD CONTROL

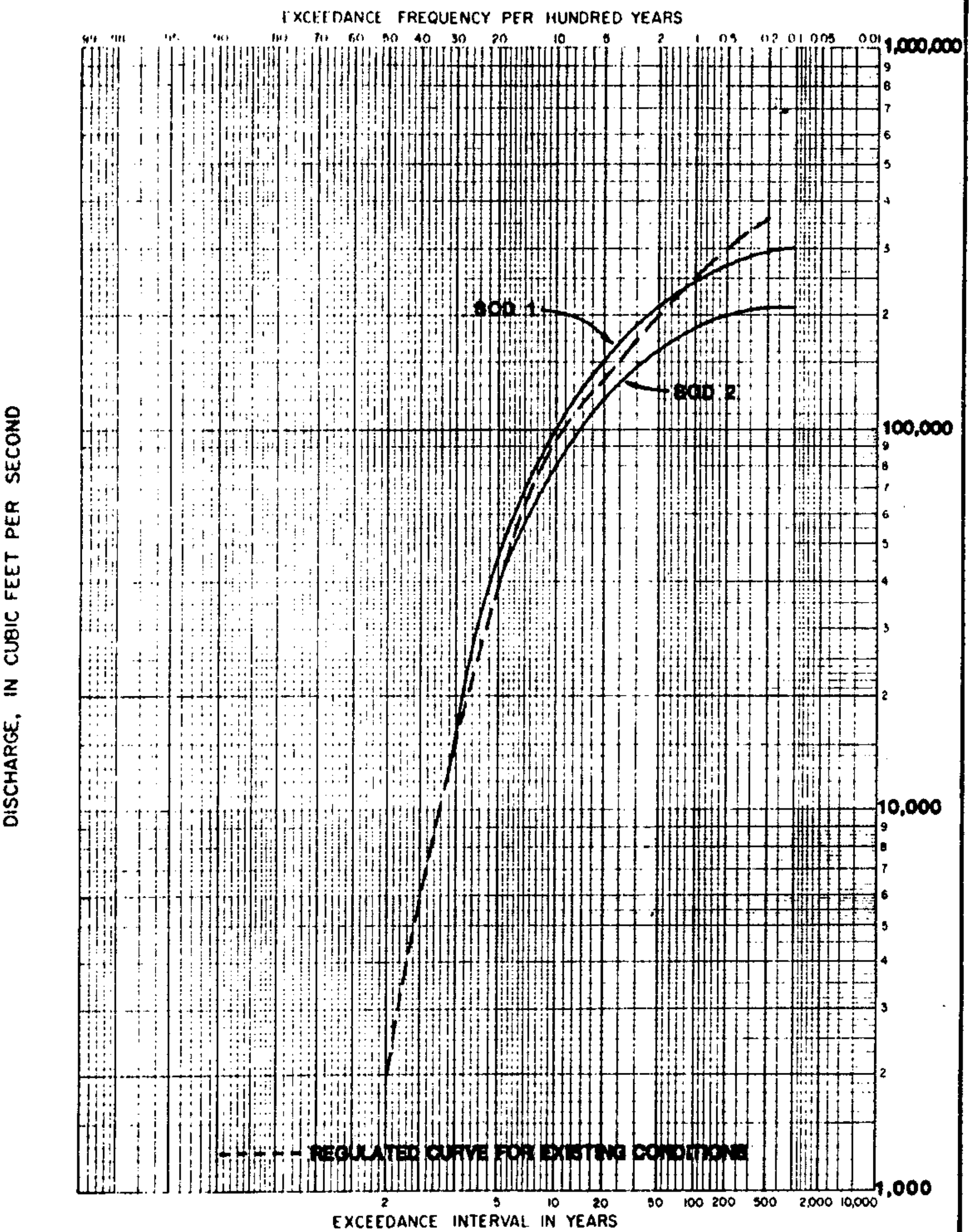
SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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

ALTERNATIVE = SOD 1 & SOD 2
W/O FLOOD CONTROL

1 FIX AT ROOSEVELT, HORSESHOE, AND BARTLETT
2 FIX AT ROOSEVELT AND CLIFF

PLATE 25a



ALTERNATIVE : : SOD 1 & SOD 2 W/O FLOOD CONTROL

1 FIX AT ROOSEVELT, HORSESHOE, AND BARTLETT
2 FIX AT ROOSEVELT AND CLIFF

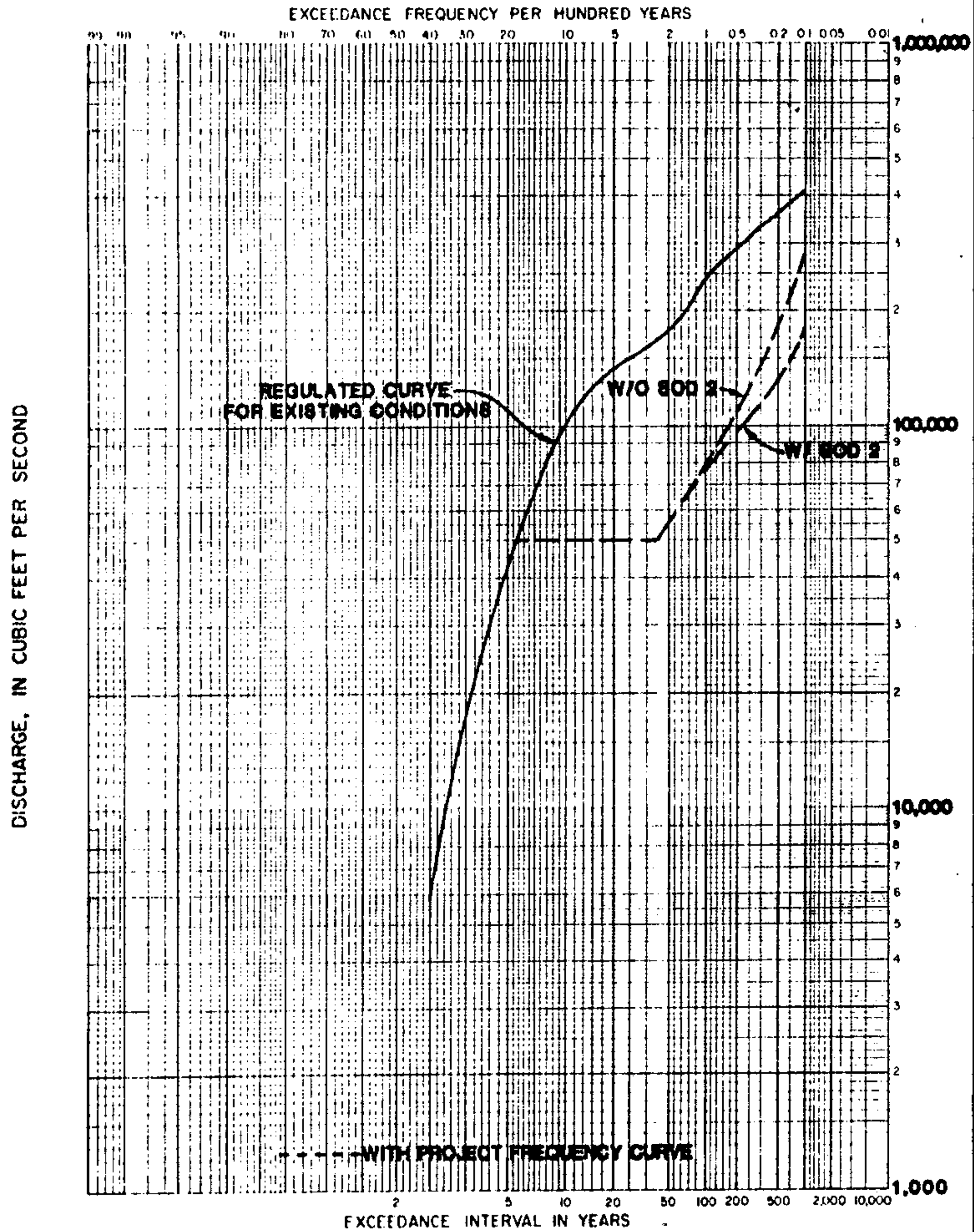
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

SOD 1 VS SOD 2 W/O FLOOD CONTROL
GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

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

PLATE 25b



ALTERNATIVE = NRNB WITH AND WITHOUT SOD 2

DESIGN = SPF

TARGET = 50,000 CFS

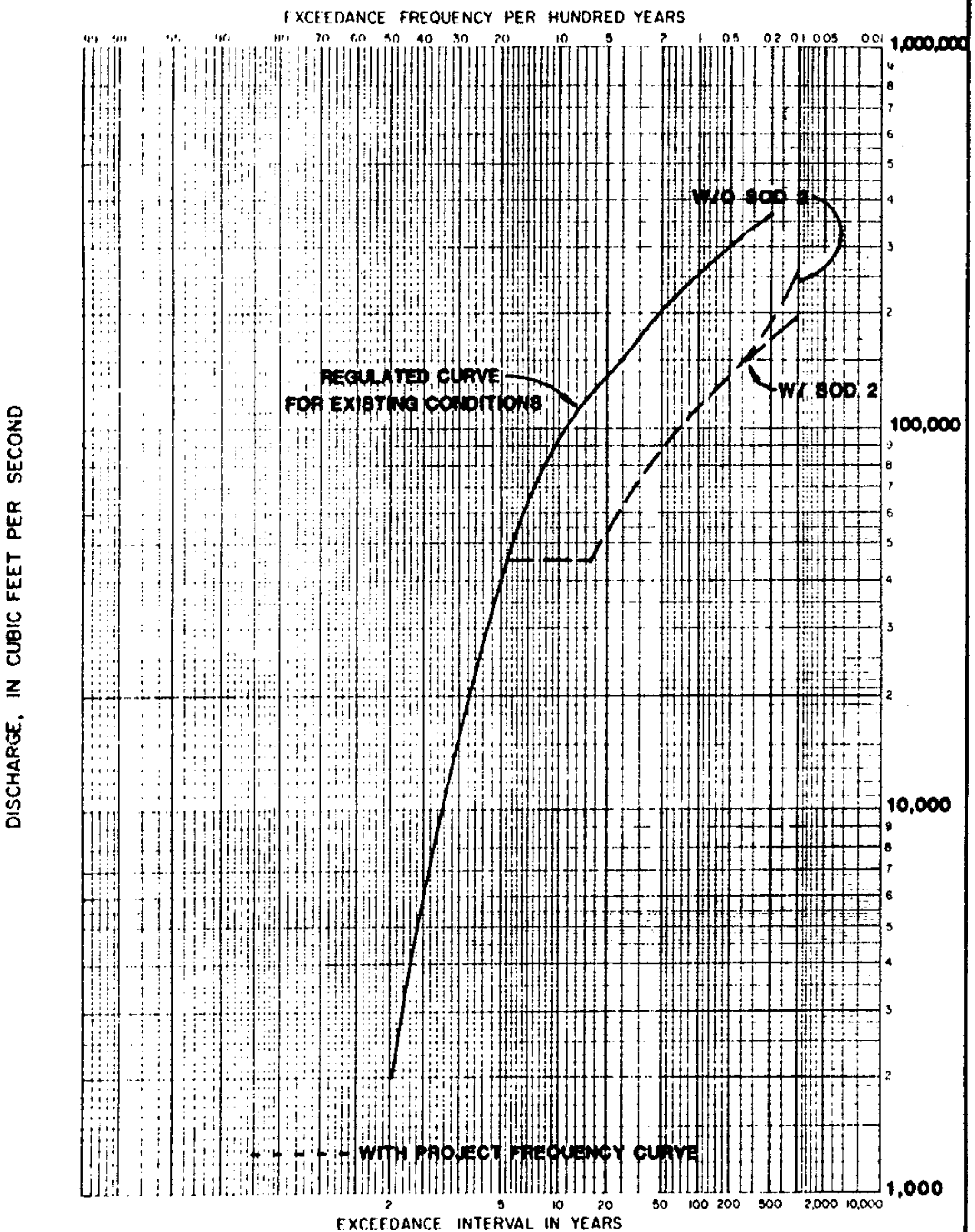
2 FIX AT ROOSEVELT AND CLIFF

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVE

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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



ALTERNATIVE = NRNB WITH
AND WITHOUT SOD 2

DESIGN = SPF

TARGET = 50,000 CFS

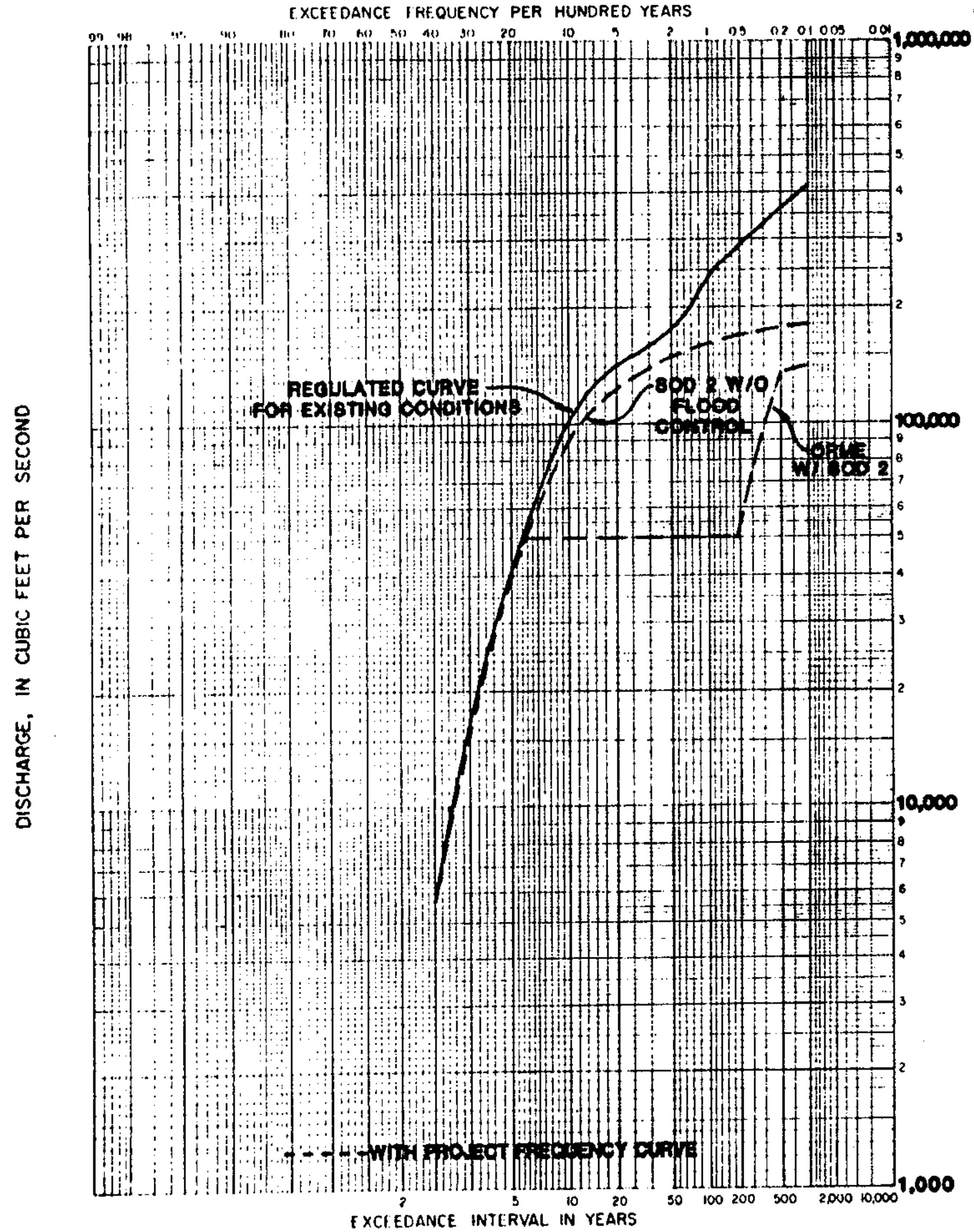
2 FIX AT ROOSEVELT AND CLIFF

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER (CP-1310)

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



ALTERNATIVE = ORME WITH SOD 2

DESIGN = SPF

TARGET = 50,000 CFS

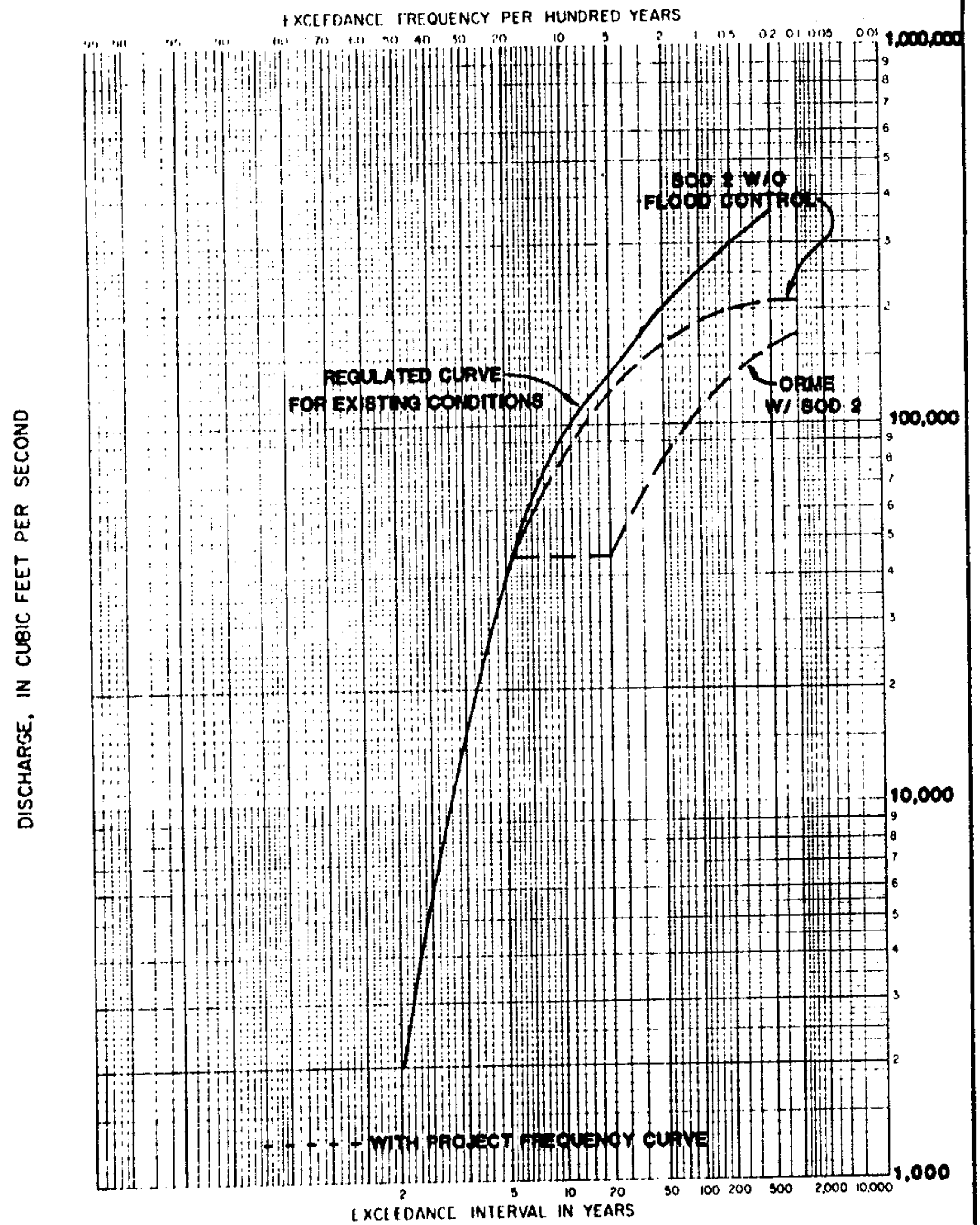
2 FIX AT ROOSEVELT AND CLIFF

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVE

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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



GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER (CP-1310)

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

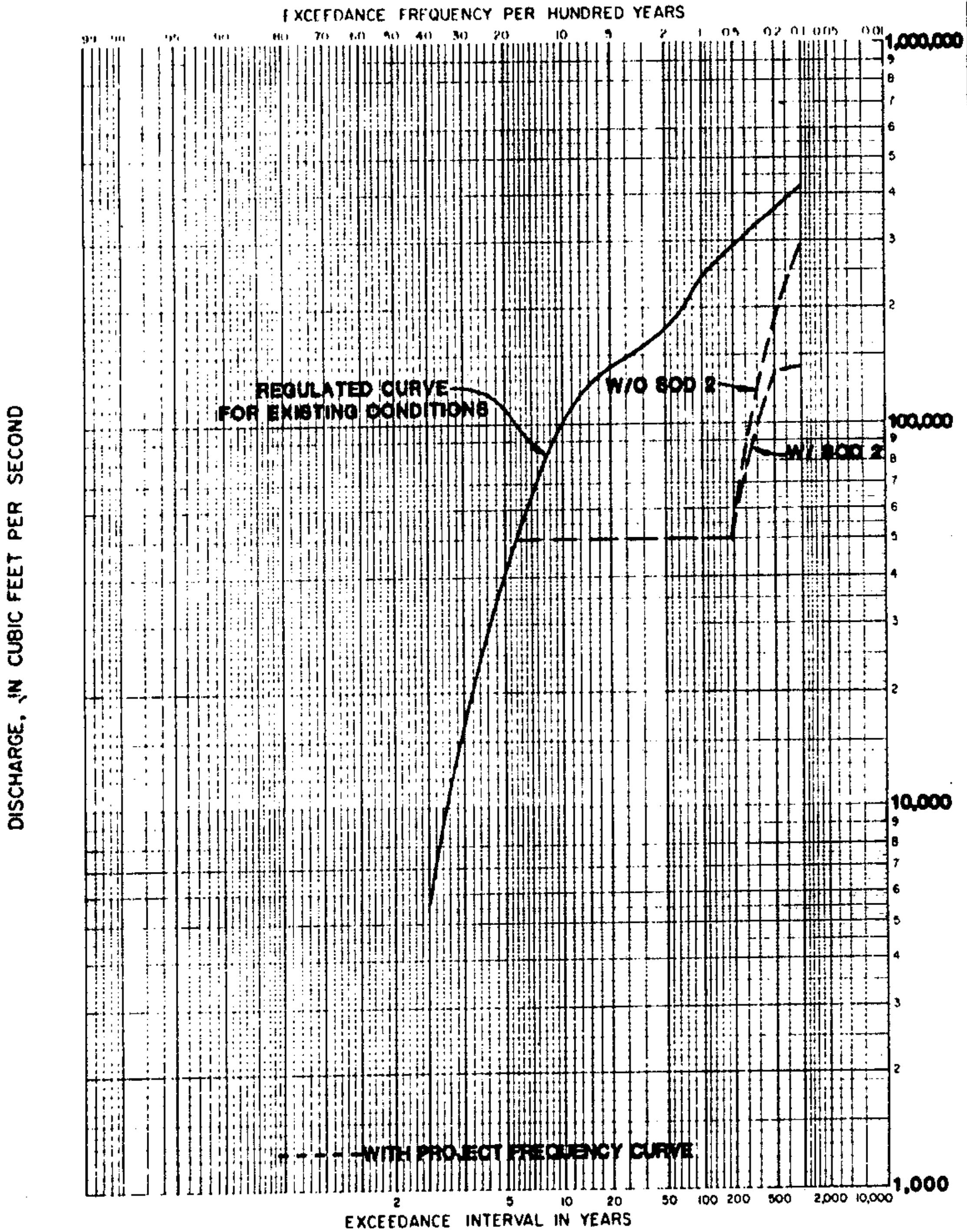
ALTERNATIVE = ORME WITH SOD 2

DESIGN = SPF

TARGET = 50,000 CFS

2 FIX AT ROOSEVELT AND CLIFF

PLATE 27b



ALTERNATIVE = ORME WITH AND WITHOUT SOD 2

DESIGN = SPF

TARGET = 50,000 CFS

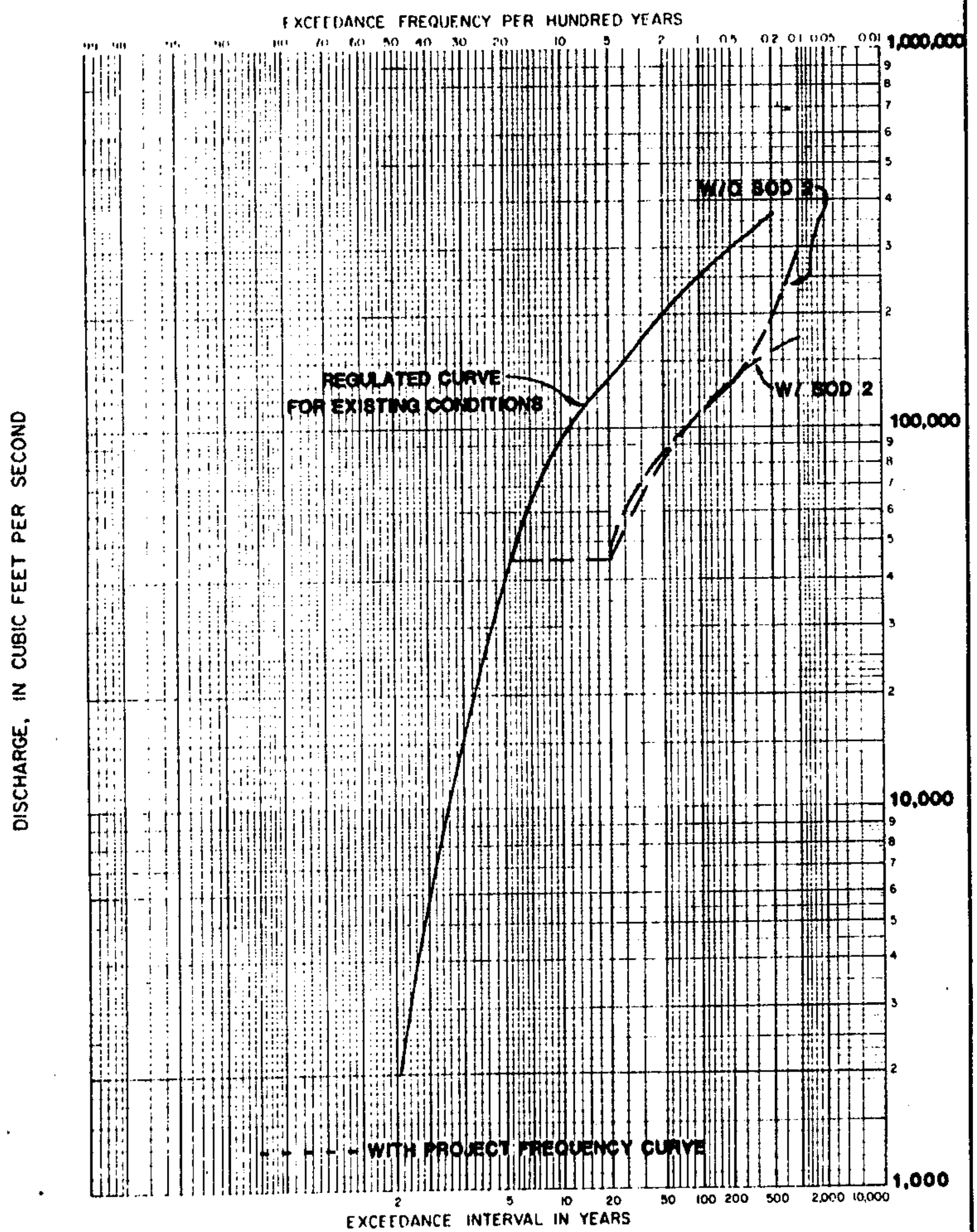
2 FIX AT ROOSEVELT AND CLIFF

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVE

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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



ALTERNATIVE = ORME WITH AND WITHOUT SOD 2

DESIGN = SPF

TARGET = 50,000 CFS

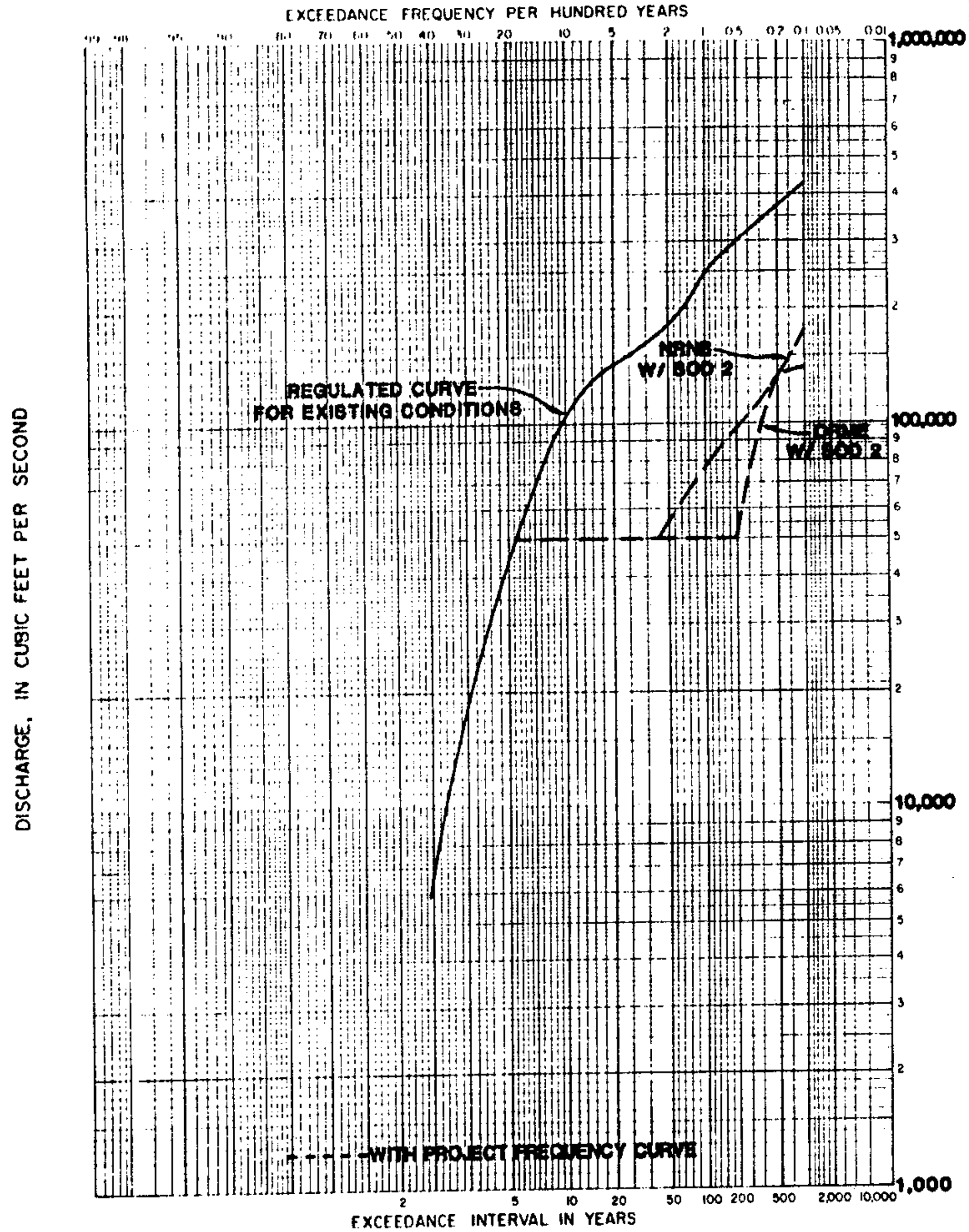
2 FIX AT ROOSEVELT AND CLIFF

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

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



ALTERNATIVE = NRNB & ORME WITH SOD 2

DESIGN = SPF

TARGET = 50,000 CFS

2 FIX AT ROOSEVELT AND CLIFF

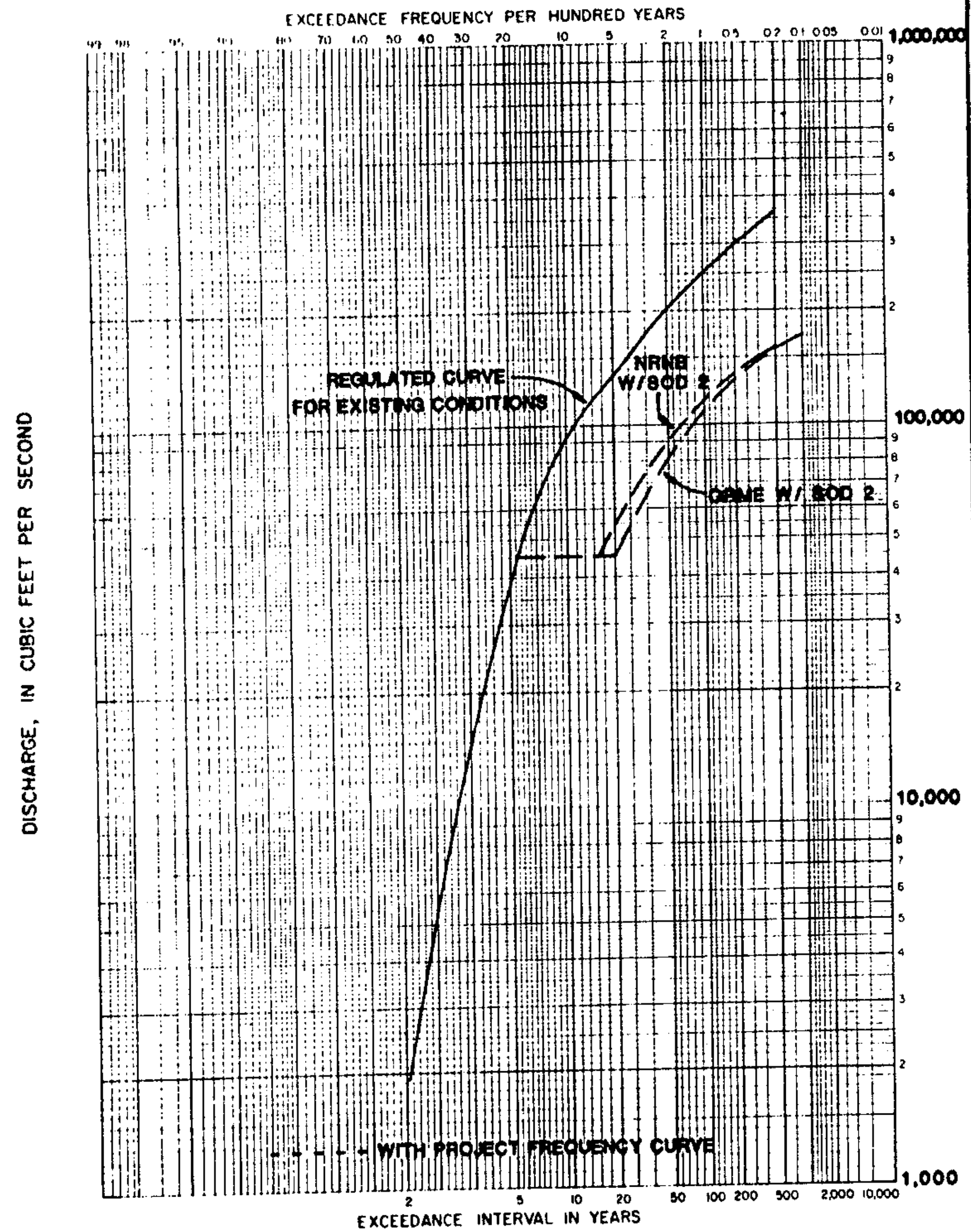
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVE

NRNB VS ORME WITH SOD 2
SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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

PLATE 29a



ALTERNATIVE = NRNB & ORME WITH SOD 2

DESIGN = SPF

TARGET = 50,000 CFS

2 FIX AT ROOSEVELT AND CLIFF

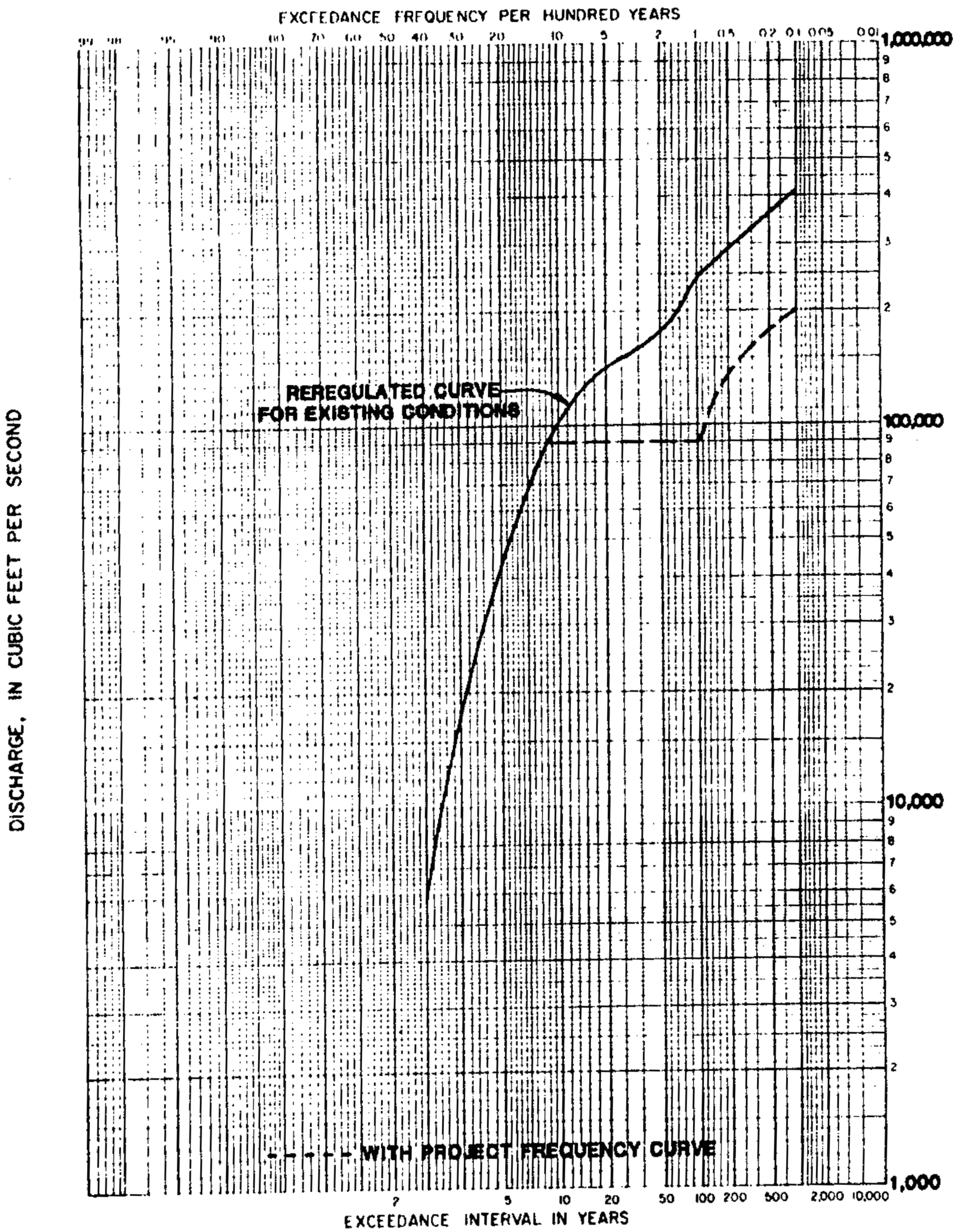
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

NRNB VS ORME WITH SOD 2
GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

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

PLATE 29b



ALTERNATIVE = REREGULATION WITH SOD 1 OR SOD 2

DESIGN = SPF

TARGET = 90,000 CFS

1 FIX AT ROOSEVELT, HORSESHOE, AND BARTLETT
2 FIX AT ROOSEVELT AND CLIFF

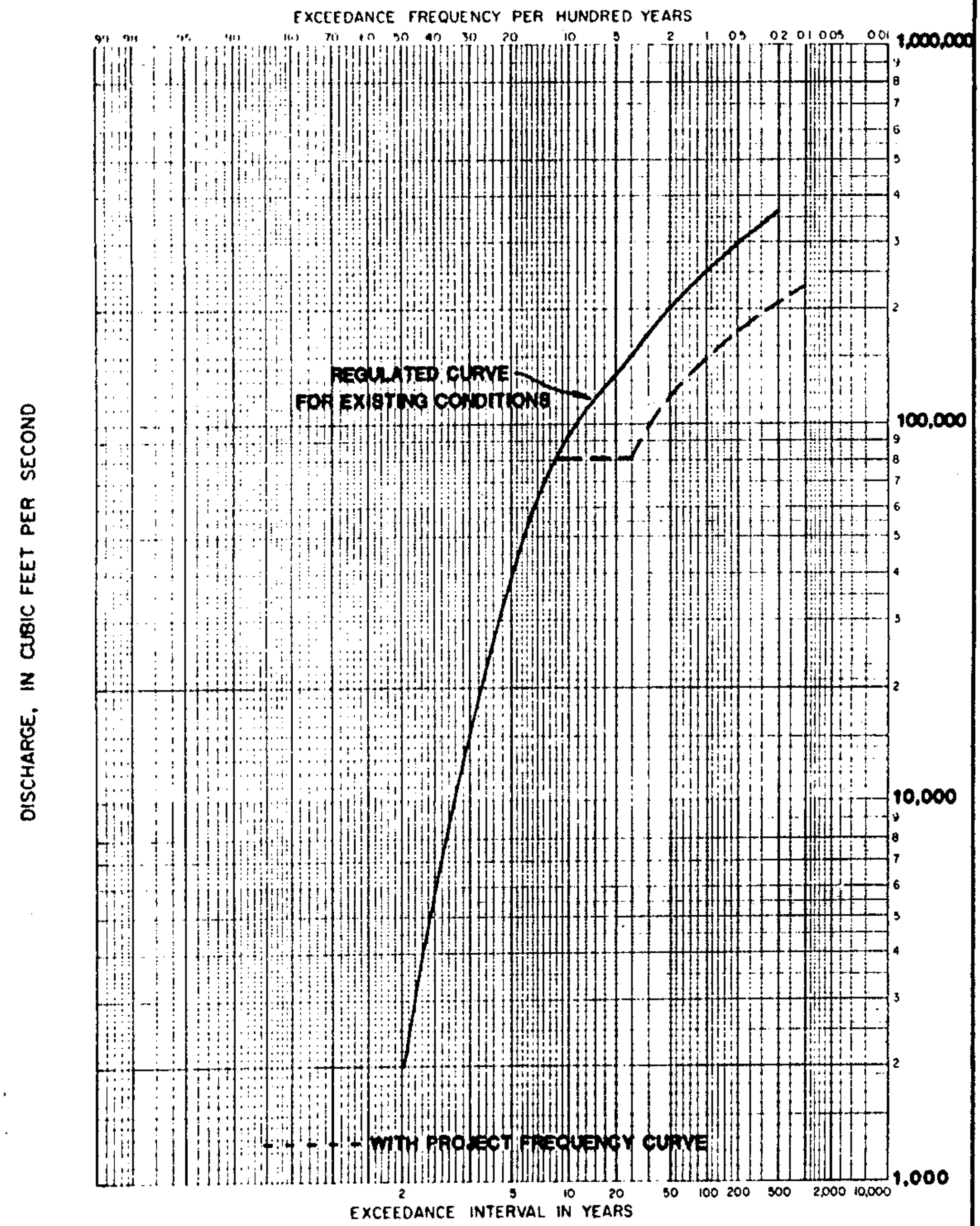
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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

PLATE 30a



ALTERNATIVE = REREGULATION
WITH SOD 1 OR SOD 2

DESIGN = SPF

TARGET = 90,000 CFS

1 FIX AT ROOSEVELT, HORSESHOE, AND BARTLETT
2 FIX AT ROOSEVELT AND CLIFF

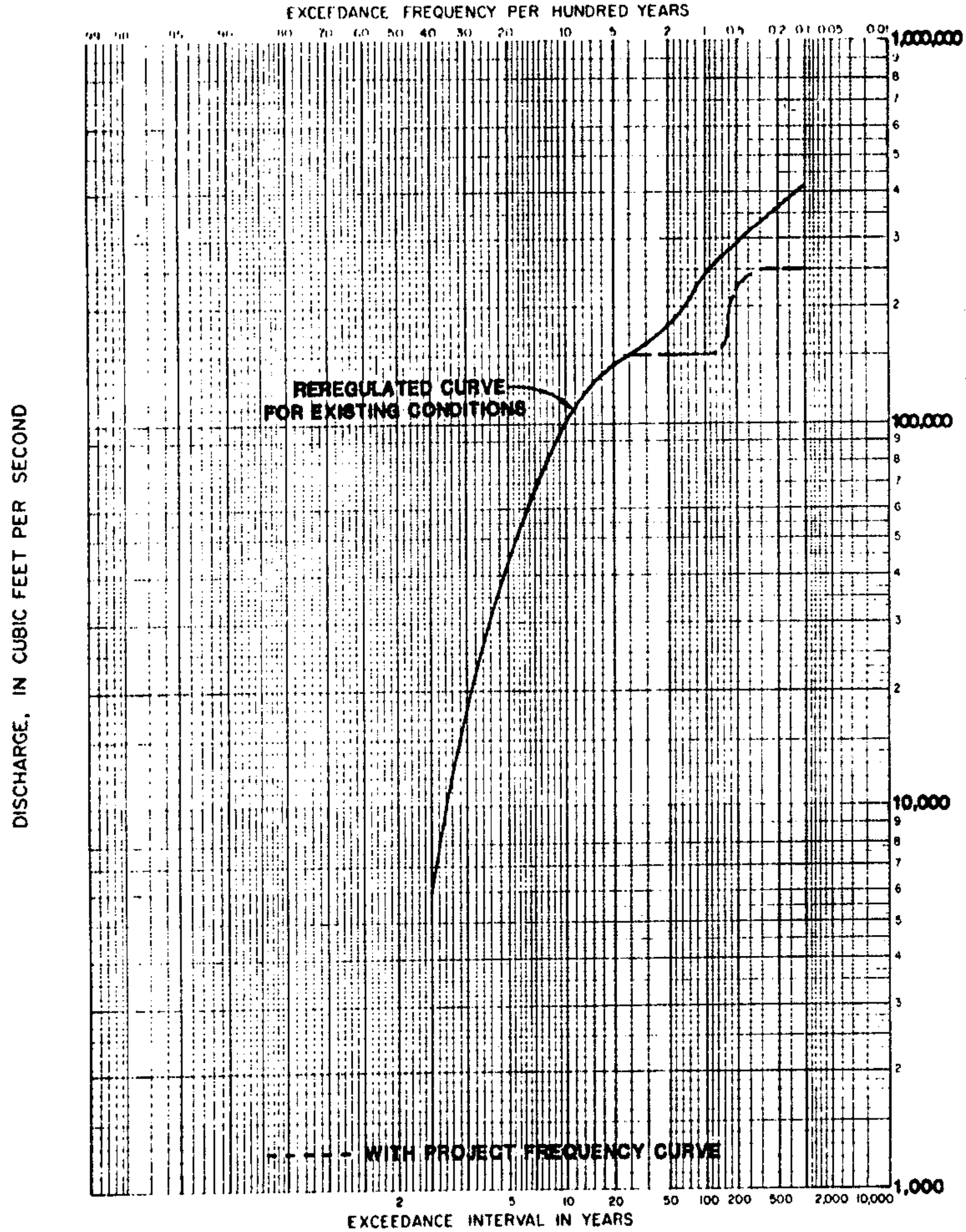
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE
WITH SALT RIVER (CP-1310)

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

PLATE 30b



ALTERNATIVE = REREGULATION WITH SOD 1 OR SOD 2

DESIGN = SPF

TARGET = 150,000 CFS

1 FIX AT ROOSEVELT, HORSESHOE, AND BARTLETT
2 FIX AT ROOSEVELT AND CLIFF

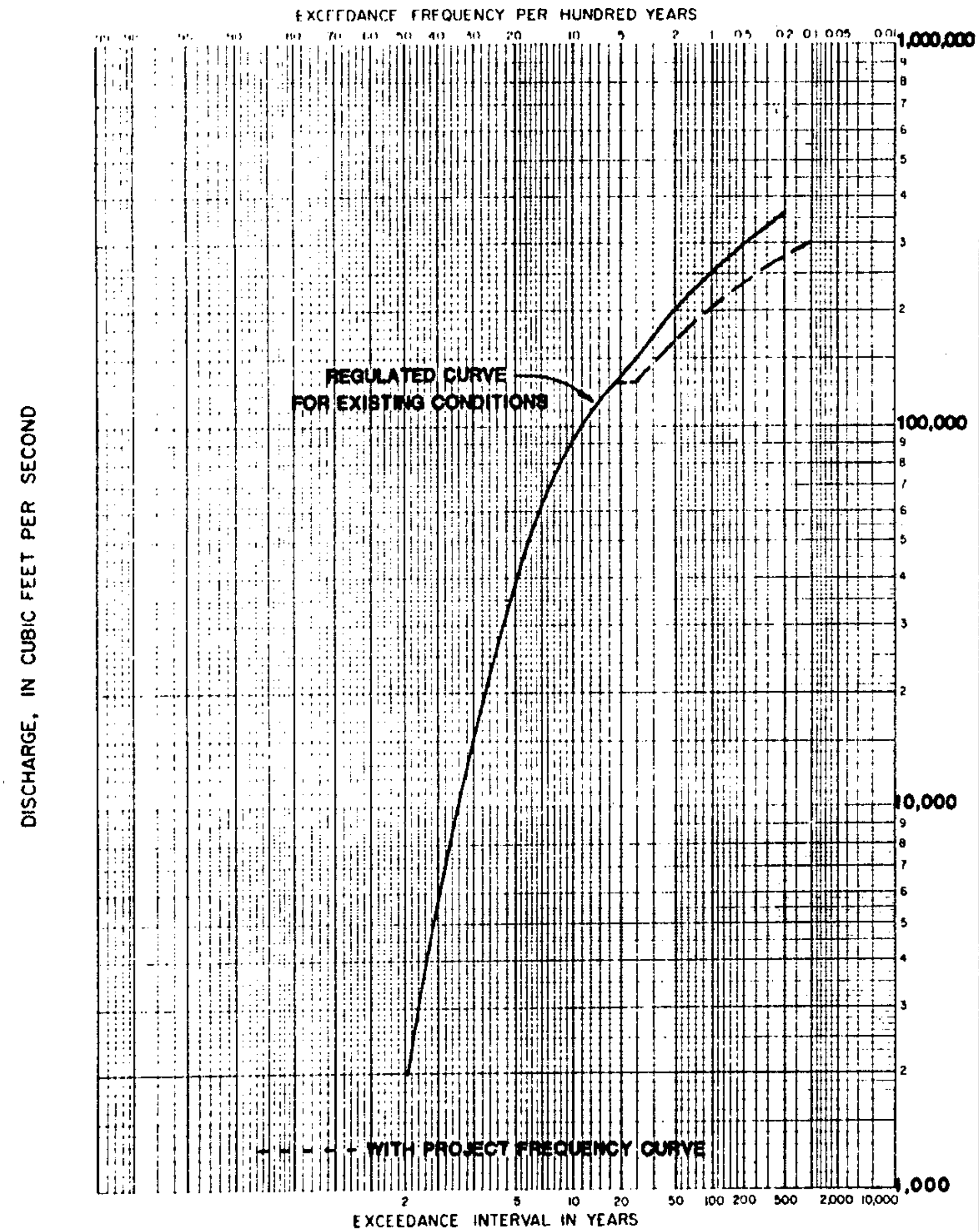
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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

PLATE 31a



ALTERNATIVE = REREGULATION WITH SOD 1 OR SOD 2

DESIGN = SPF

TARGET = 150,000 CFS

1 FIX AT ROOSEVELT, HORSESHOE, AND BARTLETT
2 FIX AT ROOSEVELT AND CLIFF

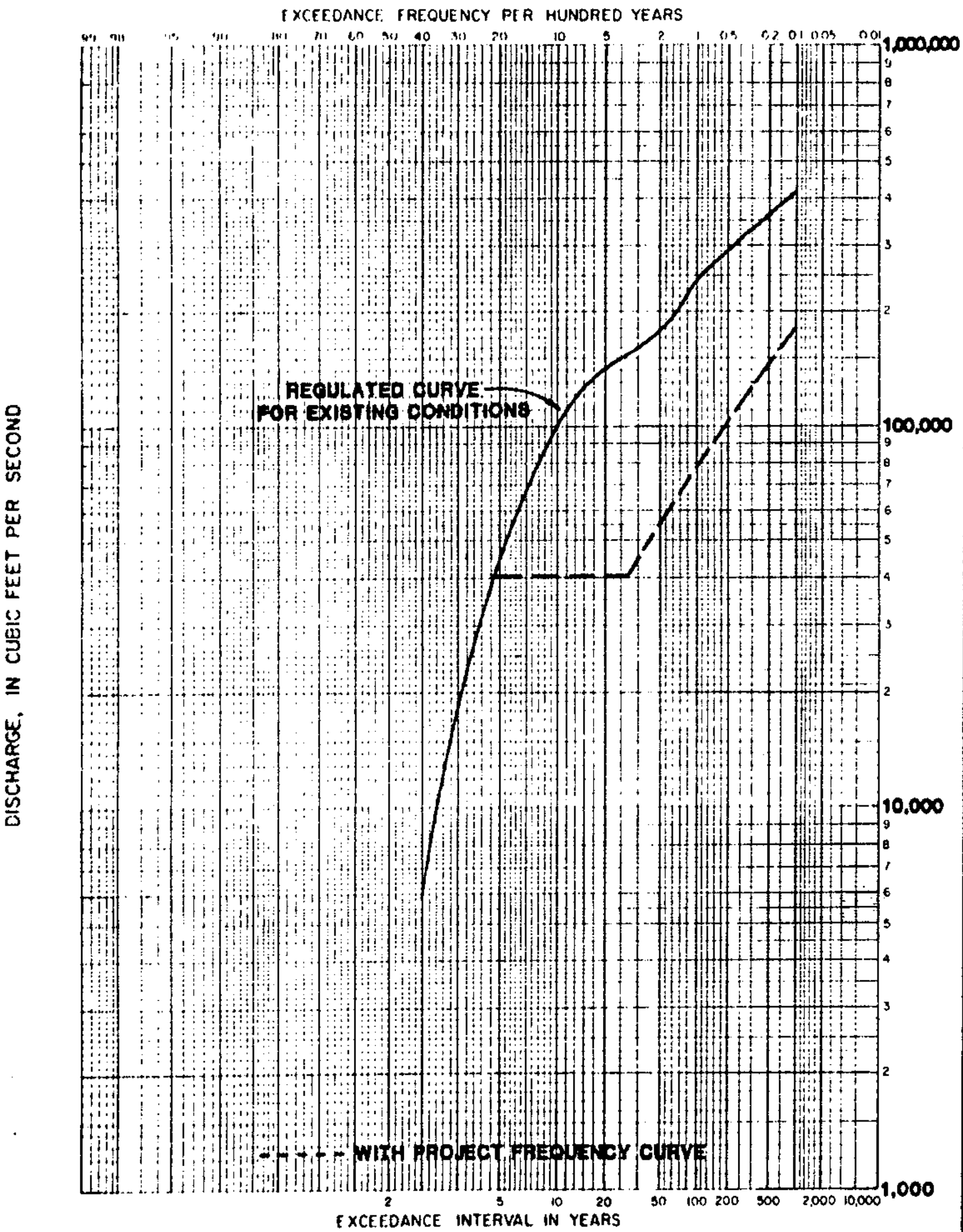
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

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

PLATE 31b



ALTERNATIVE = NRNB WITH REGULATORY STORAGE AT SALT-VERDE CONFLUENCE

DESIGN = SPF

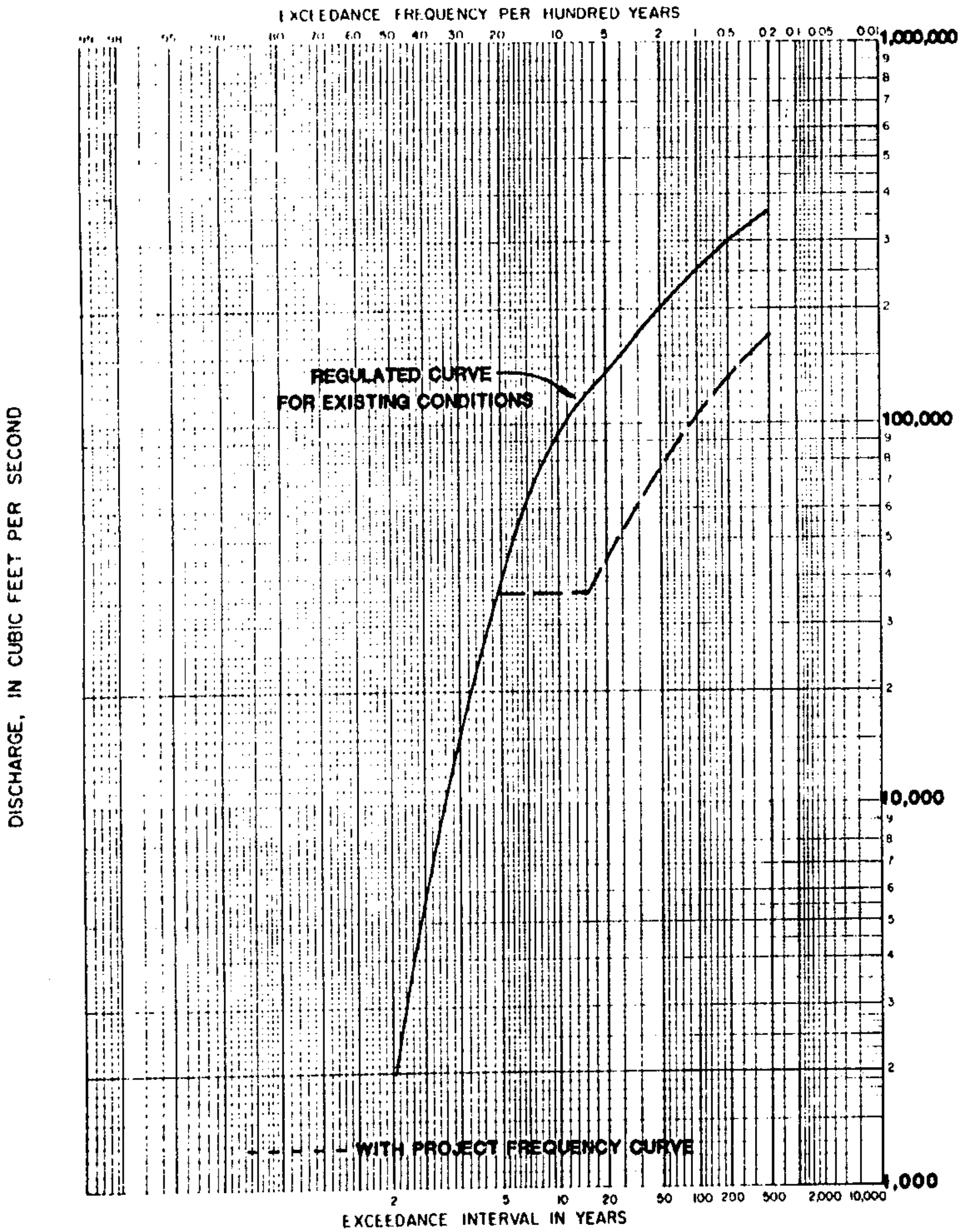
TARGET = 50,000 CFS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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



ALTERNATIVE = NRNB WITH REGULATORY STORAGE AT SALT-VERDE CONFLUENCE

DESIGN = SPF

TARGET = 50,000 CFS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP-1310)

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

PLATE 32b

GILA RIVER AND TRIBUTARIES

CENTRAL ARIZONA WATER CONTROL STUDY

APPENDIX 1

LOCAL FLOW ANALYSIS

STAGE I

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

May 1982

Appendix 1. Local Flow Analysis, Stage I

Table of Contents

Tables

Plates

Plates (Continued)

Appendix 1
Local Flow

1. Purpose.

This appendix presents a detailed hydrologic analysis and results for the hypothetical case of complete containment of inflow to Roosevelt and Horseshoe Dams. The SPF and discharge frequency curves were derived for the 1476 sq. mi. drainage area below Roosevelt and Horseshoe Dams (Plate 1-1) for the following concentration points:

a. Granite Reef Dam, CP-91

b. Tempe Bridge, CP-93

c. Central Avenue, CP-101 and

d. the confluence of the Salt and Gila Rivers, CP-103

2. DRAINAGE AREA.

The study drainage area of 1476 sq. mi. excludes the area within subareas 9 and 10 as follows. Subarea 9 north of the Paradise Valley Detention Dike (PVDD) and the small drainage area contributing to Dreamy Draw Dam were considered to be non-contributary. Subarea 10 north of the Grand Canal from the Tempe Bridge (CP 93) on the east to the western boundary of the drainage area was also deleted. Further, the remainder of drainage area 10 was subdivided into three subareas 10, 11, and 12, corresponding to Central Ave (CP-101), 67th Ave. (CP-102), and above the confluence of the Salt and Gila Rivers (CP-103). Subarea sizes are included in table 1.

Reasons for the reduced drainage area follow:

a. the most intense rainfall from the Standard Project Storm (SPS) centering was comparable to a 2-yr rainfall over most of the City of Phoenix and suburbs; therefore the PVDD will contain runoff in subarea 9.

b. similary, in subarea 10 the runoff will be contained by the Arizona Canal Diversion Channel (ACDC), and since the remaining contributory drainage area north of Grand Canal is thus much reduced, and the intensities of rainfall are low, it was assumed that the Grand Canal would deter all remaining excess rainfall between the ACDC and the Grand Canal.

c. in addition, both subareas have been further reduced by flood control structures-Dreamy Draw Dam in subarea 9, and Cave Buttes Dam in subarea 10.

3. STANDARD PROJECT FLOOD.

a. Standard Project Storm- The hybrid 1916-1938 Standard Project Storm presented in reference 2 of the main report was adopted as the Standard Project Storm for the drainage area below Roosevelt and Horseshoe Dams because of several reasons:

(1) to maintain consistency with previously documented COE hydrology cited previously in the 1957 report.

(2) because thunderstorm events are too localized to produce sustained high peaks at the concentration points in question (e.g. the June 1972 flood on Indian Bend Wash) due to the drainage area size percolation, and storage losses, a general type project storm was necessitated.

(3) likewise, although general summer storms have produced large discharges in some streams in the watershed above the project drainage area (e.g. the Labor Day 1970 flood on Tonto Creek), loss rates have historically been considerably higher for summer events; thus runoff is reduced considerably compared to winter events, which frequently occur as follow-up storms with antecedent conditions more conducive to the type of sustained flow necessary to produce large peaks in the Salt River near Phoenix.

(4) and finally, this was the most severe runoff-producing general winter storm for which adequate meteorologic information was available (the March 1978 storm was similar in magnitude and greater in intensity in some locations; preliminary estimates, however, indicated the results would be very similar to 1916-1938 storm results).

Isohyets of the 1916-1938 storm were centered over the 1476 sq. mi. lower Salt-Verde River system to produce the most severe rainfall depths consistent with normal annual and mean seasonal values in the drainage area (Plate 1-2). Maximum rainfall was positioned over the upper Sycamore Creek and the mountainous area contributary to the Salt River below Roosevelt Dam. After resulting depths of rainfall were determined in each subarea, a comparison to mean seasonal and normal annual depths of precipitation was made to check the validity of the storm transposition. Subarea depths which were not substantiated by the norms were adjusted based on the ratio of storm depth to mean seasonal precipitation. The average rainfall depth over the entire drainage area was slightly over 7-inches (table 1-2).

The temporal distributions of rainfall in the 1957 report were based on mass rainfall curves for 6-hr intervals. Because the drainage area and subareas in this study were considerably smaller, response to excess rainfall would be masked by such long time intervals. Thus 1-hr time increments were selected to adequately define the unit graphs for the subareas. Use of these mass rainfall curves to define 1-hr rainfall patterns resulted in "average" values based on 6-hr totals. The intense cells or periods of rainfall could not be inferred from these average values, thus runoff was very moderate. To arrive at sufficient description of 1-hr rainfall intensities two approaches were considered:

(1) use of regression formulae to compute 1-hr, 2-hr, 3-hr, etc. depths based on given 6-hr depths re: NOAA ATLAS 2, "Precipitation-Frequency Atlas of the Western United States", Volume VIII - Arizona,
or

(2) use of existing recorded data for use as rainfall distribution pattern.

Since results of two recording rainfall gages (Ashland and Childs, Arizona) were already tabulated in 1-hr increments for the Feb-March 1938 storm, and since the project storm was a hybird of this storm, it was reasonable to use them as storm pattern indicators. Moreover, both gages were near the drainage area in this study, therefore this recorded gage information was used to represent the 1-hr rainfall pattern. Examination of the Ashland pattern and ensuing runoff hydrographs indicated it was too severe to be used as a general drainage area pattern. The Childs' gage rainfall distribution was more moderate, produced more reasonable intensities, and was generally similar to the March 1978 time distribution, therefore it was applied to the 1476 sq. mi. drainage area (table 1-3).

Snowfall-snowmelt was not considered important due to the general altitude and temperatures of the drainage area in this study.

b. Loss Rates-The HEC-1 Loss rate Function in LADFHP was used with STRKR= 0.30. DLTKR= 1.00 and RTIOL= 2.00 (reference 14). Per-cent impervious cover (PIM) varied from 5 to 20 percent depending on mountainous terrain and degree of urbanization.

c. Unit Hydrograph-LADFHP was used to determine synthetic unit hydrographs for the subareas. Lag values were computed based on $\bar{n}$ values from reference 2 and the Phoenix Valley and Phoenix Mountain S-graphs as appropriate. The other measurable basin characteristics, e.g. slope, and stream length were used as in the main report except for subareas 9 and 10 (10 was subdivided into 10, 11, and 12) which had to be delineated again (table 1-1). In the case of the latter, the characteristics were taken from available topography.

d. Percolation Losses - Percolation losses were not considered to be critical for general winter storms. The Salt River above the Verde River is ordinarily a series of continuous lakes; a flood wave passing through the lower Salt River reservoirs would obviously not percolate in this reach. Similarly, the Verde River below Horseshoe Dam is comprised of a lake upstream of Bartlett Dam and controlled releases from Horseshoe Dam; the Verde River below Bartlett is also perennial to the Salt River; therefore, little percolation is likely. Below the confluence site percolation may occur especially downstream of Granite Reef Dam. However, losses at this point have little effect on large volume hydrograph peaks. Moreover, since the project storm was the second in a series of general winter storms, it can be argued that moisture requirements were very low - to the extent of saturated conditions - thus resulting in minimal percolation losses. The main thrust of this local flow study was determination of peak discharges, not volumes.

e. Routing - The presence of the four reservoirs below Roosevelt and Horseshoe Dams precludes exact knowledge of flood routing because operational proceedures by SRP greatly affect the flood wave. In this study SRP reservoirs were assumed to be nearly full at the beginning of the project storm (30,000 ac-ft of space was allowed in the Salt River hydropower dams) and inflow was routed through the dams using modified Puls Routing and storage - outflow information supplied by SRP. Outflow from Bartlett Dam was lagged 7-hrs through the reach from the Verde River to the Salt River based on estimated travel times. Attenuation of Bartlett Dam outflow was not felt necessary in this reach because of wet conditions and relatively full channel due to lateral inflow and Sycamore Creek inflow. The combined discharge below the Salt-Verde confluence was routed through the Salt River using modified Puls. Storage-outflow relationships were identical to those used in the main report, with the reaches selected to approximate 1-hr travel times (because the hydrographs were computed in 1-hr intervals) and provide peak dishcarges at the concentration points. (See Plate 1-3)

f. Results- The preceeding analysis resulted in SPF peak discharges of 110,000 cfs at the confluence of the Salt-Verde, 90,000 cfs below Indian Bend Wash, and 71,000 cfs at the Gila River. Subarea components and combined discharges at select concentration points are included in table 1-4.

4. DISCHARGE FREQUENCY ANALYSIS.

a. General Procedure.

To provide discharge frequency information along the Salt River with no contribution from the area above Roosevelt and Horseshoe Dams the following steps were taken:

(1) determination of volume frequency relationships for the Verde and Salt Rivers above the confluence site and Indian Bend Wash above the Salt River (at Tempe Bridge).

(2) analysis of coincident discharges between the Verde and Salt Rivers, and the Verde River and Indian Bend Wash, to determine whether any predictable relationship exists.

(3) derivation of volume frequency curves for the Salt River below the confluence with the Verde River based on results of (1) and (2) above.

(4) derivation of coincident Indian Bend volume-frequency curve for the combined Verde and Salt Rivers discharges.

(5) production of balanced hydrographs for the Salt River below the Verde River using the March 1978 flood as a pattern hydrograph, and volume-frequency curves from (3) to develop 500-yr, 200-yr, 100-yr, 50-yr, 20-yr, and 10-yr hydrographs.

(6) storage routing these hydrographs through the Salt River to the Tempe Bridge, the routed peak from each frequency hydrograph representing the n-year peak at the concentration point in question.

(7) production of balanced hydrographs for Indian Bend Wash discharge coincident with the Verde River discharge, using the Indian Bend Wash SPF component hydrograph from the Salt River SPF as the pattern hydrograph; return periods selected were the same as in (5).

(8) Storage routing the combined Indian Bend Wash-Salt river frequency hydrographs through the City of Phoenix to the Gila River, with the routed peak from each frequency hydrograph representing the n-year peak at the concentration points; no local inflow was considered to affect the peaks based on results of SPF runs and historical record. Also, since precolation losses were ignored, it was felt that the exclusion of local inflow would result in gains balancing percolation losses.

b. Volume Frequency, Verde River at Scottsdale.

The basic objective in the discharge frequency analysis was the estimation of larger magnitude events with less concern for events near the mean. Derivation of discharge frequency curves for the concentration points along the Salt River would require discharge frequency information at a location on the Salt River below the confluence with the Verde River. However, no streamgages are present below the confluence. Also, the closest existing gages, Verde River at Scottsdale and Salt River below Stewart Mountain Dam, do not directly display the information required for this study, since streamflow records include releases from Bartlett and Stewart Mountain Dams. Because of these complications, it was decided that the study could be accomplished by establishing volume frequency curves for the Salt River below its confluence with the Verde River, preparing frequency hydrographs from this data, and finally, routing the frequency hydrographs to the Gila River, after combining them with Indian Bend Wash discharge.

To determine the volume frequency relationship for the confluence site, volume frequency relationships for the Verde and Salt Rivers upstream of the site were established.

The Verde River volume frequency curves were derived by interpretation of systematic records from the Bartlett and Scottsdale gages. Annual peak recorded discharges on the Verde River at Scottsdale rarely reflect flow from the intervening drainage area below Horseshoe Dam, but rather releases from Bartlett Dam. However, during some of the years of record the peak at Scottsdale was significantly higher than the peak release from Bartlett. In these cases a peak discharge from the intervening drainage area was inferred by taking the difference in peaks at Scottsdale and Bartlett releases and adjusting them for losses in travel. During these peak events, one-day mean flows were similary computed, and a peak discharge vs. 1-day discharge relationship was established. Then, records at Bartlett Dam were re-examined to locate events for each water year when runoff occurred below Horseshoe Dam. These 1-day discharges were compared to provide annual 1-day maxima of runoff below Bartlett Dam. Annual peaks were computed using peak vs. 1-day ratios on the Verde River, peak vs. 1-day discharge on Sycamore Creek, and peak discharge on Sycamore Creek vs. peak discharge on Verde R. at Scottsdale as guides. Two-day and 3-day duration volumes were determined in the same fashion.

During the time this study was being done several large floods occurred on the Salt and Verde river system. A provisional hydrograph for the Salt River at Granite Reef dam for the flood of March 1978 was compiled by the USGS. However, gages in the lower Verde watershed for the Verde River at Scottsdale and Sycamore Creek near Ft. McDowell were washed out (March 1978). To arrive at a discharge on the Verde River at Scottsdale for the March 1978 flood, the hydrograph of Bartlett spills and Stewart Mountain outflows were compared to the hydrograph for the Salt River at Granite Reek Dam. Volumes for 1-day, 2-day, and 3-day flows were computed and lagged appropriately and the differences in volumes were considered to be the result of Verde River runoff below Bartlett Dam drainage area. Following these determinations, the peak discharge was computed through use of the peak vs. 1-day ratio, along with other peak vs. duration information (2-day, 3-day).

The result was verified using March 1978 values for peak discharge vs. drainage area relationships determined for sites on the Verde River and Salt River for the same flood; additional corraboration was provided by combining local peak inflows to Bartlett Dam, local peaks below Bartlett, peak Sycamore Creek inflow and side inflow to the Verde and estimating the resulting peak discharge.

Finally, since the intermediate drainage area between Bartlett and Hoseshoe Dams was not accounted for in determining volume information for years prior to 1978, these volumes were adjusted upwards based on March 1978 results to account for the additional volume from subarea 3 (DA = 195 sq. mi.).

Then, annual peaks and volumes were plotted on frequency paper using Beard's (median) plotting positions, with the 1978 results considered to be the largest event since 1916 after examination of historical meteorologic and hydrologic data. Graphical curves were then fit to the data (plate 1-4).

c. Volume Frequency, Salt River below Stewart Mountain Dam.

A similar analysis was made for the Salt River, but complicated by storage and releases from the three reservoirs below Roosevelt Dam (Horse Mesa, Mormon Flat, and Stewart Mountain). Daily storages, inflows to Roosevelt from Tonto Creek and the Salt River, and releases from Stewart Mountain were examined. Any evidence of abrupt system changes were indicators of runoff events. After preliminary screening, data from SRP for hourly operation on dates suggested by screening was requested. This information, when available, was then used to compute hourly runoff values for the Salt River below Roosevelt Dam. When only daily mean runoff data was available, the Verde River peak vs. 1-day ratios were used to adjust the daily flows (the few actual peak vs. 1-day Salt River flows fit the Verde River data well). Complete hourly operational information for the March and December 1978 floods was furnished by SRP, and similar to the Verde River analysis, the March 1978 event was concluded to be the largest since 1916. The annual maxima were then plotted using Beard's plotting positions; the analysis differed somewhat from the Verde analysis in that record since 1938 was used, but annual maxima values were not obvious from recorded data for the entire period of record. The years when annual maxima were unavailable were presumed to contain non-zero flows of less quantity than the years when maxima were available. Since the primary interest was in events considerably greater than annual means, this was not considered to be misleading. Available events were ranked and plotted according to the number of possible years of record and the unknown, non-zero annual maxima were ignored. Finally, graphical curves were fit to the data (Plate 1-5).

d. Coincident Analysis, Verde River and Salt River.

To derive volume frequency curves for the ungaged confluence of the Verde River and Salt River, the volume frequency curves for the Verde and Salt Rivers established in 4-b and 4-c (plates 1-4, 1-5) had to be combined in some way. The concept employed to achieve this goal is summarized thus:

Since the Verde River drainage area represents the focus or geographical center of the Salt River basin being studied, and similarly, since a storm causing significant runoff over the Verde River basin would likely be large enough in areal distribution and magnitude of precipitation to produce runoff in Indian Bend Wash and/or the Salt River, it was decided to use the Verde River runoff analysis as a pivot for determination of any coincident flow from the Salt River as well as Indian Bend Wash. Use in a pivotal concept of either Indian Bend Wash, a much smaller drainage area which is characterized by runoff from smaller local storms, or the Salt River, whose headwaters and major rainfall-runoff producing terrain is far removed from Indian Bend Wash, would not produce coincident flow of any predictable magnitudes or probability. In addition, because only the larger events (more rare) were felt to be critical to this analysis, and since it seemed obvious that the larger the runoff event, the greater in depth and areal extent the storm that spawned it would be, (especially because snowmelt is not a consideration) use of the central area to hinge the analysis is more viable.

Actual mechanization of the central pivot concept involved determination of volume frequency curves for the Verde River and the Salt River (plates 1-4, and 1-5) and coincident discharge curves for Verde River and Salt River (plate 1-6). The latter were based upon the Salt River discharge coincident with the Verde River annual peak; this analysis was performed in a one-sided manner for several basic reasons:

(1) the previously explained central - pivot concept.

(2) the difficulty in determining peaks on the Salt, as previously mentioned.

(3) the desire for an annual series rather than partial duration series.

For this analysis it was hypothesized that for smaller runoff events, i.e. those near the mean, the storm spawning the runoff would be small in areal extent and not produce any significant runoff from adjacent drainage areas. However, a large runoff event, e.g. a 100-yr. flood, would likely result from a large storm which would also produce runoff on adjacent drainages. Therefore, it was concluded that the larger the storm (ergo, the runoff), the greater the probability of coincident runoff (plate 1-7). The assumption was made that the correlation for extreme events approached 1.0, while for small events it was 0.0 by this same logic. Historical information confirms this conclusion in a qualitative sense. The problem was quantifying a relationship for the probability of an event occurring on the Salt River given the occurrence of an event on the Verde River, i.e. Pr (E_2/E_1).

This difficulty was overcome through the use of two extreme analyses- complete independence and complete dependence (plate 1-7), i.e. correlation = 0.0 and correlation= 1.0. For the latter case another curve indicating discharge on the Salt River as a function of discharge on the Verde River was developed (plate 1-6). Afterward, a coincident probability curve was developed for each extreme by the following approaches:

(a) complete dependence (more rare events) - A probability of an event on the Verde River being equalled or exceeded was selected, e.g. Pr $(E \geq E_1)=.01$; the discharge, E_1, associated with this probability was then taken from the discharge frequency curve (plate 1-4). Using this discharge, the dependent Salt River discharge was estimated (table 1-5a and plate 1-6), E_2. Since the discharge from the Verde River, E_1, is the 100-yr discharge, and there is complete correlation (assumed) between E_1 and E_2, the Salt River coincident discharge, then the 100-yr discharge on the Salt River below the Verde River is E_1+E_2. This same approach was reiterated for other values of Pr, and E, the results were plotted on frequency paper, and a smooth curve was drawn thru the data (plate 1-8).

(b) complete independence (frequent events)- For this approach, considered as the lower limit for discharge frequency, it was assumed that an event on the Verde would occur with <u>no</u> coincident Salt River flow, or an event on the Salt River would occur with <u>no</u> coincident Verde River flow. This situation could be described in the following manner: an observer located just downstream from the confluence of the Verde and Salt Rivers might measure an event, E, on the Salt River which emanated entirely from the Verde river. The flow would have a return period of n_1 times per 100-yrs on the Verde River. Similarly, the observer might measure the same event, E, which was the result entirely of Salt River discharge, and would have a return period of n_2 times per 100-yrs. Thus the frequency of this event, E, occurring on the Salt River below the confluence with the Verde River would be the sum of n_1+n_2 times per 100-yrs (table 1-6a). This would be a minimum value since it is obvious that this event, E, could result from combinations of flow from both the Verde and Salt Rivers which would increase the probability of E being equalled or exceeded.

Thus, a curve representing completely independent (non-coincident) flow was derived by selecting an event, E, as described above, and using the discharge frequency curves for the Verde River and Salt River (Plates 1-4, 1-5) to determine the frequency of occurrence, n_1 and n_2 respectively, for a given time period, e.g. 100-yrs. The combined discharge frequency curve for the Salt River below the confluence with the Verde River is determined by plotting the discharge, E, versus the sum of the frequency of occurrence, $n_1 + n_2$; a series of discharges were examined, probabilities determined, discharges plotted, and a smooth curve drawn through them (plate 1-8).

It may be observed that this alternate approach, complete independence or non-coincidence, amounts to mutual exclusivity. The analysis, however, is reasonable from the rainfall-runoff approach described previously. That is, for small events (discharge near the mean annual discharge), the storm resulting in such flow is likely to be small in areal extent, therefore runoff from a single basin causing the peak becomes more likely. Also, it should be noted that this approach (non-coincident discharge) was used to establish the curve for more frequent events (near the mean).

The composite discharge frequency curve for the combined Salt-Verde Rivers was determined by use of the extreme analysis results (plate 1-8). The two curves were used to represent upper and lower limits of discharge frequency and a transition zone between the two limits was assumed. The discussion of the reasons for the upper and lower limits has been presented previously; the exact transition is somewhat arbitrary, but not particularly sensitive since upper limit values were of prime concern in this report.

Other durations were computed by a similar analysis, but rather than use coincident curves for durations other than peaks (which curves proved too ambiguous), the Verde River volume frequency curves were used along with the duration flows expected for the frequency peak on the Salt River. The results are shown on plates 1-9 and 1-10.

e. Frequency Hydrographs, Salt River below the Verde River to the Tempe Bridge.

The volume frequency curves (plate 1-9) were used along with the March 1978 flood hydrograph of inflow to Stewart Mountain Dam (as computed from storage-release information) as the pattern hydrograph to determine n-year balanced hydrographs. This event was selected because it was 1) historical, thus actually had occurred, 2) documented, and 3) the most severe flood known to have occured since 1916, and thus representing the shape of a severe hydrograph. The balanced hydrograph computer program was used to determine 500-, 200-, 100-, 50-, 20-, and 10-yr frequency hydrographs. These hydrographs were then routed successively through 3 reaches using Modified Puls routing. The reach lengths were coincident with CP-91, CP-92, and CP-93, and were determined such that the travel time equalled 1-hr, the hydrograph interval; storage-outflow relations were those documented for the main report. Routed peaks were assumed to represent the n-year frequency discharge associated with each frequency hydrograph. Results are shown on plate 1-11, 1-12, and 1-13. As previously stated, no percolation losses nor local inflows were used.

f. Coincident Analysis-Verde River and Indian Bend Wash.

The same approach was attempted to correlate discharge on the Verde River with Indian Bend Wash discharge. However, since the Verde River flow was now combined with Salt River flow, a modification was necessary.

To begin with, the discharge frequency curve for Indian Bend Wash was taken from tabulated values in the Phase 2 GDM for Indian Bend Wash for CP-1101, Indian Bend Wash near Tempe Bridge. The 10, 25, 50, and 100-yr discharges were plotted on frequency paper and a smooth curve fitted through them. To arrive at volumes, annual peak discharges and 1-day, 2-day, and 3-day simultaneous flows were tabulated. Relationships between these flows were established and these relationships were used to compute 1-day, 2-day, and 3-day volumes to coincide with the peak frequency discharges. The resulting volume frequency curve is shown on plate 1-14.

Following this, a coincident Indian Bend Wash and Verde River discharge curve was prepared from historical data, using the SPF results as a guideline (plate 1-15).

Next, n-year discharges on the Verde River were tabulated vs. the coincident discharge on Indian Bend Wash from plate 1-15. As in the Verde-Salt coincident flow analysis, flows of 1-day, 2-day, and 3-day durations associated with the peaks on Indian Bend Wash were also tabulated. However, combination of these n-year discharges was handled differently. The extremes of complete dependence and independence were again examined (tables 1-5b, 1-5c). Rather than use the Verde River as the pivot, the combined and routed n-year hydrographs on the Salt River above Indian Bend Wash, CP-93, were used. This is consistent with the analytical approach taken throughout; the switch in pivots was done for purposes of combining Indian Bend Wash discharges with Salt River discharges tables 1-5c, 1-6b). Moreover, while the Salt River discharge, and discharge frequency curves were used for combination purposes, the Salt River n-year hydrographs were determined by use of the Verde River as central pivot, therefore, the Verde River was still implicitly the focus for the analysis. Results of the Salt River-Indian Bend Volume Frequency analyisis and development are shown in plates 1-16 through 1-19.

g. Frequency Hydrographs, Salt River Below the Tempe Bridge to the confluence with the Gila River.

The volume frequency curves for the Salt River below the Tempe Bridge (plate 1-18) were used to determine n-year peak discharges, and 1-day, 2-day, and 3-day flows. Pattern hydrographs based on the routed n-year hydrographs of the Salt River at CP-93 (above the Tempe Bridge) were used to determine the shape of the n-year hydrographs of the Salt River below the Tempe Bridge through use of the Balanced Hydrograph program. The resulting 500-yr, 200-yr, 100-yr, 50-yr, 20-yr, and 10-yr hydrographs, were then routed through the City of Phoenix in three reaches using Modified Puls routing. As in reaches 91-93, lengths were determined by travel time, and storage-outflow relations from the main report. Again, routed peaks were assumed to represent the n-year frequency hydrograph (plates 1-20 through 1-22). Also, as has been mentioned previously; no percolation nor local inflow was assumed.

TABLE 1-1

Subarea Unit Graph Characteristics
Salt River Basin

Subarea No.	D.A (sq.mi.)	L (mi.)	L_{ca} (mi.)	Slope (ft/mi.)	Bn Value	S-Graph
7	105	12.7	1.6	237	.047	Phoenix Mountain
8	160	20.0	10.5	177	.047	Phoenix Mountain
6	126	15.7	6.7	260	.047	Phoenix Mountain
5	59	6.3	2.0	111	.047	Phoenix Mountain
31	195	19.3	6.3	122	.047	Phoenix Mountain
42	211	40.0	20.0	110	.047	Phoenix Mountain
41	251	21.0	15.0	9.5	.047	Phoenix Mountain
9	190	18.0	9.0	12.5	.030	Phoenix Valley
10	48	7.9	3.9	5.6	.030	Phoenix Valley
11	82	7.5	5.1	5.6	.030	Phoenix Valley
12	49	5.5	3.6	5.6	.030	Phoenix Valley

TABLE 1-2

Subarea Standard Project Storm (SPS) Precipitation

Salt River Basin

Subarea No.	D.A. (sq.mi.)	Total Incident Rainfall (in)	Total Effective Rainfall (in)
7	105	10.50	3.69
8	160	8.40	2.22
6	126	7.52	1.76
5	59	6.90	1.36
31	195	8.13	2.07
42	211	9.18	2.73
41	251	7.50	1.76
9	190	5.08	1.09
10	48	3.75	0.96
11	82	3.75	0.96
12	49	3.75	0.96

TABLE 1-3

Subarea Rainfall Pattern
Incremental Hourly Depths in Percent of Total Rainfall

Period	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
	.04	.03	.03	.03	.03	.03	.02	.01	.00	.00
(11)	0.	0.	.00	0.	0.	0.	0.	0.	0.	0.
(21)	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
(31)	0.	0.	0.	0.	0.	0.	0.	0.	.00	0.
(41)	.00	.00	.00	.00	.00	.01	.01	.00	.01	.00
(51)	.00	0.	0.	0.	0.	0.	0.	0.	0.	0.
(61)	0.	0.	0.	0.	0.	0.	0.	0.	0.	.00
(71)	.00	.02	.03	.03	.03	.03	.03	.03	.02	.02
(81)	.02	.00	0.	0.	0.	0.	0.	0.	0.	0.
(91)	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
(101)	0.	.00	.03	.07	.04	.03	.02	.01	.01	.02
(111)	.09	.11	.02	0.	0.	0.	0.	0.	0.	0.

TABLE 1-4

Subarea SPF Component Discharges in cfs

LOCATION	CP	SUBAREA	SPF COMPONENT (cfs)	DISCHARGE (cfs)
Inflow to Horse Mesa		7	44,124	-
Inflow to Mormon Flat		8	31,364	
	3			29,900[a]
Inflow to Stewart Mtn		6	26,586	
	4			38,000[a]
Inflow below Stewart Mtn.		5	13,123	
Inflow to Bartlett		31	38,062	
Inflow to Verde below Bartlett		41	25,103	
Sycamore Creek		42	31,738	
	54			110,000[b]
	91			110,000[b]
	92			103,000[b]
Indian Bend Wash		9	20,460	
Below IBW	93 (d/s)			90,000[b]
		10	4,379	
	101			82,000[b]
		11	6,931	
Inflow from City of Phoenix (Subareas 10-12)	102			81,000[b]
		12	4,854	
	103			70,000[b]

a - result of combination after reservoir routing w/ 10,000 ac-ft space available at each salt River reservoir.

b - SPF values

TABLE 1-5a

COINCIDENT DISCHARGE FREQUENCY
DEPENDENT ANALYSIS

Salt River with Verde River

Pr	VERDE R. Q (cfs)	SALT R. Q (cfs)	Q (cfs)
.001	80,000	45,000	125,000
.002	66,000	43,000	109,000
.005	49,000	38,000	87,000
.01	38,000	35,000	73,000
.02	28,500	30,000	58,500
.05	18,000	23,000	41,000
.10	12,100	17,000	29,100
.20	7,200	9,300	16,500
.50	2,500	600	3,100
.70	1,300	-	1,300

TABLE 1-5b

COINCIDENT DISCHARGE FREQUENCY
DEPENDENT ANALYSIS

Pr	VERDE R. Q (cfs)	INDIAN BEND WASH Q (cfs)
.002	66,000	25,000
.005	49,000	12,000
.010	38,000	5,400
.020	28,500	2,350
.050	18,000	610
.100	12,100	190

TABLE 1-5c

COINCIDENT DISCHARGE FREQUENCY
DEPENDENT ANALYSIS

	SALT RIVER AT TEMPE BRIDGE				DEPENDENT ANALYSIS - INDIAN BEND WASH, ASSOCIATED Q's			
Pr	Q Peak (cfs)	Q 1-Day (cfs)	Q 2-Day (cfs)	Q 3-Day (cfs)	Q Peak (cfs)	Q 1-Day (cfs)	Q 2-Day (cfs)	Q 3-Day (cfs)
.002	59,733	33,786	21,936	16,857	25,000	6,800	4,000	2,600
.005	43,130	25,938	16,320	12,523	12,000	4,000	2,600	1,700
.010	33,955	20,458	12,690	9,655	5,400	1,000	700	460
.020	25,714	15,475	9,718	7,203	2,350	480	330	210
.050	16,882	10,182	6,316	4,600	610	110	70	46
.100	11,016	6,638	4,122	2,954	190	29	17	11

TABLE 1-5d

COINCIDENT DISCHARGE FREQUENCY
COMBINED RESULTS - DEPENDENT ANALYSIS

CP-93, SALT RIVER + INDIAN BEND WASH AT TEMPE BRIDGE

Pr	Q peak (cfs)	Q 1-Day (cfs)	Q 2-Day (cfs)	Q 3-Day (cfs)
.002	84,733	40,586	25,936	19,457
.005	55,130	29,938	18,920	14,223
.010	39,355	21,458	13,390	10,115
.020	28,064	15,955	10,048	7,413
.050	17,492	10,292	6,386	4,646
.100	11,206	6,667	4,139	2,965

TABLE 1-6a

COINCIDENT DISCHARGE FREQUENCY
INDEPENDENT ANALYSIS

Salt River with Verde River

Peak Q (cfs)	Exceedence Per 100 Yrs	Combined Exceedence Per 100 Yrs
60,000	.28 + .02	.30
49,000	.5 + .14	.69
38,000	1 + .50	1.5
28,500	2 + 1.6	3.6
18,000	5 + 5.8	10.8
12,100	10 + 12.5	22.5
7,200	20 + 27	47
2,500	50 + 49	99

TABLE 1-6b

COINCIDENT DISCHARGE FREQUENCY
COMBINED RESULTS - INDEPENDENT ANALYSIS

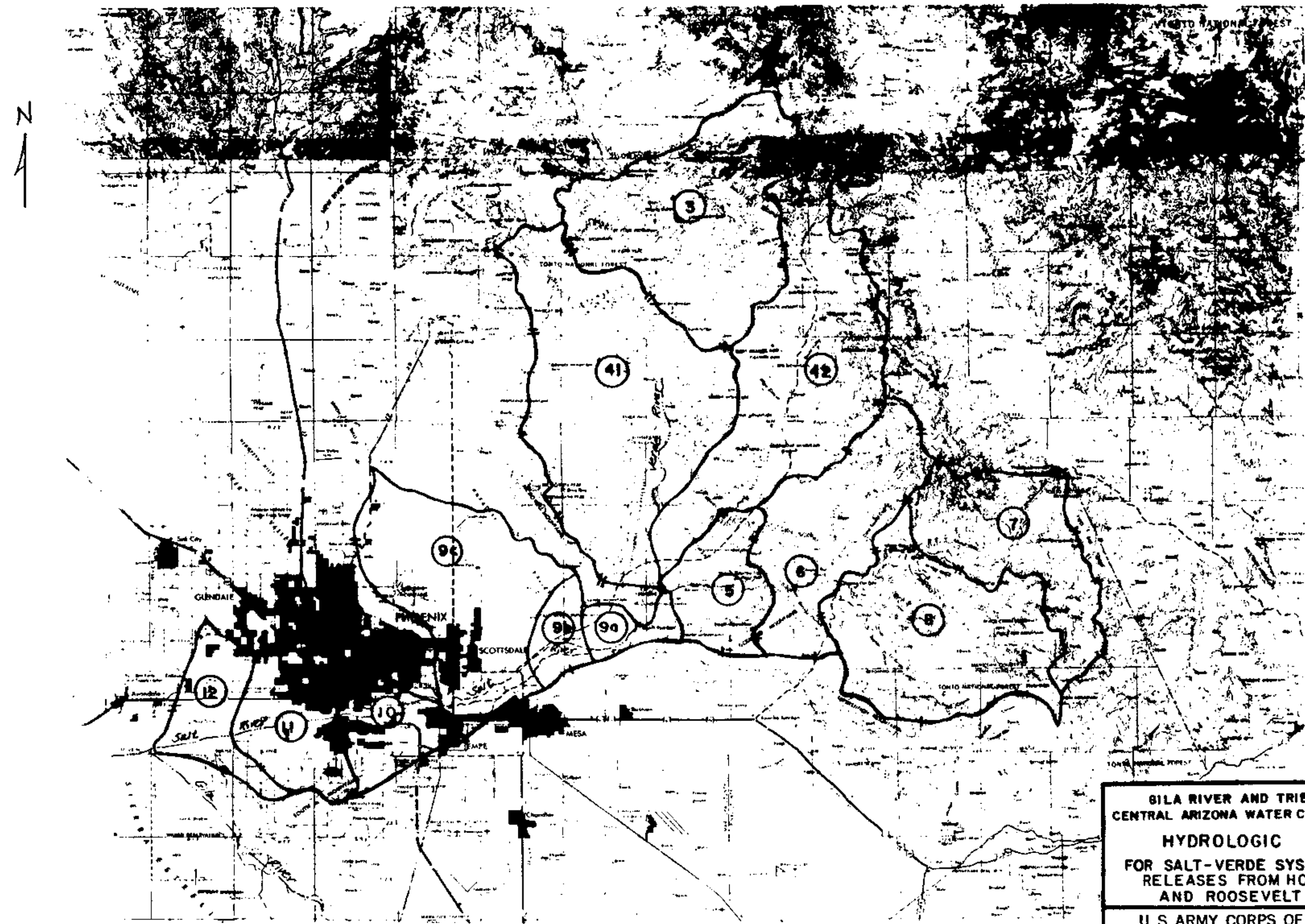
CP-93, SALT RIVER + INDIAN BEND WASH AT TEMPE BRIDGE

Q peak (cfs)	Combined Exceedences Per 100 Yrs	Q 1-Day (cfs)	Combined Exceedences Per 100 Yrs	Q 2-Day (cfs)	Combined Exceedences Per 100 Yrs	Q 3-Day (cfs)	Combined Exceedences Per 100 Yrs
73,000	0.1[a]+.17[b]=.27[c]	40,000	.1+.00=.10	26,000	.1+.00=.10	19,500	.1+.00=.10
60,000	0.2+.25=.45	34,000	.2+.00=.20	22,000	.2+.00=.20	16,500	.2+.00=.20
43,000	0.5+.49=.99	26,000	.5+.01=.51	16,500	.5+.00=.50	12,500	.5+.00=.50
34,000	1+.8=1.80	20,500	1+.027=1.03	13,000	1+.01=1.01	10,000	1+.00=1.00
25,500	2+1.36=3.36	15,500	2+.06=2.06	9,800	2+.03=2.03	7,400	2+.01=2.01
16,500	5+3.2=8.2	10,200	5+.18=5.18	6,200	5+.09=5.09	4,600	5+.07=5.07
11,200	10+6=16.0	6,600	10+.43=10.43	4,000	10+.28=10.28	2,950	10+.23=10.23
7,000	20+12=32.0	3,800	20+1.1=21.10	2,300	20+.78=20.78	1,600	20+.78=20.78
2,900	50+34=84.0	1,200	50+5.2=55.2	720	50+4=54.00	480	50+4.3=54.30

a - 1st exceedence in each case represents the frequency of the event, i.e., the referenced peak discharge, in the Salt River above CP-93.

b - 2nd exceedence in each case represents the frequency of the event, i.e., the referenced peak discharge, in Indian Bend Wash above CP-93.

c - 3rd exceedence in each case represents the independently combined discharge, in the Salt River below CP-93.

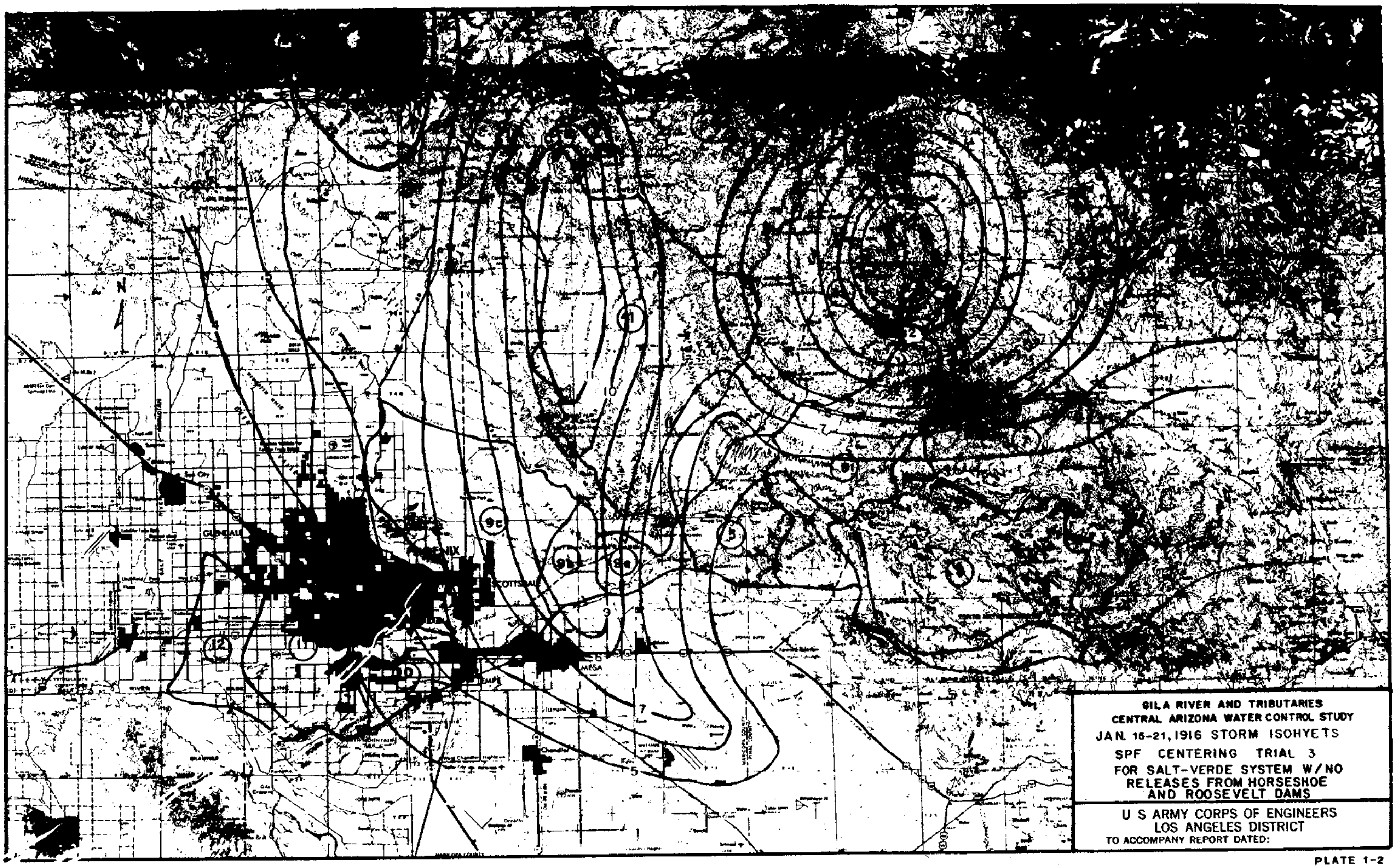


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

HYDROLOGIC AREA

FOR SALT-VERDE SYSTEM W/NO RELEASES FROM HORSESHOE AND ROOSEVELT DAMS

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



GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

JAN. 15-21, 1916 STORM ISOHYETS

SPF CENTERING TRIAL 3

FOR SALT-VERDE SYSTEM W/NO
RELEASES FROM HORSESHOE
AND ROOSEVELT DAMS

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

PLATE 1-2

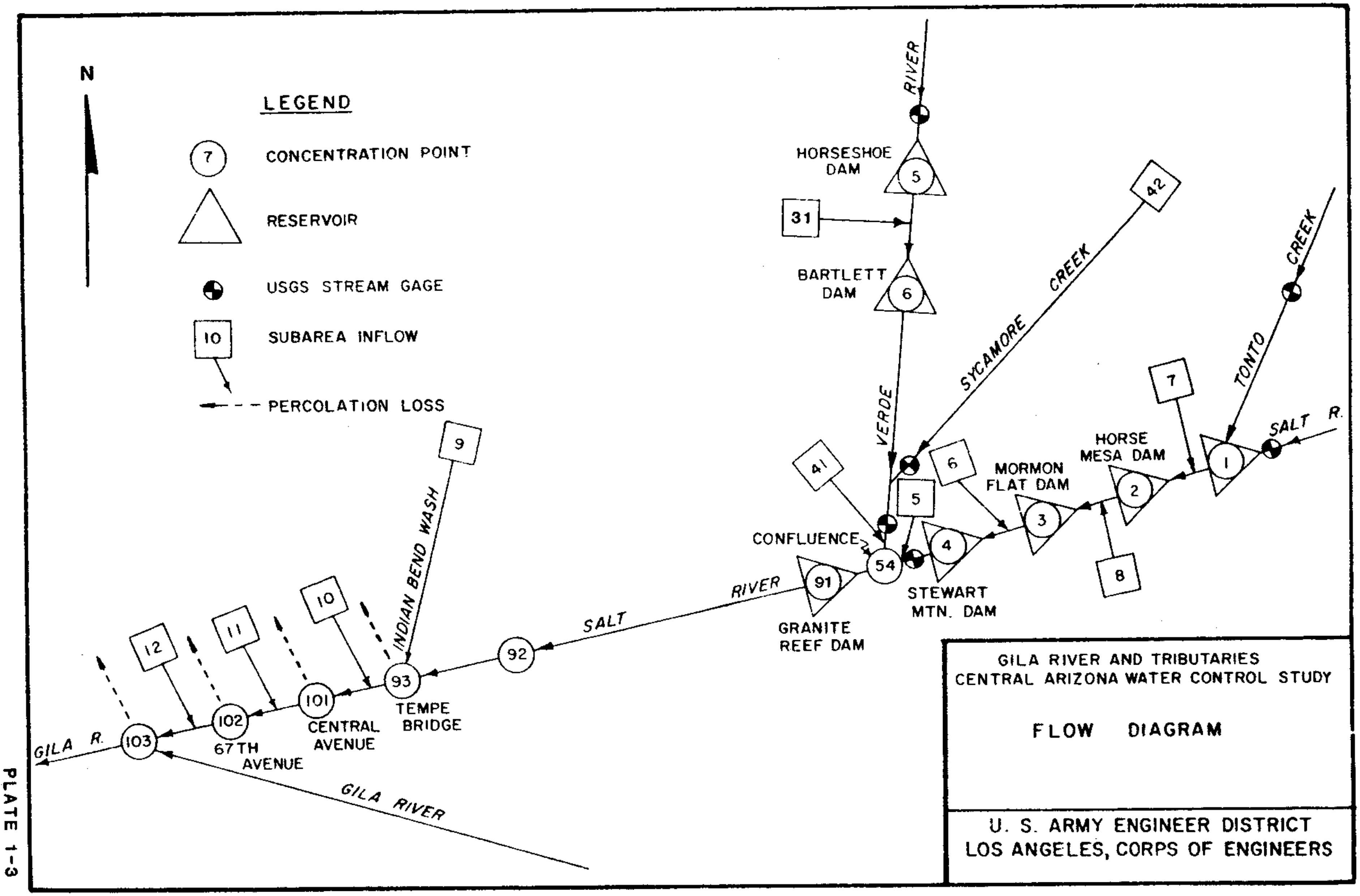
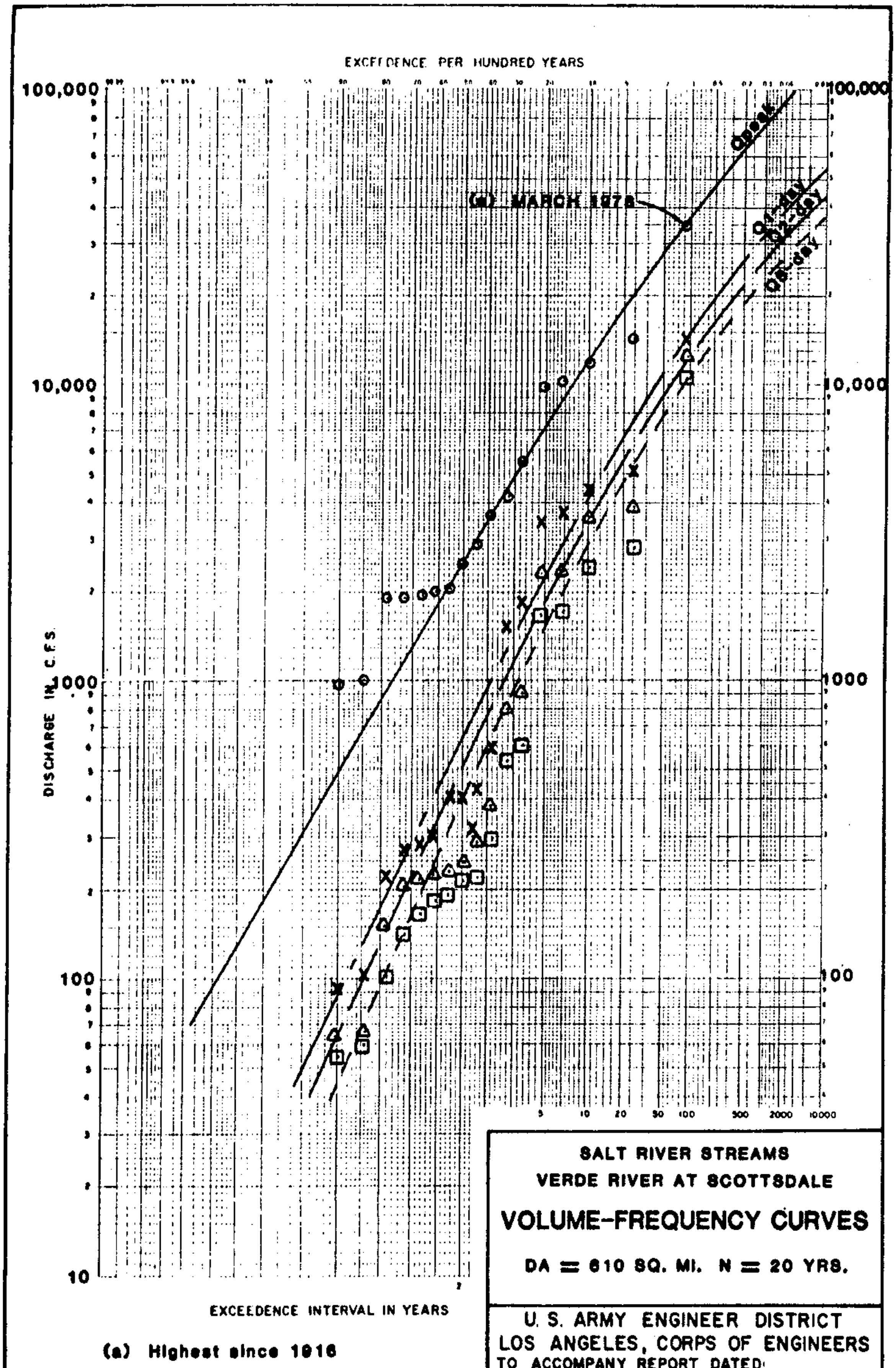

N
LEGEND
7 CONCENTRATION POINT
RESERVOIR
USGS STREAM GAGE
10 SUBAREA INFLOW
PERCOLATION LOSS
RIVER
HORSESHOE DAM
5
31
BARTLETT DAM
6
VERDE
SYCAMORE CREEK
42
TONTO CREEK
SALT R.
7
1
HORSE MESA DAM
2
MORMON FLAT DAM
3
8
6
5
41
4
STEWART MTN. DAM
CONFLUENCE
54
91
GRANITE REEF DAM
RIVER
SALT
92
9
INDIAN BEND WASH
93
TEMPE BRIDGE
10
101
CENTRAL AVENUE
11
102
67TH AVENUE
12
103
GILA R.
GILA RIVER
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY
FLOW DIAGRAM
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

PLATE 1-3



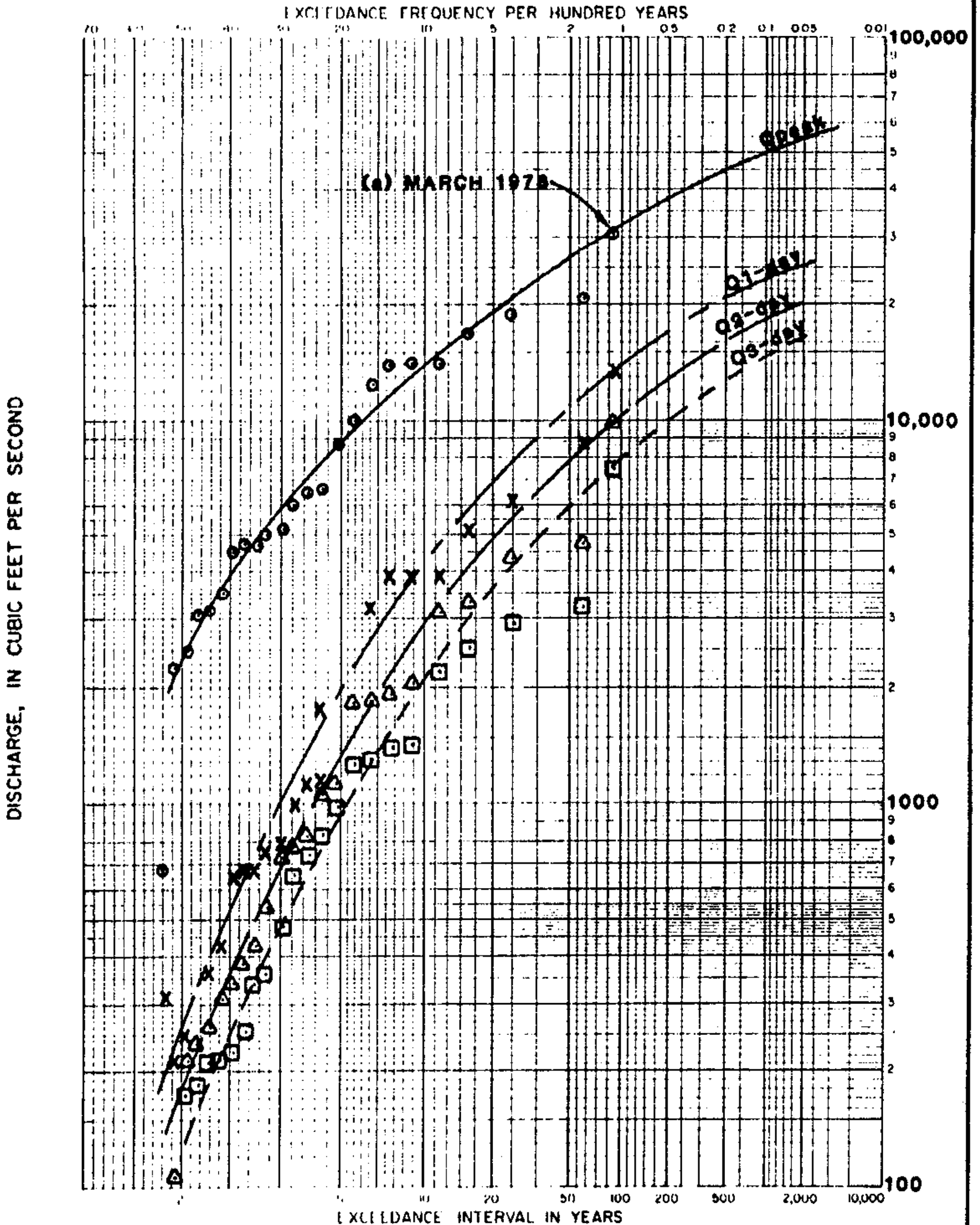
SALT RIVER STREAMS

VERDE RIVER AT SCOTTSDALE

VOLUME-FREQUENCY CURVES

DA = 610 SQ. MI. N = 20 YRS.

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



SALT RIVER STREAMS

SALT RIVER AT STEWART MTN.

VOLUME-FREQUENCY CURVES

DA = 391 SQ. MI. N = 42 YRS.

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

(a) Highest since 1916

PLATE 1-5

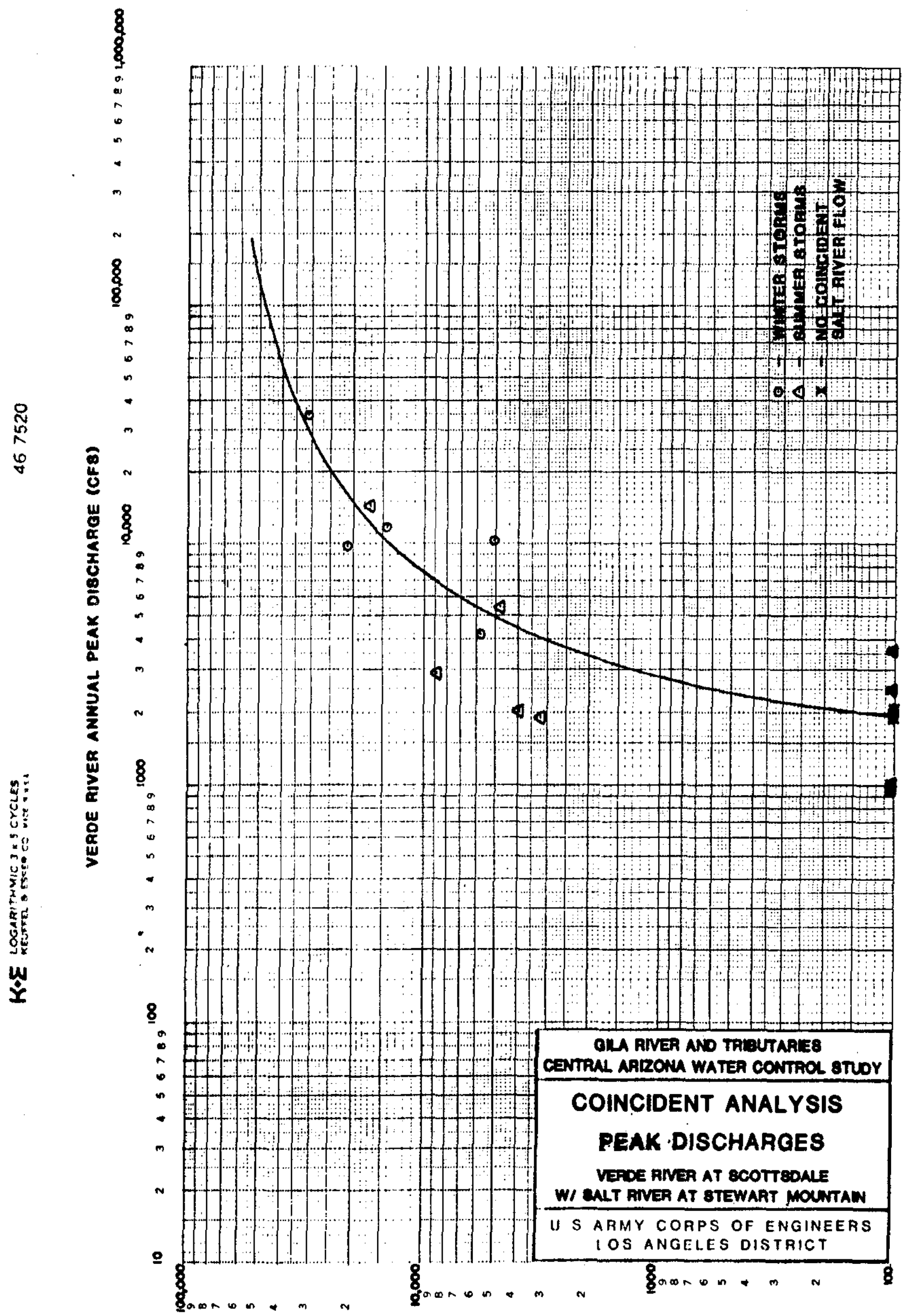
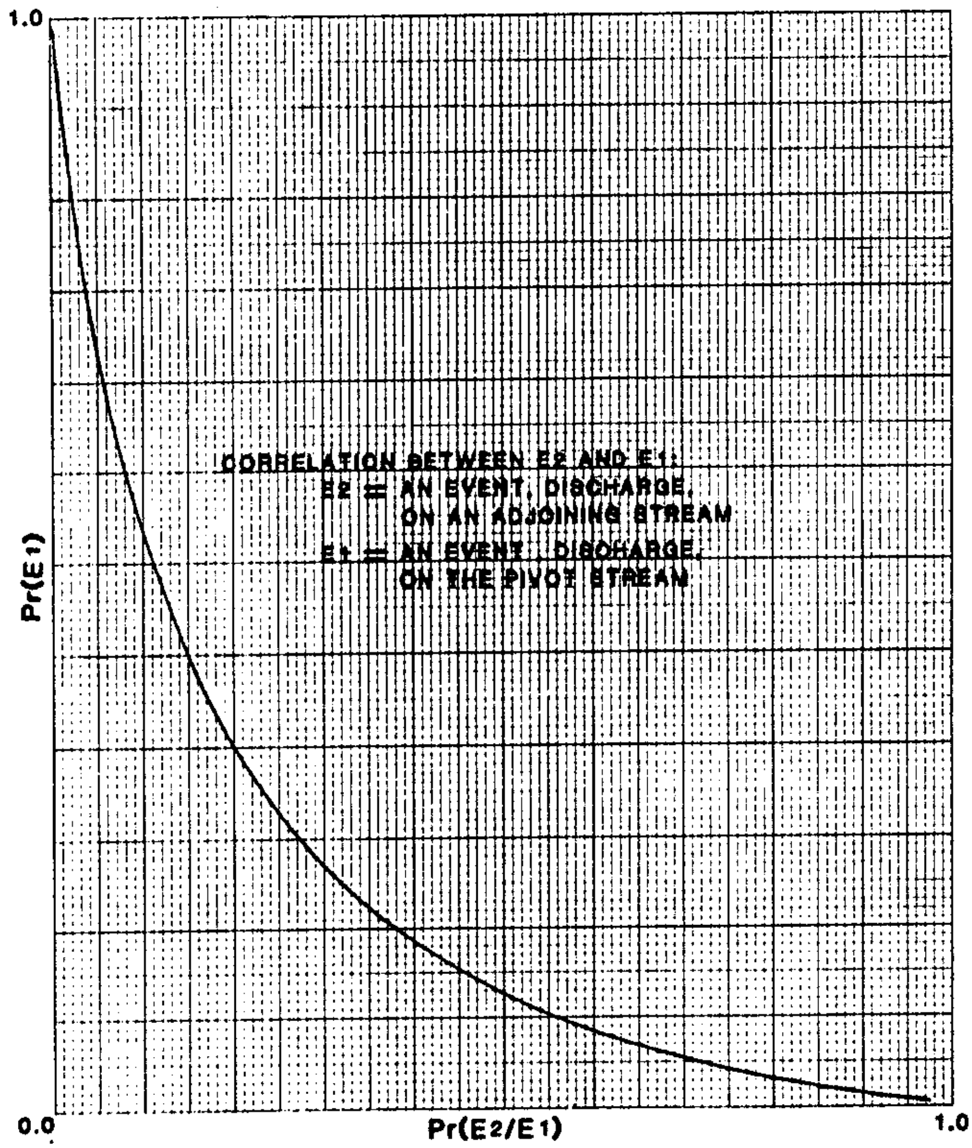

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY
COINCIDENT ANALYSIS
PEAK DISCHARGES
VERDE RIVER AT SCOTTSDALE
W/ SALT RIVER AT STEWART MOUNTAIN
U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
VERDE RIVER ANNUAL PEAK DISCHARGE (CFS)
SALT RIVER COINCIDENT PEAK DISCHARGE (CFS)
⊙ — WINTER STORMS
Δ — SUMMER STORMS
X — NO COINCIDENT SALT RIVER FLOW
K+E LOGARITHMIC 3 X 5 CYCLES
46 7520

PLATE 1-6

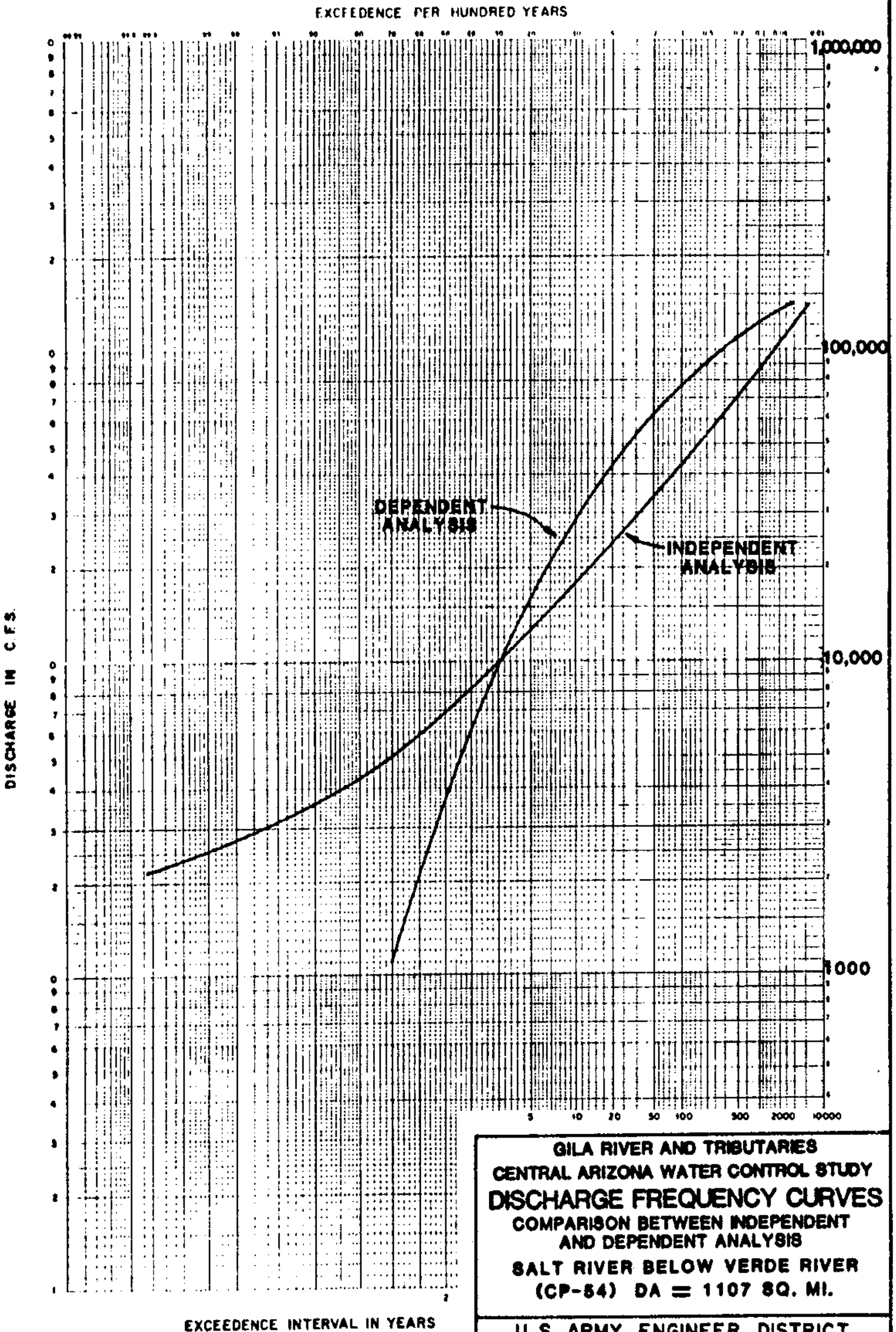


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

CORRELATION/PROBABILITY CURVE

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

PLATE 1-7



GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

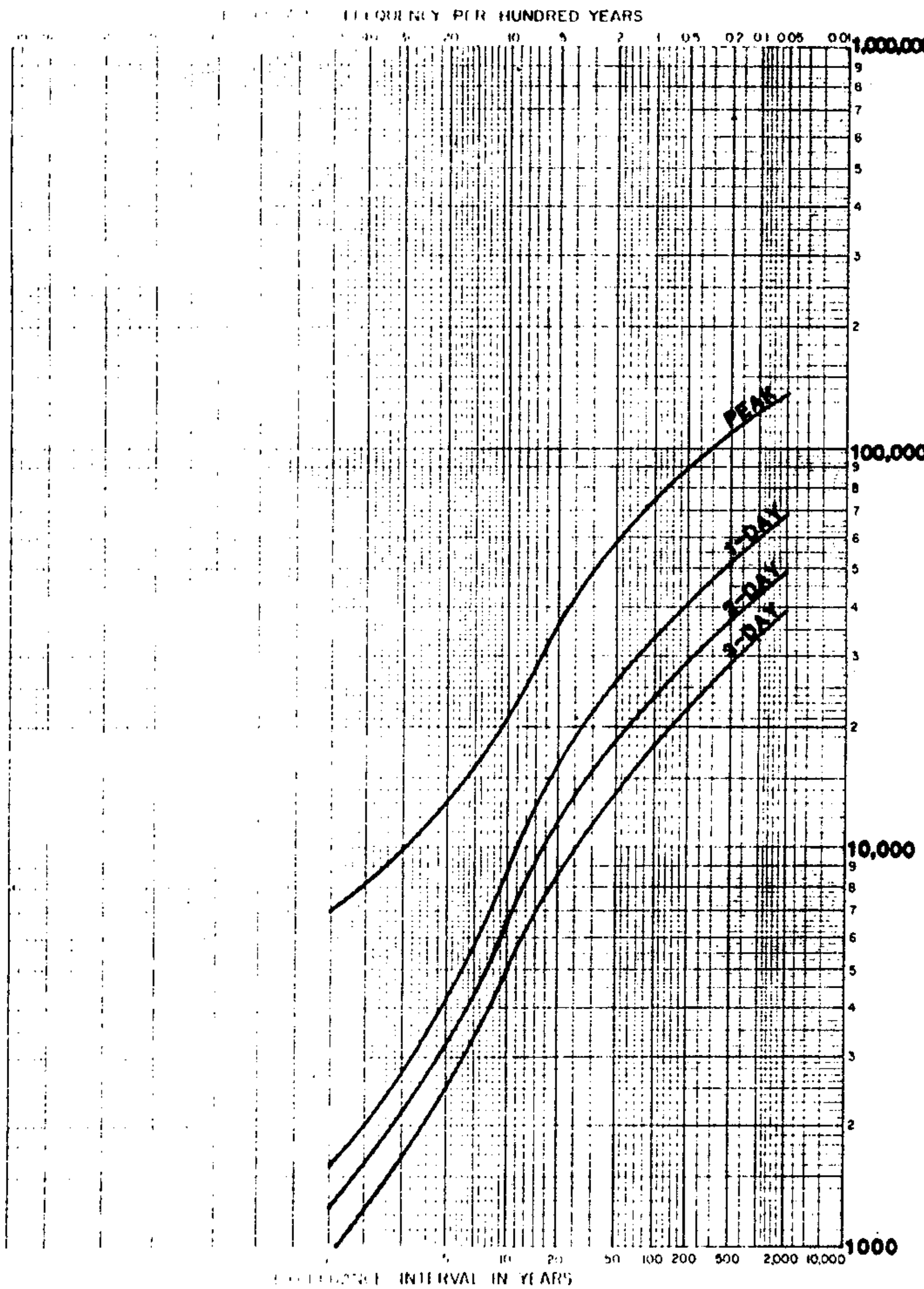
DISCHARGE FREQUENCY CURVES

COMPARISON BETWEEN INDEPENDENT AND DEPENDENT ANALYSIS

SALT RIVER BELOW VERDE RIVER
(CP-54) DA = 1107 SQ. MI.

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

PLATE 1-8



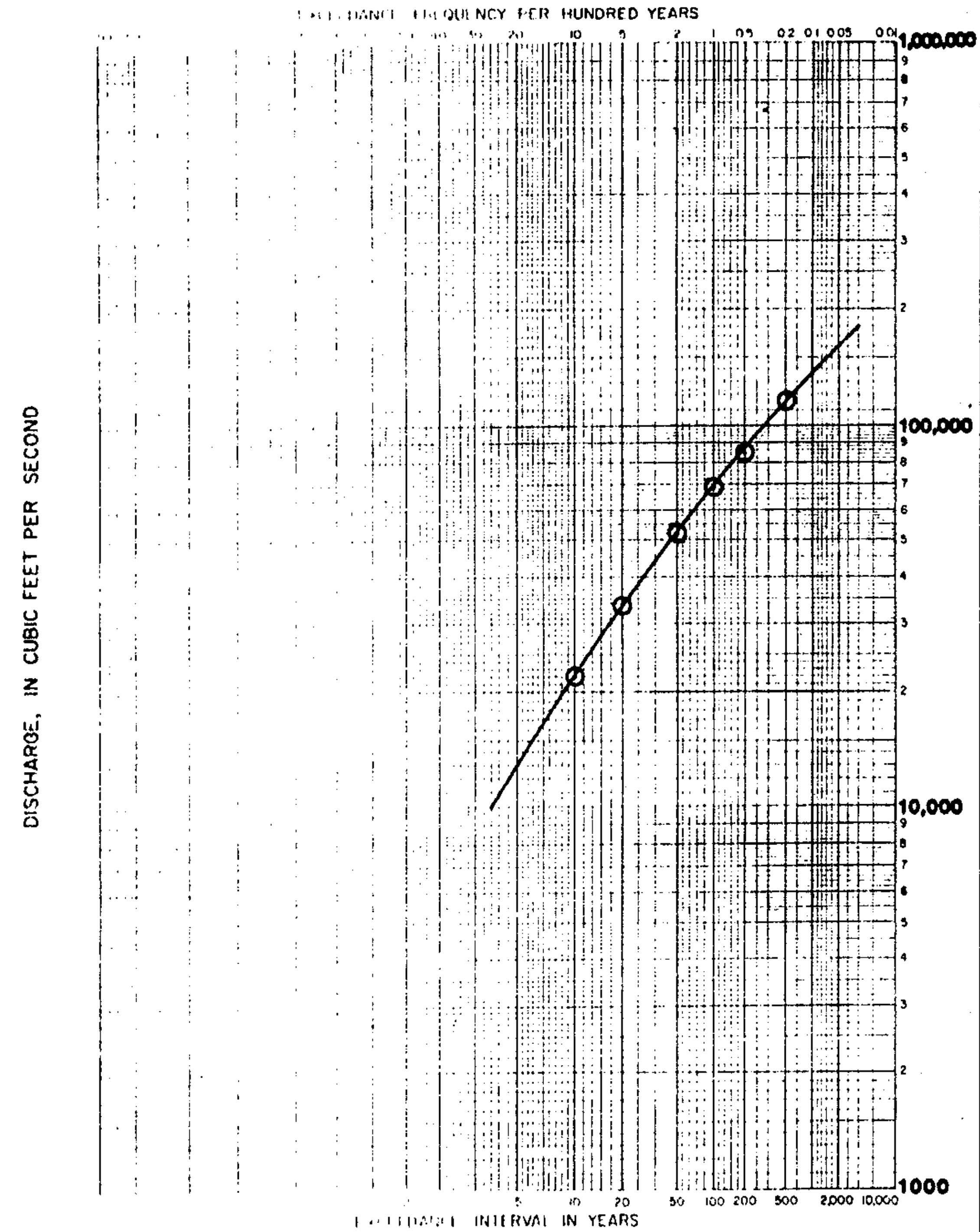
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY CURVES

SALT RIVER BELOW VERDE RIVER

(CP-64) DA = 1107 SQ. MI.

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



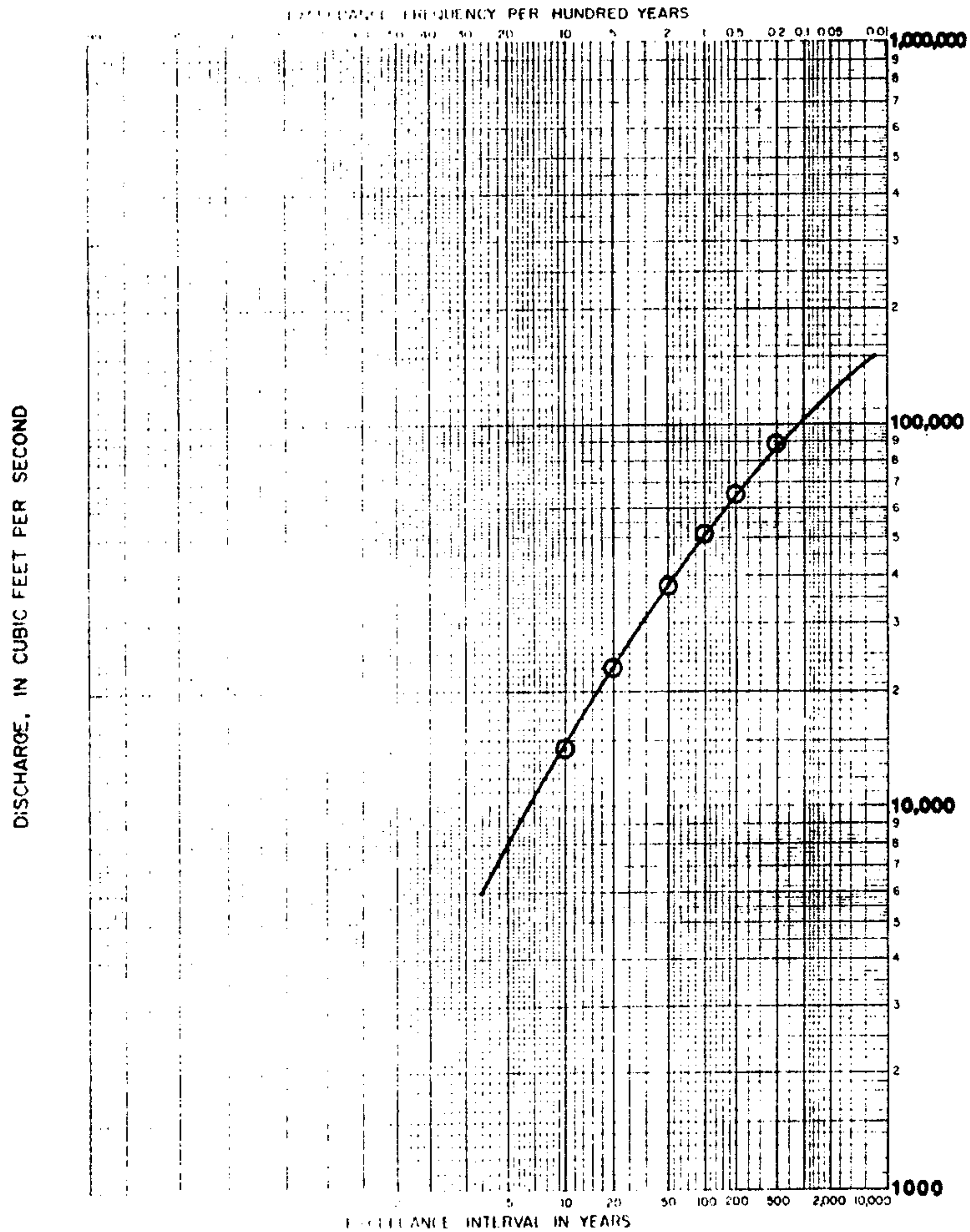
⊙ – Results of routing n-year frequency hydrographs

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE FREQUENCY CURVE
SALT RIVER BELOW VERDE RIVER
(CP-54) DA = 1107 SQ. MI.

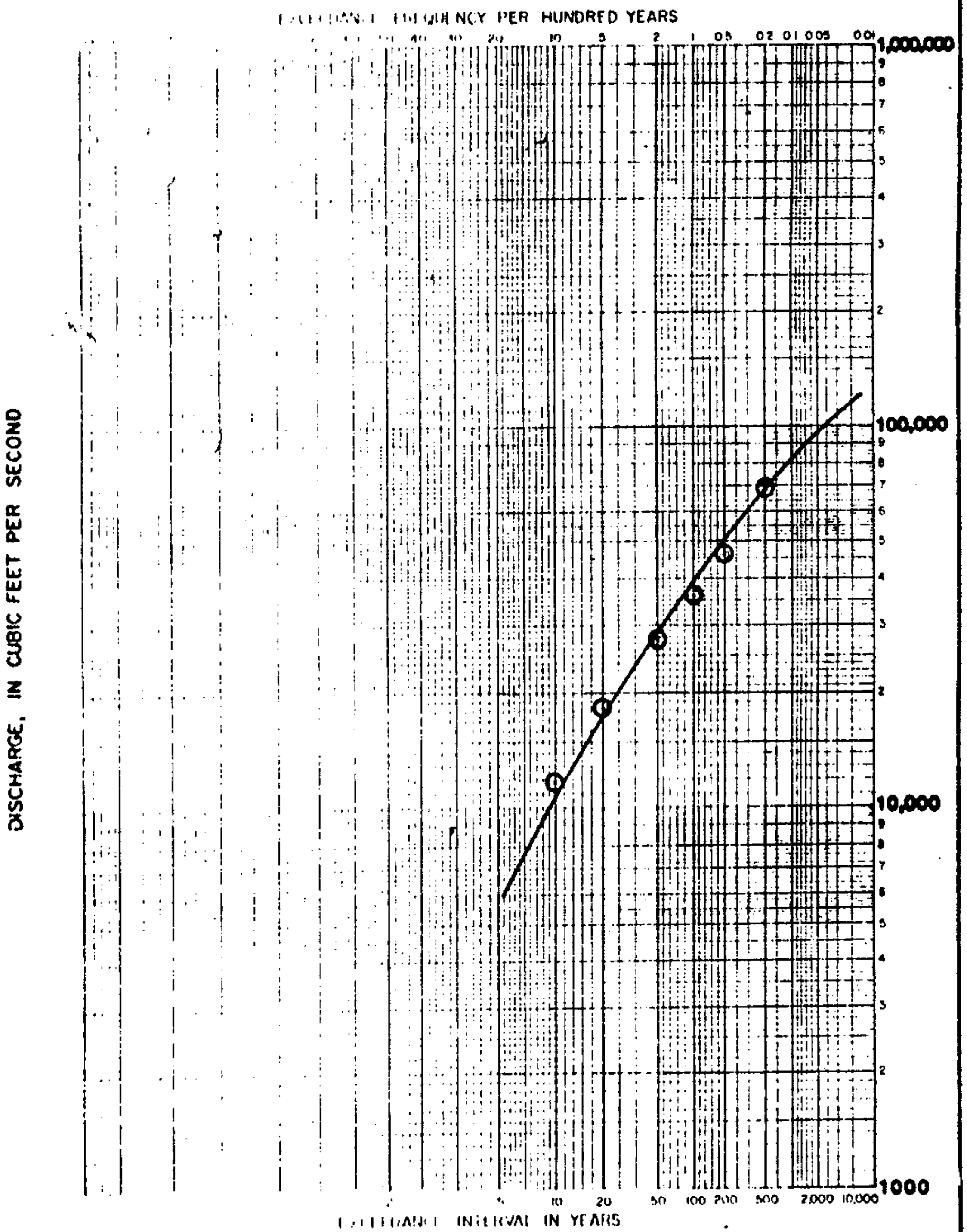
U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 1-10



⊙ - Results of routing n-year frequency hydrographs

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY
DISCHARGE FREQUENCY CURVE SALT RIVER NEAR GRANITE REEF DAM (CP-91)
U S ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT



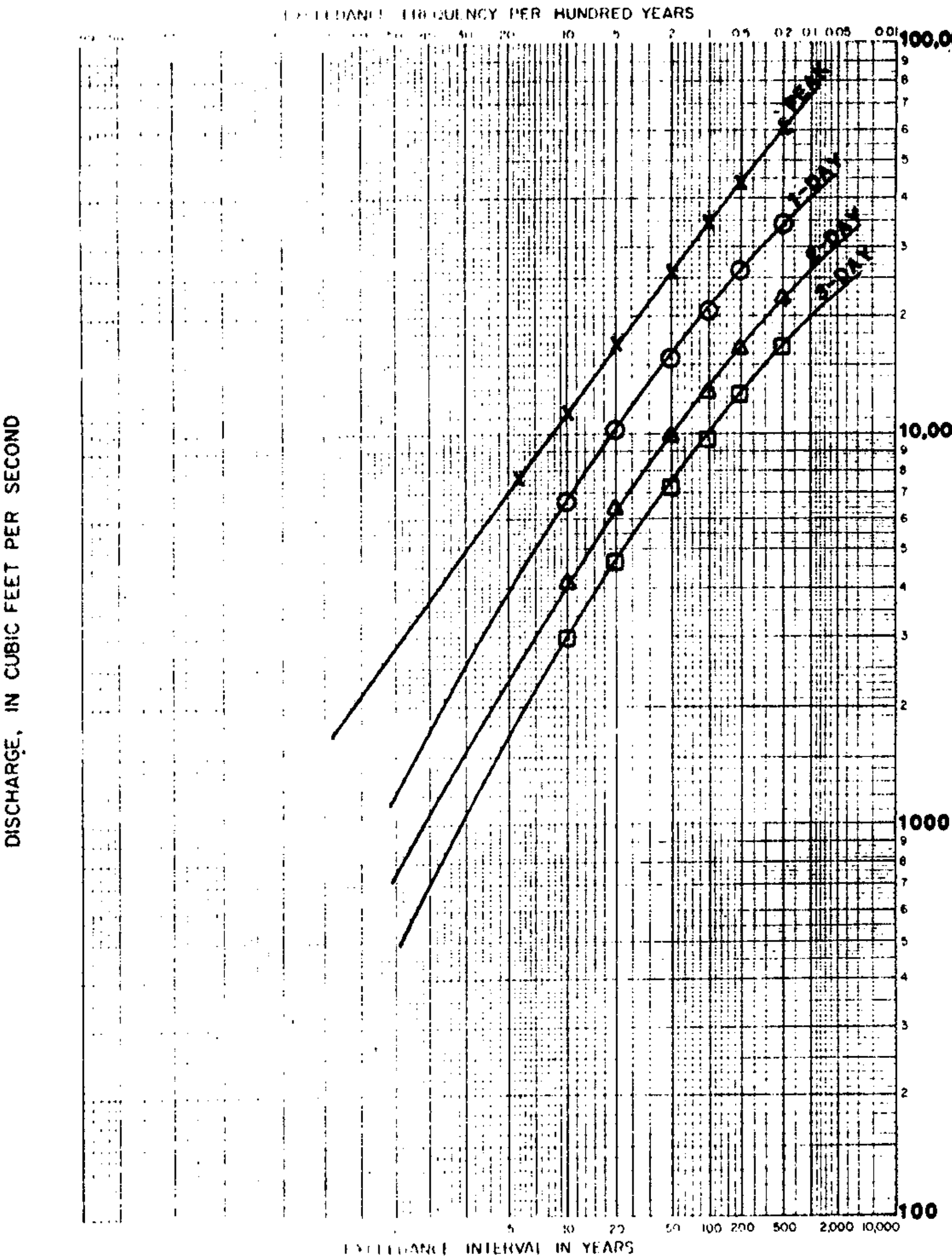
⊙ - Results of routing n-year frequency hydrographs

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE FREQUENCY CURVE

SALT RIVER AT GILBERT ROAD (CP-82)
DA = 1160 SQ. MI.

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



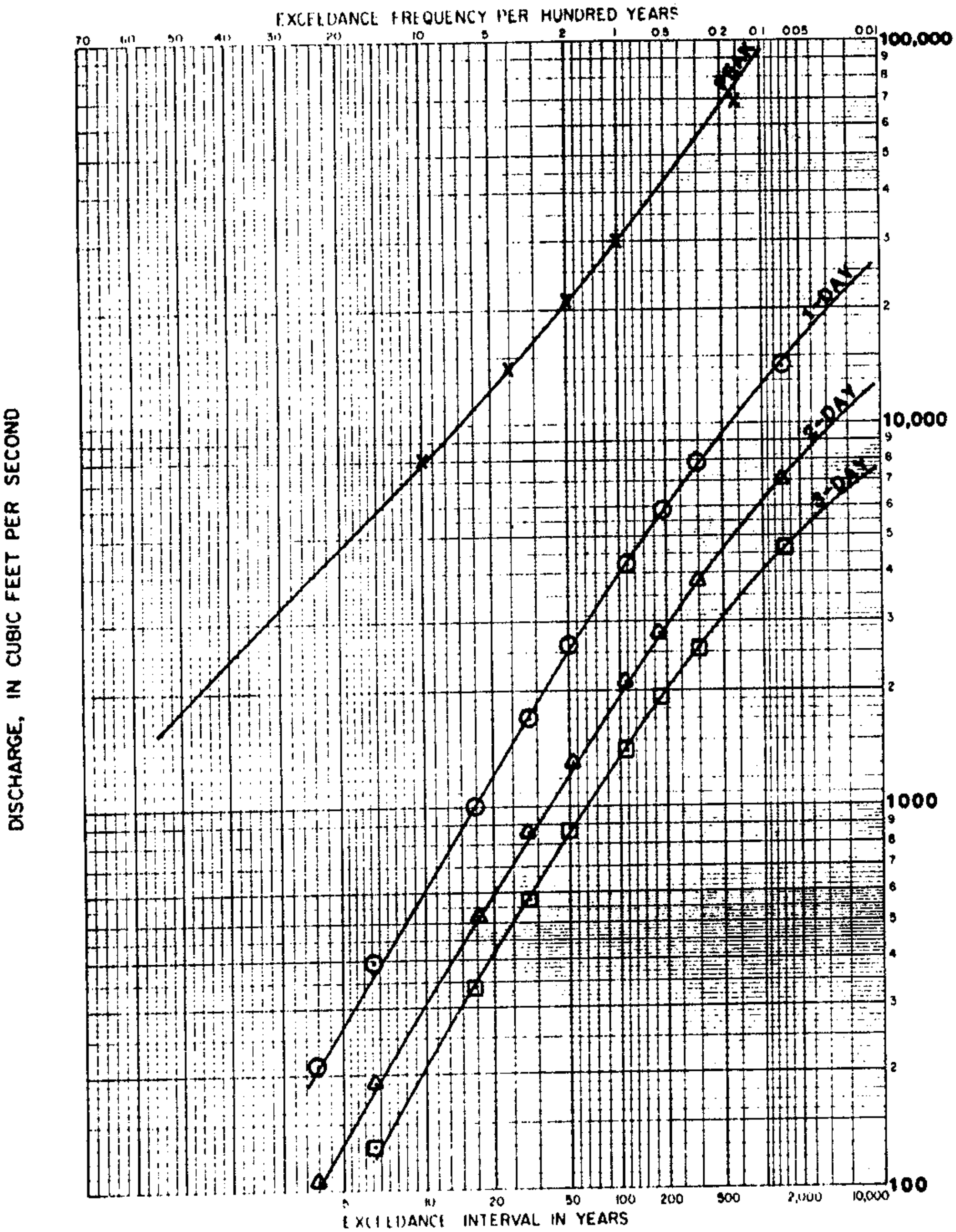
X, ⊙, △, ⊡ - Routed n-year frequency hydrographs

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY CURVES

SALT RIVER
ROUTED TO TEMPE BRIDGE
ABOVE INDIAN BEND WASH (CP-93)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



X – Peak probabilities, coincident analysis

⊙–1-Day
△–2-Day
⊡–3-Day

Volume corresponding to peak discharges

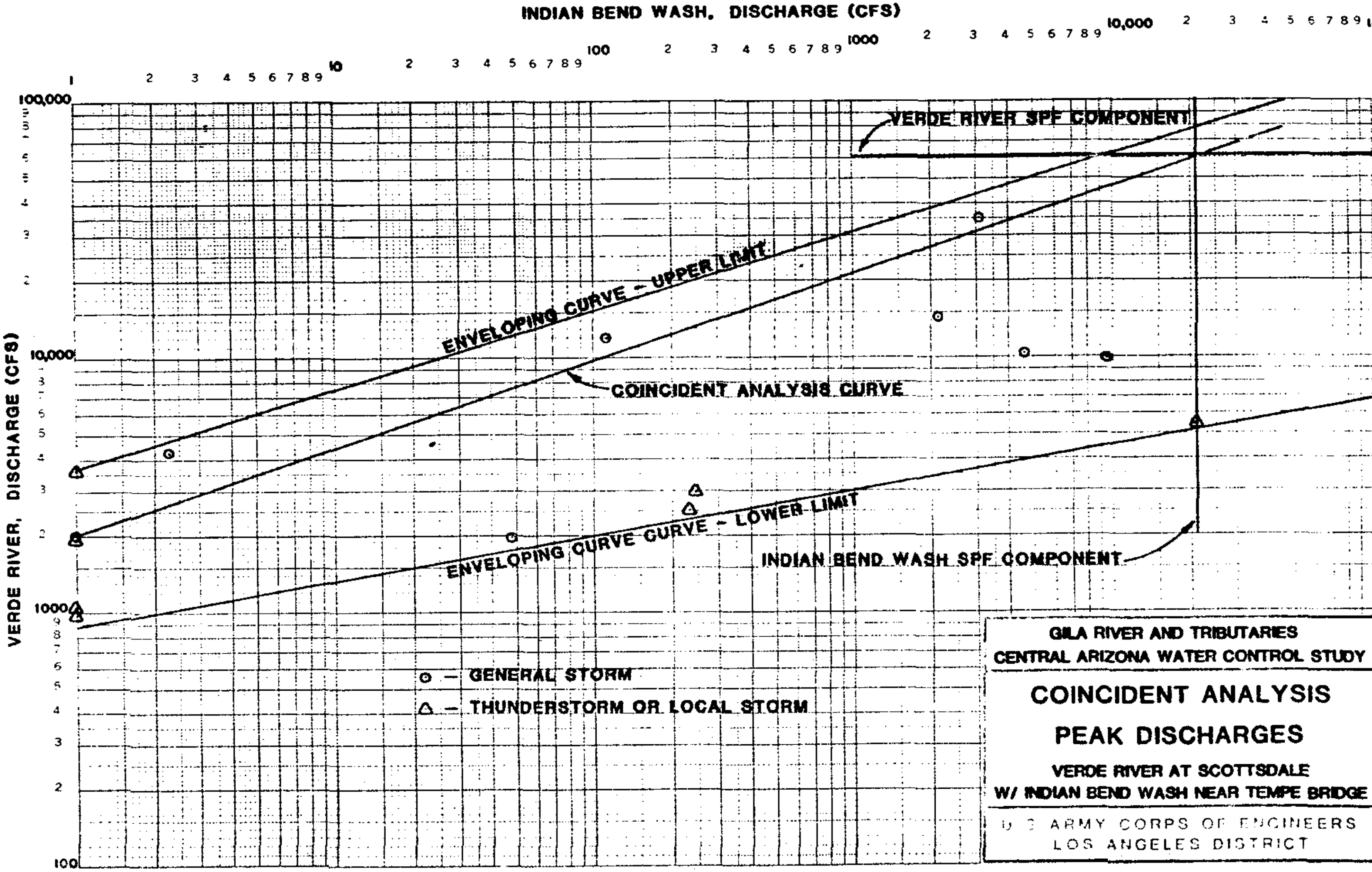
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY CURVES

INDIAN BEND WASH AT SALT RIVER

(CP-1101)

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

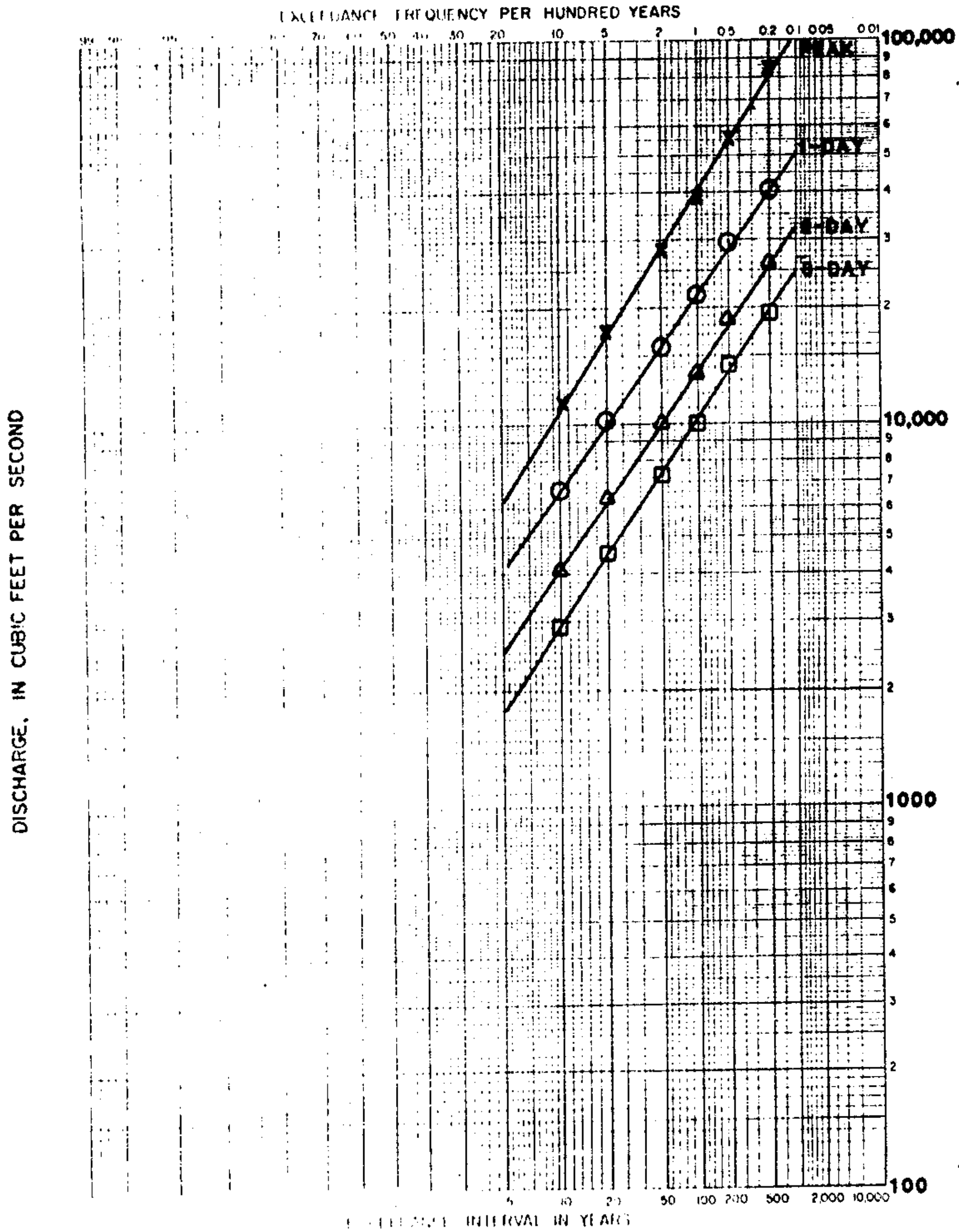


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

COINCIDENT ANALYSIS
PEAK DISCHARGES

VERDE RIVER AT SCOTTSDALE
W/ INDIAN BEND WASH NEAR TEMPE BRIDGE

U. S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



X - Combined peak discharges, dependent analysis

O - 1-Day
Δ - 2-Day
⊡ - 3-Day

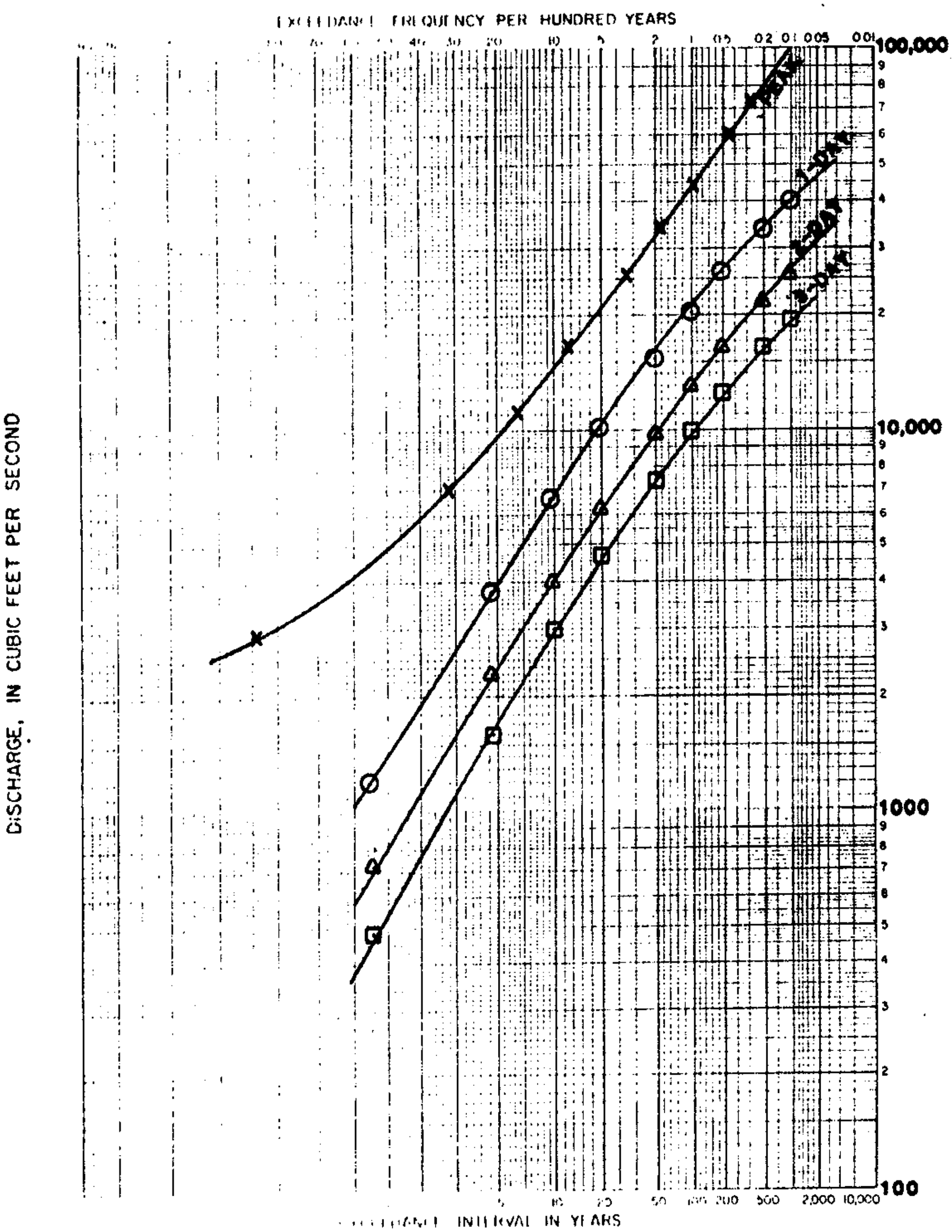
Volume corresponding to peak discharges

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY CURVES

DEPENDENT ANALYSIS FOR SALT RIVER COMBINED WITH INDIAN BEND WASH BELOW TEMPE BRIDGE (CP-93)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



X - Combined peak probabilities, independent analysis

⊙-1-Day	Volume corresponding to peak discharges
△-2-Day	
⊡-3-Day	

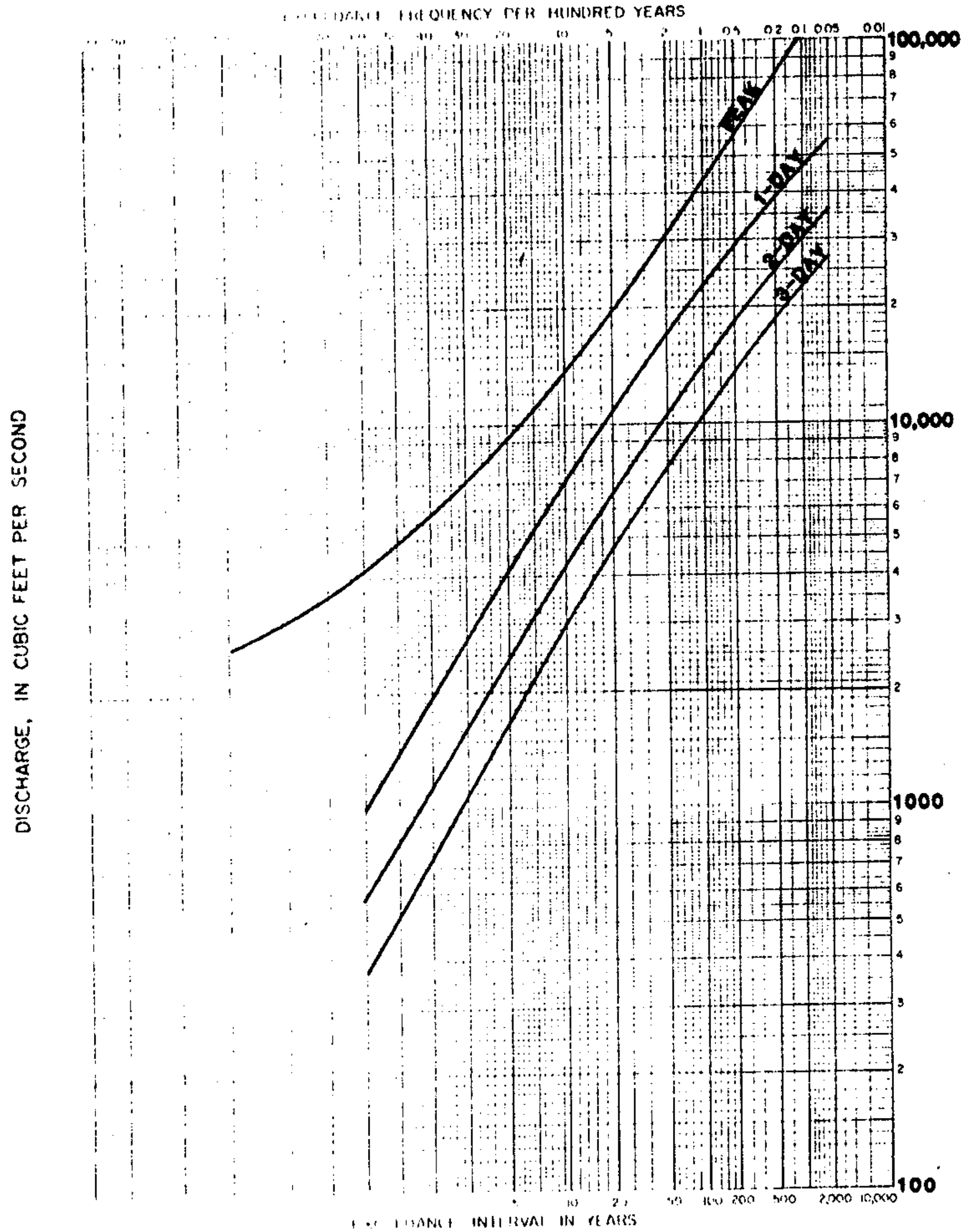
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY CURVES

INDEPENDENT ANALYSIS FOR SALT RIVER COMBINED WITH INDIAN BEND WASH BELOW TEMPE BRIDGE (CP-98)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 1-17

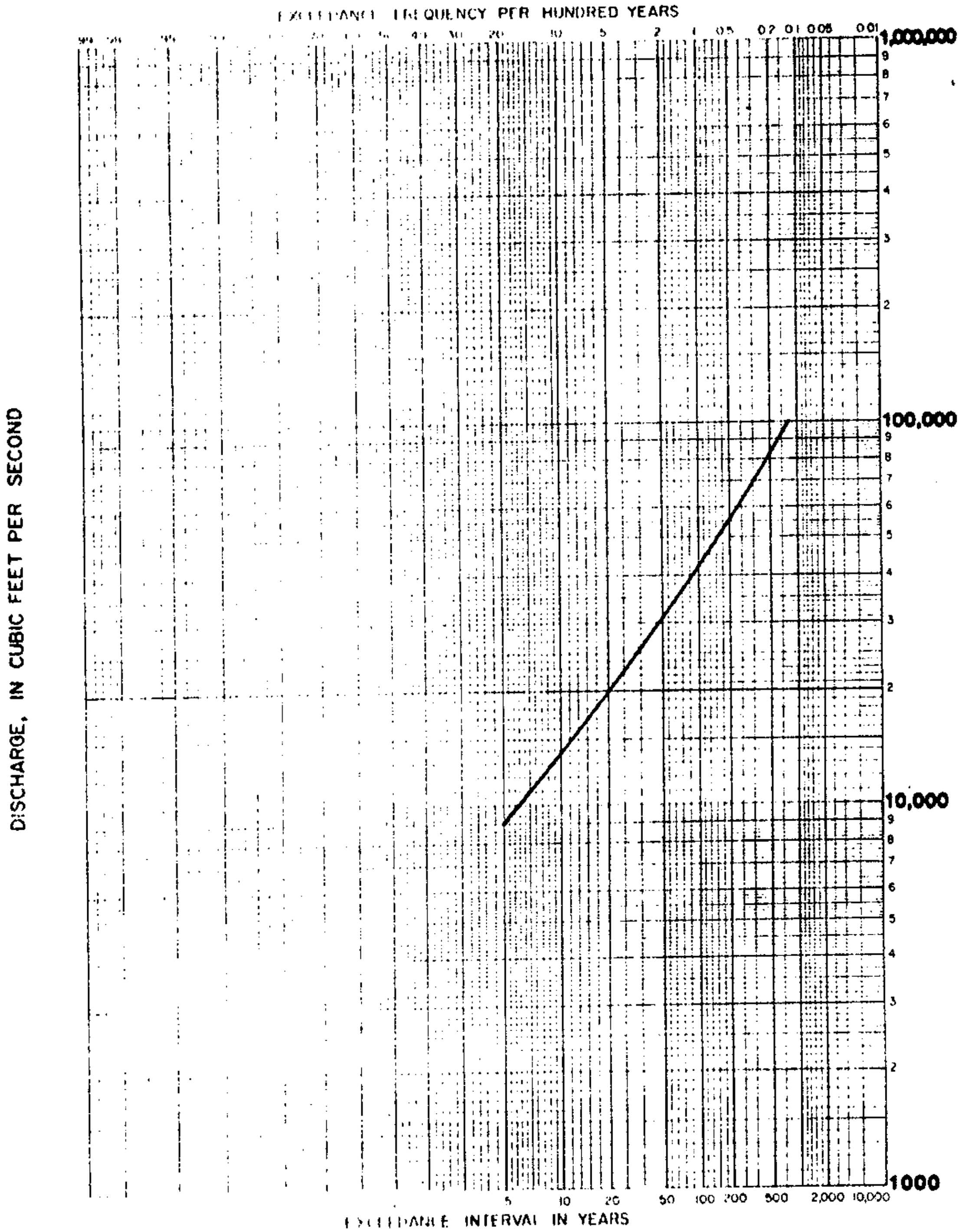


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY CURVES

COMPOSITE ANALYSIS FOR SALT RIVER COMBINED WITH INDIAN BEND WASH BELOW TEMPE BRIDGE (CP-93)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



GILA RIVER AND TRIBUTARIES

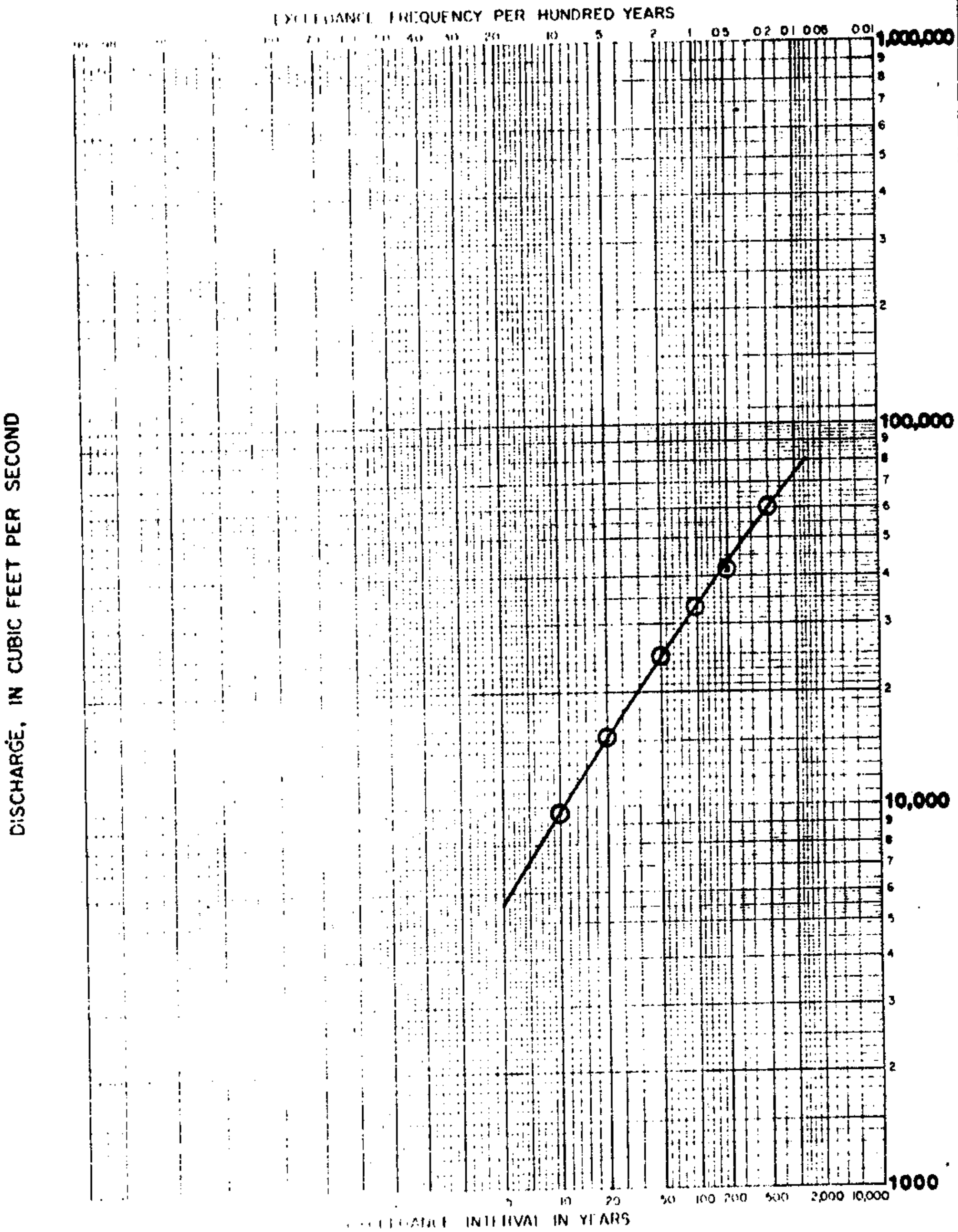
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE FREQUENCY CURVE

SALT RIVER PLUS INDIAN BEND WASH

BELOW TEMPE BRIDGE (CP-93)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

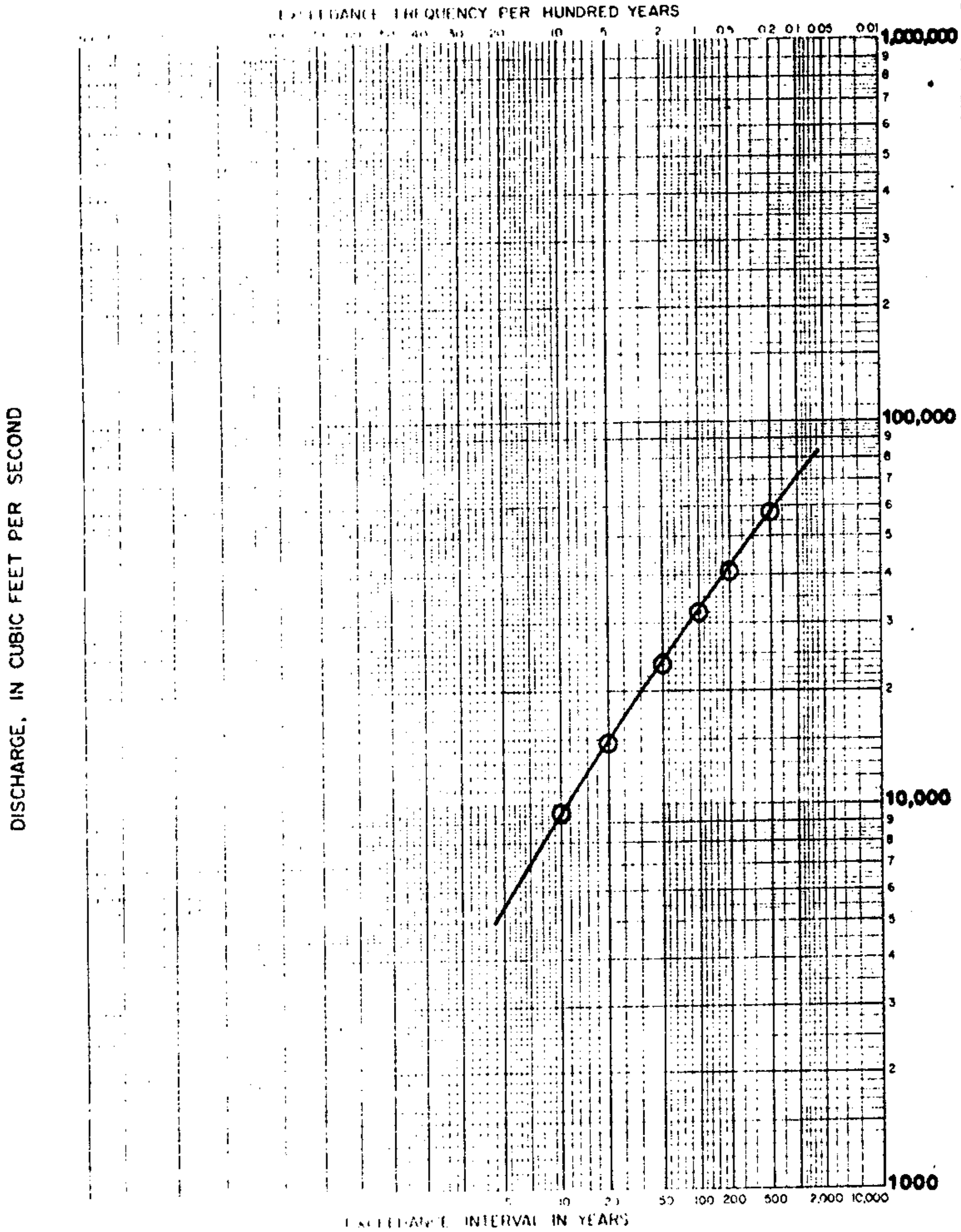


⊙ - Results of routing n-year frequency hydrographs

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE FREQUENCY CURVE
SALT RIVER AT CENTRAL AVENUE (CP-101)
DA = 1345 SQ. MI.

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



⊙ — Results of routing n-year frequency hydrographs

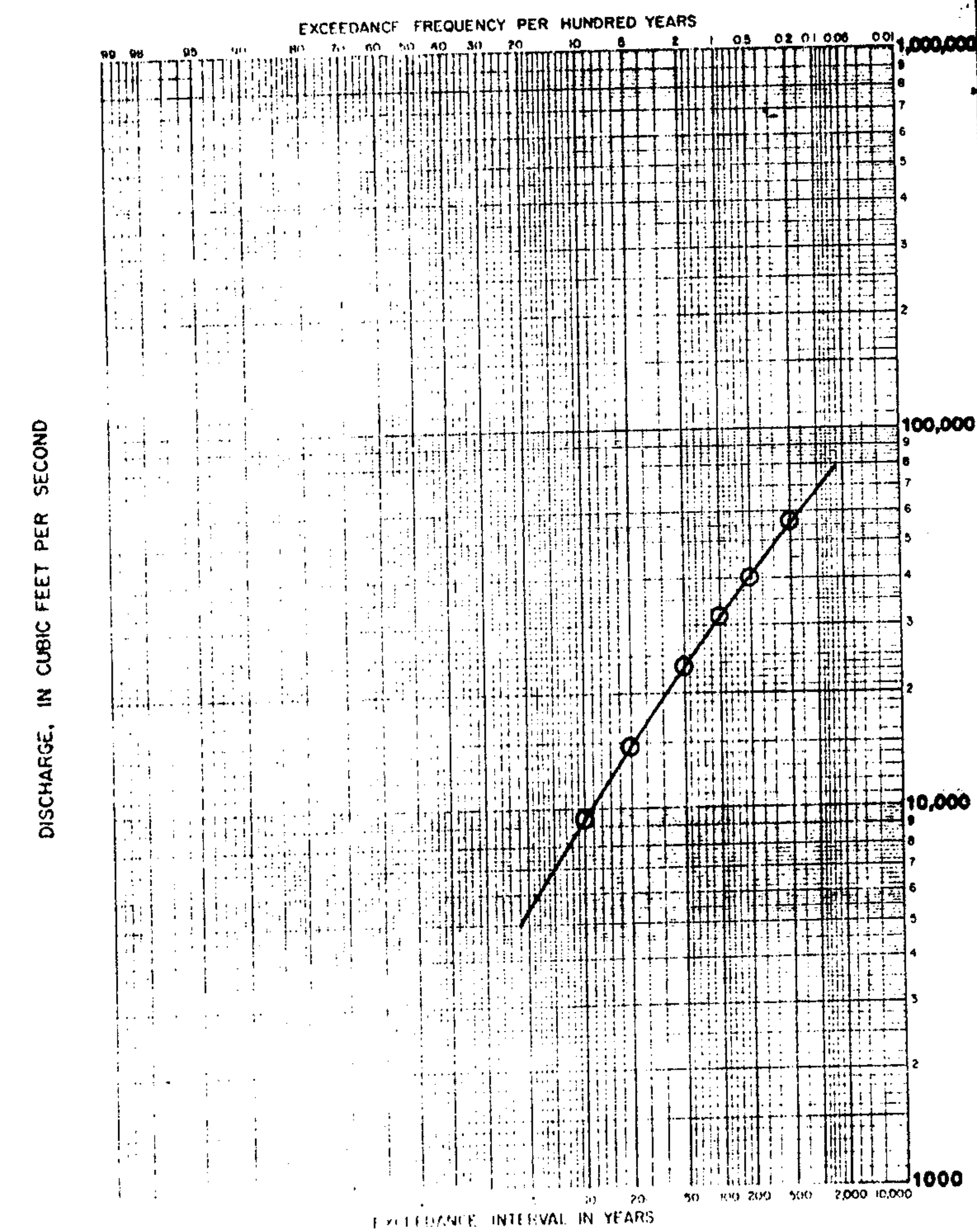
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE FREQUENCY CURVE

SALT RIVER AT 67TH AVENUE (CP-102)

DA =1427 SQ. MI.

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



⊙ - Results of routing n-year frequency hydrographs

GILA RIVER AND TRIBUTARIES

CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE FREQUENCY CURVE

SALT RIVER AT CONFLUENCE

WITH GILA RIVER (CP-103)

DA = 1476 SQ. MI.

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

APPENDIX 2

INTERMEDIATE PROJECT CONDITIONS RESULTS

STAGE II

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

May 1982

Appendix 2. Intermediate Project Condition Results, Stage II

Table of Contents

Appendix 2

Intermediate Project Condition Results, Stage II

2-1. Purpose.

This section has been included in order to present previously published results of the Stage II hydrologic study. These include only those Stage II results which were modified or screened-out during Stage III. The discussion of the purpose and methods for development of these intermediate results is included in section 8 of the main report.

TABLE 2-1a

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP40)

		Design flood (yrs.)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	290,000	245,000	175,000	141,000	102,000
2. Project conditions	Alternatives								
a. Structural elements:	ORME	50	50,000	320,000	230,000	155,000	50,000	50,000	50,000
New Roosevelt = NR	NR+NH			265,000	200,000	150,000	50,000	50,000	50,000
New Horseshoe = NH	NR+NB			265,000	200,000	155,000	50,000	50,000	50,000
New Bartlett = NB	R+H+B			215,000	150,000	110,000	50,000	50,000	50,000
Cliff = NB									
Confluence = ORME	NH	50	100,000	245,000	175,000	140,000	100,000	100,000	100,000
	NB			240,000	170,000	130,000	100,000	100,000	100,000
b. Reregulation elements:	ORME			340,000	265,000	200,000	100,000	100,000	100,000
Roosevelt = R	NR+NH			315,000	245,000	180,000	100,000	100,000	100,000
Horseshoe = H	NR+NB			300,000	240,000	175,000	100,000	100,000	100,000
Bartlett = B	R+H+B			255,000	195,000	150,000	100,000	100,000	100,000
	NR	50	150,000	350,000	265,000	205,000	150,000	141,000	102,000
Note: N/A = not applicable	NH			275,000	220,000	185,000	150,000	141,000	102,000
	NB			285,000	225,000	185,000	150,000	141,000	102,000
	ORME			355,000	270,000	215,000	150,000	141,000	102,000
	NR+NH			345,000	265,000	210,000	150,000	141,000	102,000

TABLE 2-1b

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP40)

		Design flood (yrs.)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	290,000	245,000	175,000	141,000	102,000
2. Project conditions	Alternatives								
a. Structural elements:	NR+NB	50	150,000	340,000	260,000	210,000	150,000	141,000	102,000
New Roosevelt = NR	R+H+B			275,000	220,000	185,000	150,000	141,000	102,000
New Horseshoe = NH									
New Bartlett = NB	Not required	50	200,000*	SAME AS EXISTING CONDITIONS					
Cliff = NB									
Confluence = ORME	NH	50	100,000	245,000	175,000	140,000	100,000	100,000	100,000
	NB			240,000	170,000	130,000	100,000	100,000	100,000
b. Reregulation elements:	ORME	100	50,000	250,000	145,000	50,000	50,000	50,000	50,000
Roosevelt = R	NR+NB			235,000	135,000	50,000	50,000	50,000	50,000
Horseshoe = H	R+H+B			180,000	88,000	50,000	50,000	50,000	50,000
Bartlett = B									
	ORME	100	100,000	275,000	175,000	100,000	100,000	100,000	100,000
	NR+NH			305,000	215,000	100,000	100,000	100,000	100,000
Note: N/A = not applicable	R+H+B[a]			215,000	150,000	100,000	100,000	100,000	100,000
	R+H+B[b]			205,000	135,000	100,000	100,000	100,000	100,000

* 50-yr flood for existing conditions is only 175,000 cfs.

a With extra storage.

b With flood control outlets.

TABLE 2-1c

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP40)

		Design flood (yrs.)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	290,000	245,000	175,000	141,000	102,000
2. Project conditions	Alternatives								
a. Structural elements:	NR	100	150,000	340,000	250,000	150,000	150,000	141,000	102,000
New Roosevelt = NR	NH			260,000	190,000	150,000	150,000	141,000	102,000
New Horseshoe = NH	NB			260,000	195,000	150,000	150,000	141,000	102,000
New Bartlett = NB	ORME			295,000	210,000	150,000	150,000	141,000	102,000
Cliff = NB	NR+NH			325,000	230,000	150,000	150,000	141,000	102,000
Confluence = ORME	NR+NB			320,000	225,000	150,000	150,000	141,000	102,000
	R+B			235,000	180,000	150,000	150,000	141,000	102,000
b. Reregulation elements:									
Roosevelt = R	NR	100	200,000	365,000	295,000	200,000	175,000	141,000	102,000
Horseshoe = H	NH			330,000	255,000	200,000	175,000	141,000	102,000
Bartlett = B	NB			320,000	250,000	200,000	175,000	141,000	102,000
	ORME			340,000	255,000	200,000	175,000	141,000	102,000
	NR+NH			325,000	250,000	200,000	175,000	141,000	102,000
Note: N/A = not applic-	NR+NB			340,000	255,000	200,000	175,000	141,000	102,000
able	R			260,000	215,000	188,000	175,000	141,000	102,000

TABLE 2-1d

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP40)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	295,000	245,000	175,000	141,000	102,000
2. Project conditions	Alternatives								
a. Structural elements:	ORME	SPF	50,000	180,000	50,000	50,000	50,000	50,000	50,000
New Roosevelt = NR	NR+NH			210,000	80,000	50,000	50,000	50,000	50,000
New Horseshoe = NH	NR+NB			195,000	86,000	50,000	50,000	50,000	50,000
New Bartlett = NB									
Cliff = NB									
Confluence = ORME	ORME	SPF	100,000	210,000	100,000	100,000	100,000	100,000	100,000
	NR+NH			185,000	125,000	100,000	100,000	100,000	100,000
b. Reregulation elements:	NR+NB			190,000	125,000	100,000	100,000	100,000	100,000
Roosevelt = R									
Horseshoe = H									
Bartlett = B	NB	SPF	150,000	195,000	160,000	150,000	150,000	141,000	102,000
	ORME			255,000	175,000	150,000	150,000	141,000	102,000
	NR+NH			255,000	180,000	150,000	150,000	141,000	102,000
Note: N/A = not applicable	NR+NB			270,000	190,000	150,000	150,000	141,000	102,000
	R+H+B[a]			225,000	160,000	150,000	150,000	141,000	102,000
	R+H+B[b]			190,000	150,000	150,000	150,000	141,000	102,000

[a] With extra storage.
[b] With flood control outlets.

TABLE 2-1e

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP40)

	Alternatives	Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	290,000	245,000	175,000	141,000	102,000
2. Project conditions	Alternatives								
a. Structural elements:	NR	SPF	200,000	315,000	220,000	200,000	175,000	141,000	102,000
New Roosevelt = NR	NH			285,000	215,000	200,000	175,000	141,000	102,000
New Horseshoe = NH	NB			285,000	220,000	200,000	175,000	141,000	102,000
New Bartlett = NB	ORME			275,000	205,000	200,000	175,000	141,000	102,000
Cliff = NB	NR+NH			300,000	230,000	200,000	175,000	141,000	102,000
Confluence = ORME	NR+NB			305,000	230,000	200,000	175,000	141,000	102,000
	R+H+B			220,000	170,000	170,000	170,000	141,000	102,000
b. Reregulation elements:									
Roosevelt = R									
Horseshoe = H									
Bartlett = B									

Note: N/A = not applicable

TABLE 2-2a

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP113)

		Peak Discharge, cfs							
		Design flood (yrs)	Design target (cfs)	Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	310,00	250,000	185,000	145,000	125,000	85,000
2. Project conditions	Alternatives								
a. Structural elements:	ORME	50	50,000	260,000	200,000	140,000	44,000	44,000	44,000
New Roosevelt = NR	NR+NH			225,000	175,000	130,000	44,000	44,000	44,000
New Horseshoe = NH	NR+NB			235,000	180,000	130,000	44,000	44,000	44,000
New Bartlett = NB	R+H+B			180,000	130,000	100,000	44,000	44,000	44,000
Cliff = NB									
Confluence = ORME	NH	50	100,000	205,000	155,000	125,000	90,000	90,000	85,000
	NB			195,000	145,000	115,000	90,000	90,000	85,000
b. Reregulation elements:	ORME			285,000	225,000	165,000	90,000	90,000	85,000
Roosevelt = R	NR+NH			255,000	200,000	155,000	90,000	90,000	85,000
Horseshoe = H	NR+NB			250,000	200,000	150,000	90,000	90,000	85,000
Bartlett = B	R+H+B			215,000	165,000	130,000	90,000	90,000	85,000
	NR	50	150,000	285,000	225,000	180,000	130,000	125,000	85,000
	NH			230,000	185,000	160,000	130,000	125,000	85,000
	NB			235,000	190,000	160,000	130,000	125,000	85,000
	ORME			290,000	230,000	180,000	130,000	125,000	85,000
	NR+NH			285,000	225,000	180,000	130,000	125,000	85,000

Note: N/A = not applicable

TABLE 2-2b

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP113)

				Peak Discharge, cfs					
		Design flood (yrs)	Design target (cfs)	Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	310,000	250,000	185,000	145,000	125,000	85,000
Project conditions	Alternatives								
a. Structural elements:	NR+NB	50	150,000	280,000	220,000	175,000	130,000	125,000	85,000
New Roosevelt = NR	R+H+B			225,000	190,000	160,000	130,000	125,000	85,000
New Horseshoe = NH									
New Bartlett = NB	Not required	50	200,000	SAME AS EXISTING CONDITIONS					
Cliff = NB									
Confluence = ORME		100	50,000						
	ORME			210,000	120,000	44,000	44,000	44,000	44,000
b. Reregulation elements:	NR+NH			190,000	105,000	44,000	44,000	44,000	44,000
Roosevelt = R	NR+NB			200,000	110,000	44,000	44,000	44,000	44,000
Horseshoe = H	R+H+B			150,000	80,000	49,000	44,000	44,000	44,000
Bartlett = B									
	ORME	100	100,000	235,000	150,000	90,000	90,000	90,000	85,000
	NR+NH			250,000	175,000	90,000	90,000	90,000	85,000
Note: N/A = not applicable	NR+NB			240,000	170,000	90,000	90,000	90,000	85,000
	R+H+B[a]			188,000	135,000	90,000	90,000	90,000	85,000
	R+H+B[b]			170,000	120,000	90,000	90,000	90,000	85,000

[a] With extra storage.
[b] With flood control outlets.

TABLE 2-2c

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP113)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	310,000	250,000	185,000	145,000	125,000	85,000
2. Project conditions	Alternatives								
a. Structural elements:	NR	100	150,000	270,000	205,000	130,000	130,000	125,000	85,000
New Roosevelt = NR	NH			220,000	165,000	130,000	130,000	125,000	85,000
New Horseshoe = NH	NB			215,000	165,000	130,000	130,000	125,000	85,000
New Bartlett = NB	ORME			245,000	175,000	130,000	130,000	125,000	85,000
Cliff = NB	NR+NH			260,000	195,000	130,000	130,000	125,000	85,000
Confluence = ORME	NR+NB			265,000	195,000	130,000	130,000	125,000	85,000
	R+B			195,000	155,000	130,000	130,000	125,000	85,000
b. Reregulation elements:									
Roosevelt = R	NR	100	200,000	295,000	235,000	170,000	145,000	125,000	85,000
Horseshoe = H	NH			275,000	215,000	170,000	145,000	125,000	85,000
Bartlett = B	NB			260,000	210,000	170,000	145,000	125,000	85,000
	ORME			275,000	215,000	170,000	145,000	125,000	85,000
	NR+NH			265,000	210,000	170,000	145,000	125,000	85,000
Note: N/A = not applicable	NR+NB			275,000	215,000	170,000	145,000	125,000	85,000
	R			215,000	185,000	160,000	145,000	125,000	85,000

TABLE 2-2d

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP113)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs — Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	310,000	250,000	185,000	145,000	125,000	85,000
2. Project conditions	Alternatives								
a. Structural elements:	ORME	SPF	50,000	150,000	44,000	44,000	44,000	44,000	44,000
New Roosevelt = NR	NR+NH			165,000	67,000	44,000	44,000	44,000	44,000
New Horseshoe = NH	NR+NB			160,000	72,000	44,000	44,000	44,000	44,000
New Bartlett = NB									
Cliff = NB									
Confluence = ORME	ORME	SPF	100,000	170,000	90,000	90,000	90,000	90,000	85,000
	NR+NH			160,000	110,000	90,000	90,000	90,000	85,000
b. Reregulation	NR+NB			165,000	110,000	90,000	90,000	90,000	85,000
Roosevelt = R									
Horseshoe = H		SPF	150,000						
Bartlett = B	NB			165,000	140,000	130,000	130,000	125,000	85,000
	ORME			215,000	150,000	130,000	130,000	125,000	85,000
	NR+NH			215,000	155,000	130,000	130,000	125,000	85,000
Note: N/A = not applicable	NR+NB			225,000	165,000	130,000	130,000	125,000	85,000
	R+H+B[a]			195,000	140,000	130,000	130,000	125,000	85,000
	R+H+B[b]			160,000	135,000	130,000	130,000	125,000	85,000

[a] With extra storage.
[b] With flood control outlets.

TABLE 2-2e

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP113)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs — Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	310,000	250,000	185,000	145,000	125,000	85,000
2. Project conditions	Alternatives								
a. Structural elements:	NR	SPF	200,000	265,000	190,000	170,000	145,000	125,000	85,000
New Roosevelt = NR	NH			235,000	180,000	170,000	145,000	125,000	85,000
New Horseshoe = NH	NB			235,000	185,000	170,000	145,000	125,000	85,000
New Bartlett = NB	ORME			130,000	175,000	170,000	145,000	125,000	85,000
Cliff = NB	NR+NH			245,000	195,000	170,000	145,000	125,000	85,000
Confluence = ORME	NR+NB			250,000	195,000	170,000	145,000	125,000	85,000
	R+H+B			185,000	145,000	145,000	145,000	125,000	85,000
b. Reregulation elements:									
Roosevelt = R									
Horseshoe = H									
Bartlett = B									

Note: N/A = not applicable

TABLE 2-3a

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP1310)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs					
				Frequency, yrs.					
				500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	295,000	250,000	200,000	135,000	95,000
2. Project conditions	Alternatives								
a. Structural elements:	ORME	50	50,000	330,000	260,000	200,000	140,000	59,000	44,000
New Roosevelt = NR	NR+NH			330,000	245,000	185,000	120,000	55,000	44,000
New Horseshoe = NH	NR+NB			330,000	245,000	185,000	125,000	59,000	44,000
New Bartlett = NB	R+H+B			250,000	185,000	145,000	100,000	53,000	44,000
Cliff = NB									
Confluence = ORME	NH	50	100,000	275,000	220,000	180,000	140,000	90,000	90,000
	NB			270,000	215,000	175,000	135,000	91,000	90,000
b. Reregulation elements:	ORME			330,000	270,000	225,000	170,000	95,000	90,000
Roosevelt = R	NR+NH			335,000	265,000	210,000	160,000	90,000	90,000
Horseshoe = H	NR+NB			330,000	260,000	215,000	160,000	93,000	90,000
Bartlett = B	R+H+B			285,000	225,000	185,000	145,000	90,000	90,000
	NR	50	150,000	335,000	270,000	220,000	170,000	130,000	95,000
	NH			295,000	240,000	205,000	165,000	130,000	95,000
Note: N/A = not applicable	NB			290,000	235,000	200,000	165,000	130,000	95,000
	ORME			335,000	270,000	225,000	180,000	130,000	95,000
	NR+NH			335,000	270,000	225,000	175,000	130,000	95,000

TABLE 2-3b

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP1310)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs — Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	295,000	250,000	200,000	135,000	95,000
2. Project conditions	Alternatives								
a. Structural elements:	NR+NB	50	150,000	335,000	265,000	225,000	175,000	130,000	95,000
New Roosevelt = NR	R+H+B			305,000	250,000	205,000	165,000	130,000	95,000
New Horseshoe = NH									
New Bartlett = NB	Not required	50	200,000	SAME AS EXISTING CONDITIONS					
Cliff = NB									
Confluence = ORME									
	ORME	100	50,000	230,000	170,000	130,000	95,000	55,000	44,000
a. Reregulation elements:	NR+NH			225,000	165,000	130,000	92,000	50,000	44,000
Roosevelt = R	NR+NB			230,000	165,000	125,000	90,000	52,000	44,000
Horseshoe = H	R+H+B			190,000	140,000	110,000	80,000	50,000	44,000
Bartlett = B		100	100,000						
	ORME			285,000	220,000	175,000	135,000	92,000	90,000
	NR+NH			300,000	230,000	185,000	135,000	90,000	90,000
Note: N/A = not applicable	NR+NB			300,000	235,000	185,000	135,000	90,000	90,000
	R+H+B[a]			255,000	200,000	165,000	130,000	90,000	90,000
	R+H+B[b]			240,000	190,000	160,000	125,000	90,000	90,000

[a] With extra storage.
[b] With flood control outlets.

TABLE 2-3c

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP1310)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs. Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	295,000	250,000	200,000	135,000	95,000
2. Project conditions	Alternatives								
a. Structural elements:	NR	100	150,000	315,000	245,000	205,000	160,000	130,000	95,000
New Roosevelt = NR	NH			280,000	230,000	195,000	160,000	130,000	95,000
New Horseshoe = NH	NB			275,000	225,000	190,000	160,000	130,000	95,000
New Bartlett = NB	ORME			290,000	230,000	195,000	160,000	130,000	95,000
Cliff = NB	NR+NH			300,000	240,000	200,000	160,000	130,000	95,000
Confluence = ORME	NR+NB			310,000	245,000	200,000	165,000	130,000	95,000
	R+B			255,000	215,000	185,000	155,000	130,000	95,000
b. Reregulation elements:									
Roosevelt = R	NR	100	200,000	340,000	265,000	220,000	170,000	135,000	95,000
Horseshoe = H	NH			330,000	260,000	215,000	170,000	135,000	95,000
Bartlett = B	NB			325,000	265,000	220,000	175,000	135,000	95,000
	ORME			315,000	255,000	215,000	175,000	135,000	95,000
Note: N/A = not applicable	NR+NH			325,000	255,000	210,000	170,000	135,000	95,000
	NR+NB			335,000	270,000	220,000	175,000	135,000	95,000
	R			280,000	230,000	200,000	160,000	135,000	95,000

TABLE 2-3d

PROJECT CONDITIONS

RESULTS OF DISCHARGE FREQUENCY ANALYSIS

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP1310)

	Alternatives	Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs — Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	295,000	250,000	200,000	135,000	95,000
2. Project conditions									
a. Structural elements:	ORME	SPF	50,000	160,000	120,000	99,000	75,000	47,000	44,000
New Roosevelt = NR	NR+NH			185,000	140,000	115,000	85,000	46,000	44,000
New Horseshoe = NH	NR+NB			175,000	135,000	105,000	80,000	48,000	44,000
New Bartlett = NB									
Cliff = NB									
Confluence = ORME	ORME	SPF	100,000	210,000	170,000	145,000	115,000	90,000	90,000
	NR+NH			240,000	190,000	155,000	120,000	90,000	90,000
b. Reregulation	NR+NB			235,000	190,000	160,000	125,000	90,000	90,000
Roosevelt = R									
Horseshoe = H									
Bartlett = B	NB	SPF	150,000	230,000	200,000	180,000	155,000	130,000	95,000
	ORME			275,000	225,000	190,000	160,000	130,000	95,000
	NR+NH			265,000	220,000	185,000	150,000	130,000	95,000
Note: N/A = not applic-	NR+NB			280,000	225,000	190,000	155,000	130,000	95,000
able	R+H+B[a]			235,000	210,000	185,000	158,000	130,000	95,000
	R+H+B[b]			225,000	195,000	175,000	150,000	130,000	95,000

[a] With extra storage.
[b] With flood control outlets.

TABLE 2-3e

PROJECT CONDITIONS

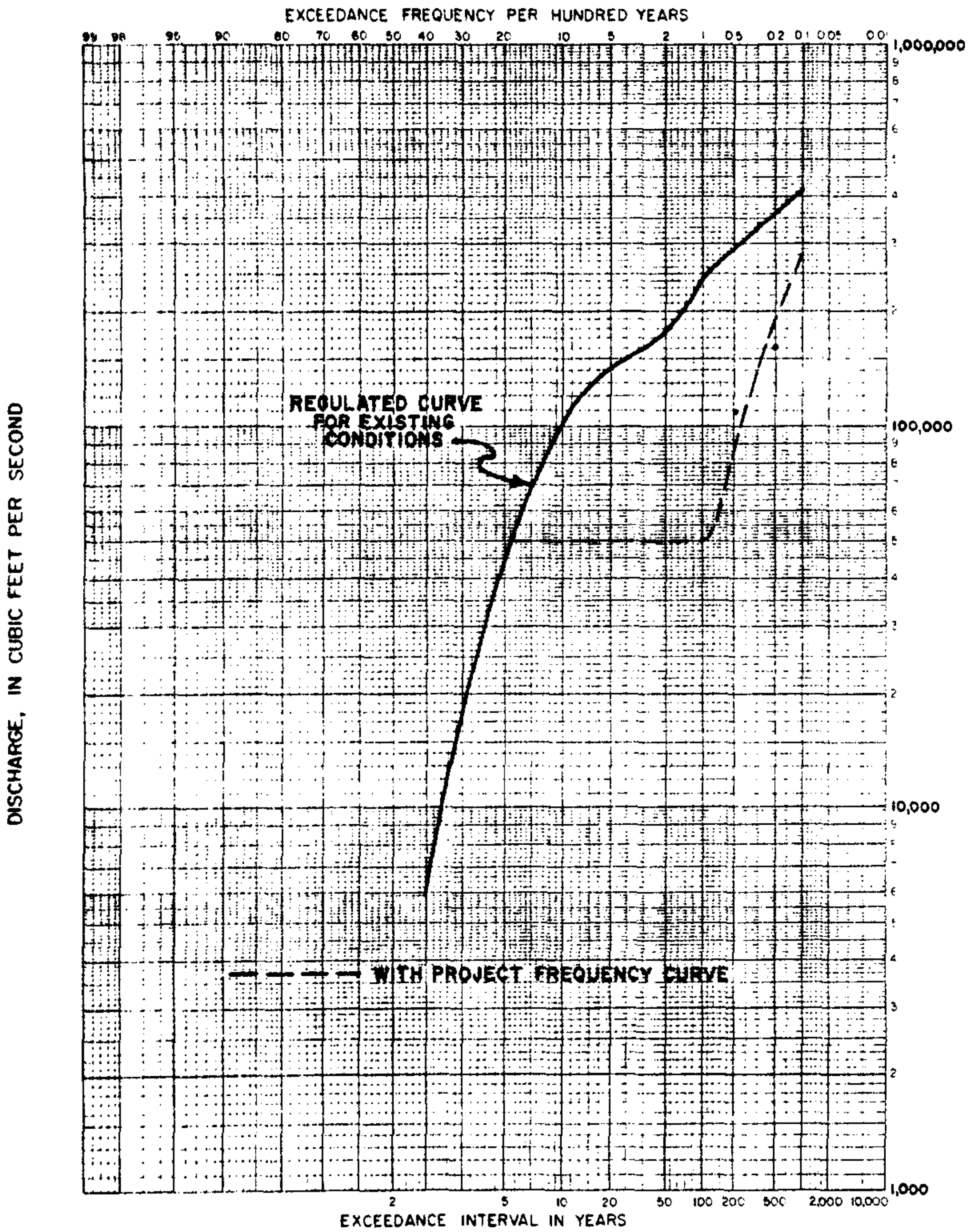
RESULTS OF DISCHARGE FREQUENCY ANALYSIS

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER (CP1310)

		Design flood (yrs)	Design target (cfs)	Peak Discharge, cfs — Frequency, yrs. 500	200	100	50	20	10
1. Existing conditions		N/A	N/A	360,000	295,000	250,000	200,000	135,000	95,000
2. Project conditions	Alternatives								
a. Structural elements:	NR	SPF	200,000	300,000	240,000	200,000	170,000	135,000	95,000
New Roosevelt = NR	NH			290,000	235,000	195,000	170,000	135,000	95,000
New Horseshoe = NH	NB			300,000	235,000	195,000	170,000	135,000	95,000
New Bartlett = NB	ORME			300,000	245,000	205,000	170,000	135,000	95,000
Cliff = NB	NR+NH			305,000	240,000	200,000	170,000	135,000	95,000
Confluence = ORME	NR+NB			315,000	250,000	205,000	170,000	135,000	95,000
	R+H+B			240,000	210,000	185,000	160,000	135,000	95,000

b. Reregulation elements:
Roosevelt = R
Horseshoe = H
Bartlett = B

Note: N/A = not applicable.



ALTERNATIVE = NRNB

DESIGN = SPF

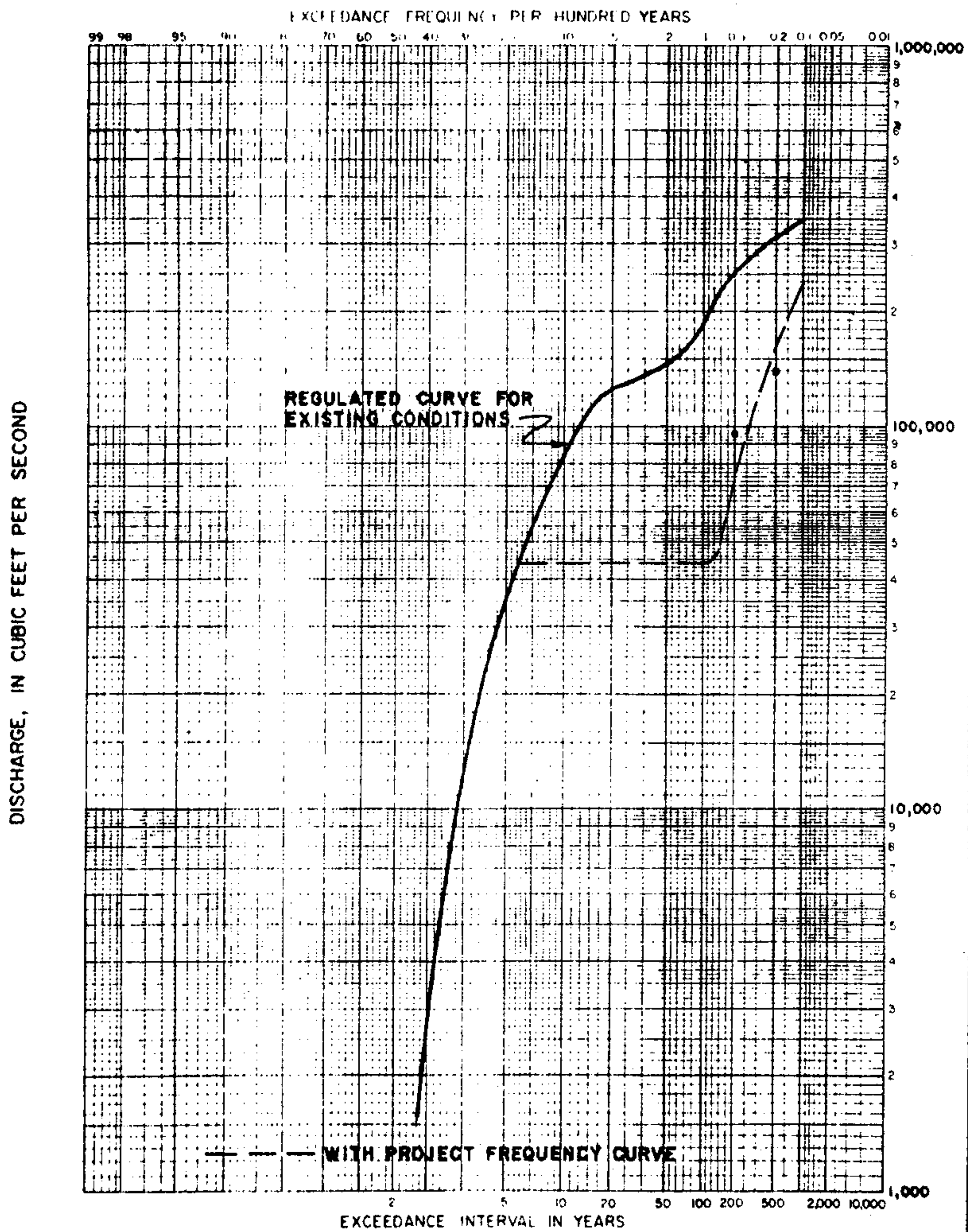
TARGET = 50,000 CFS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED



ALTERNATIVE = NR & NB

DESIGN = SPF

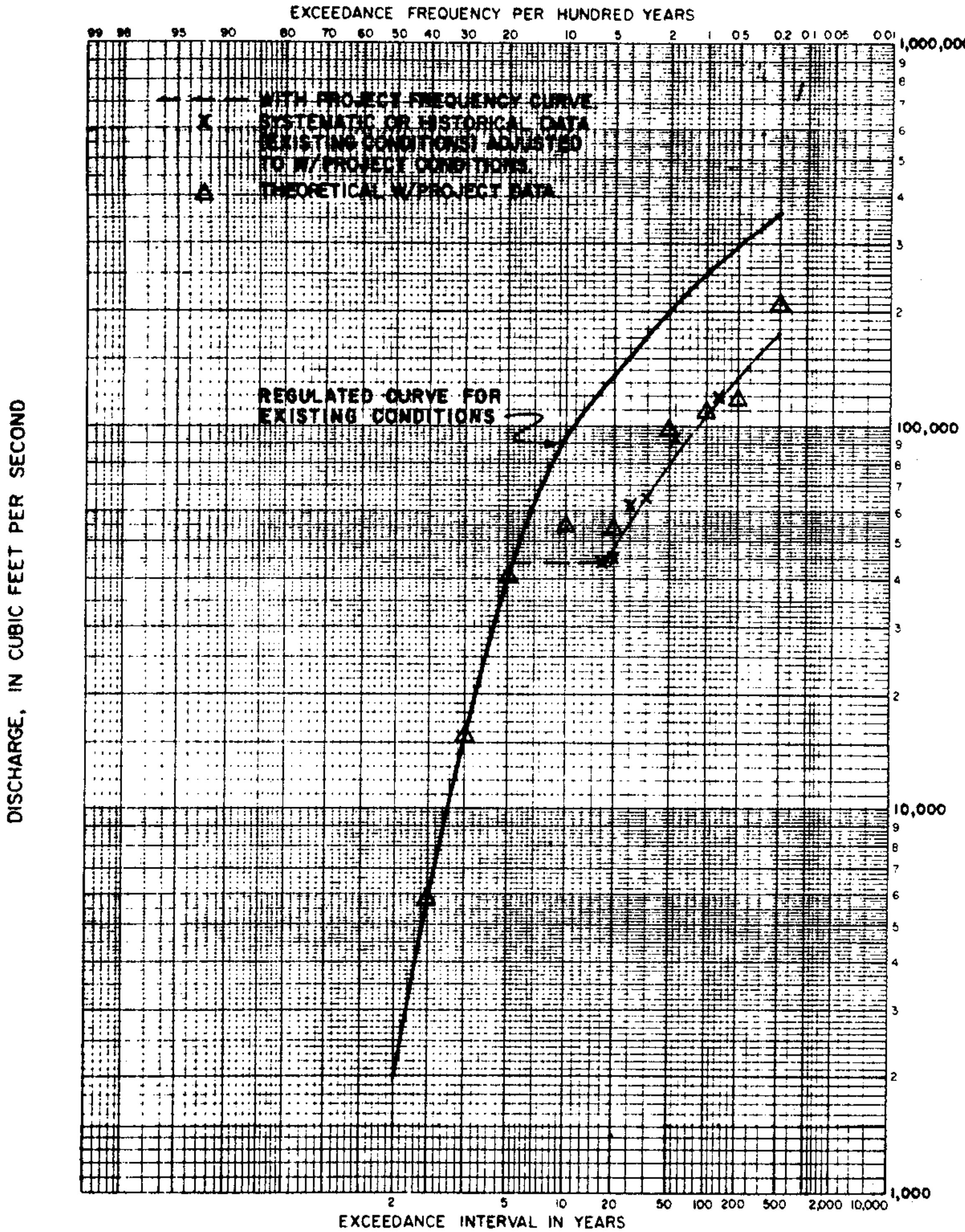
TARGET = 50,000 CFS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE - FREQUENCY CURVES

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP 113)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED



ALTERNATIVE = NR & NB

DESIGN = SPF

TARGET = 50,000 CFS

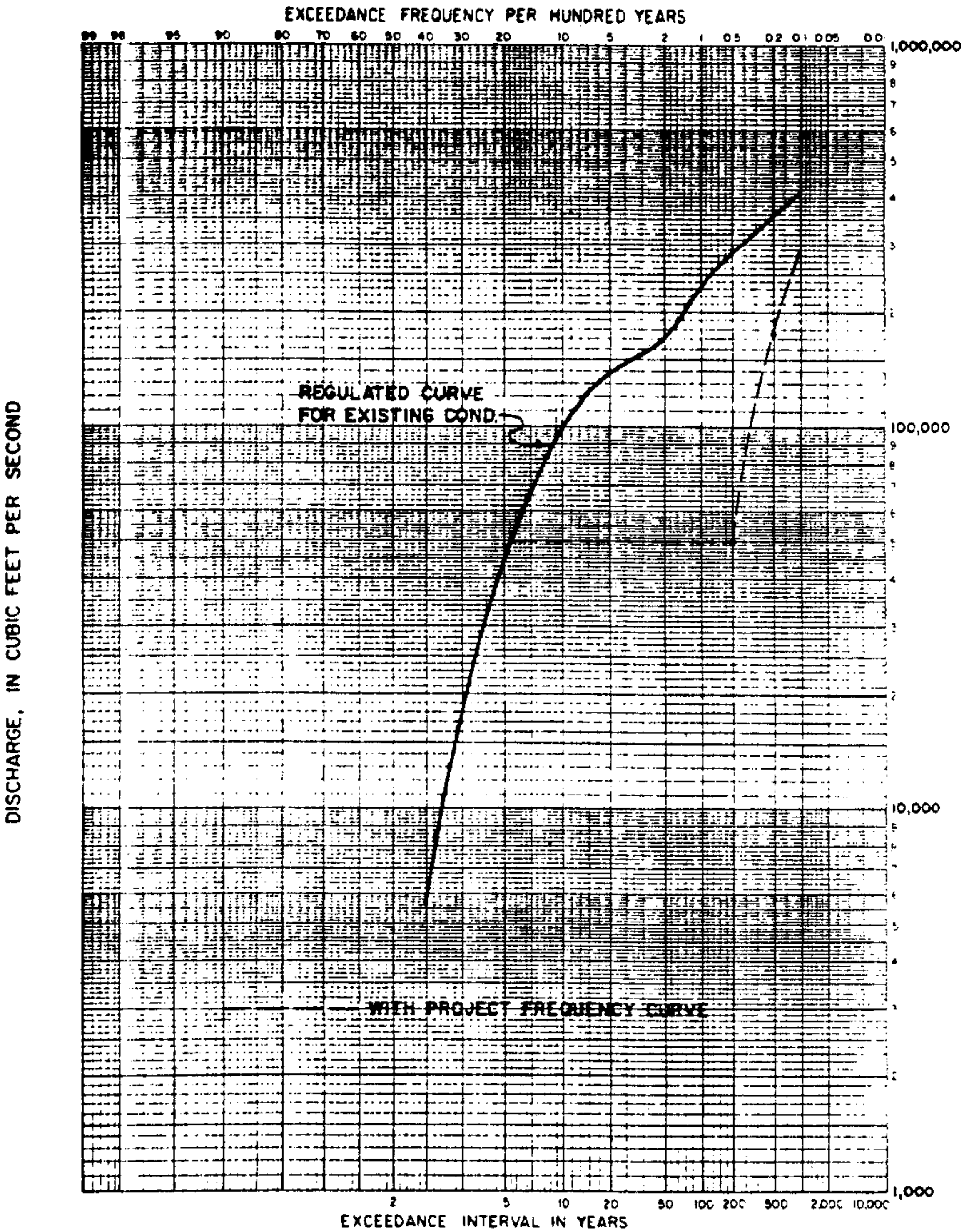
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED

PLATE 2-1c



ALTERNATIVE = ORME

DESIGN = SPF

TARGET = 50,000 CFS

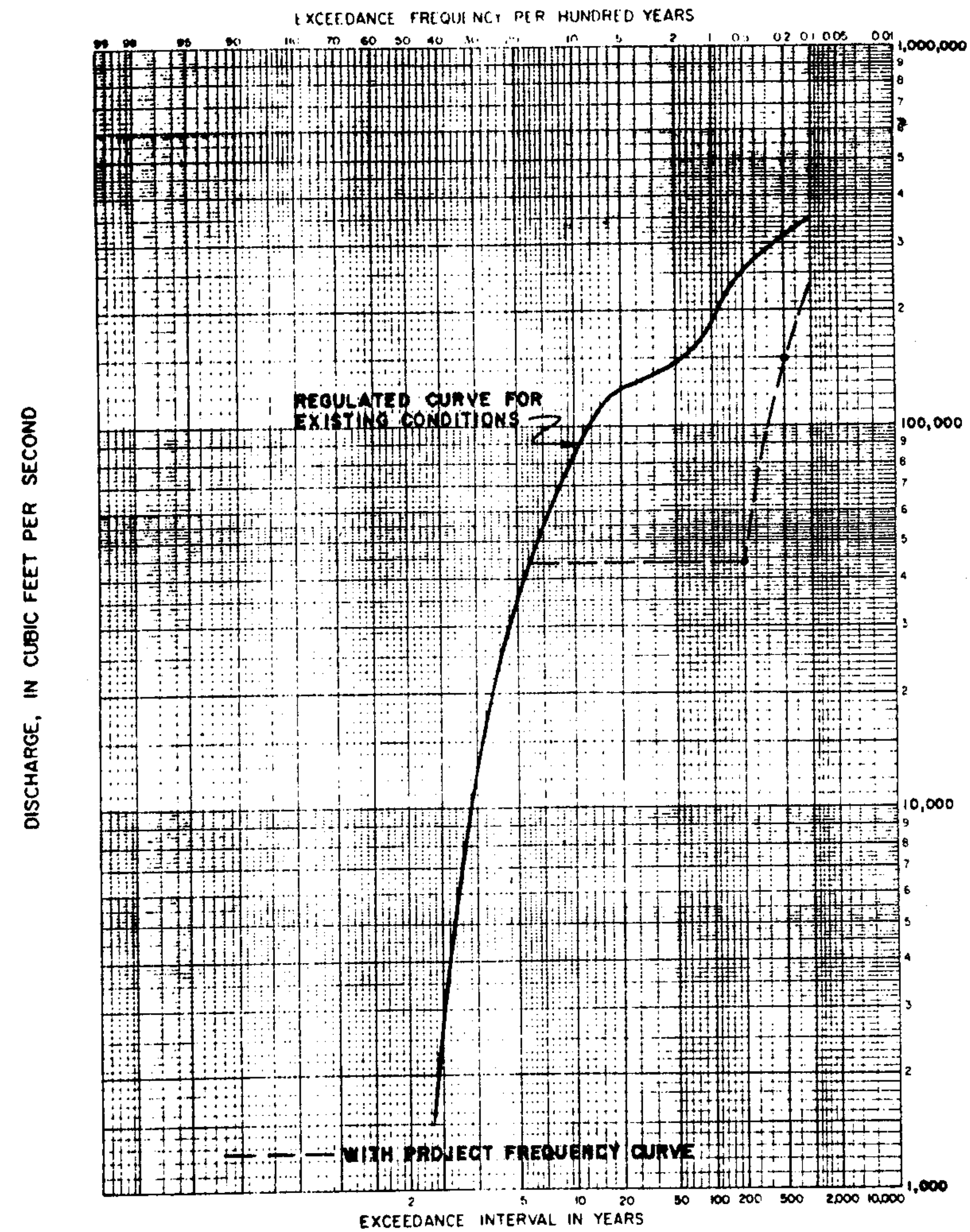
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

TO ACCOMPANY REPORT DATED



ALTERNATIVE = ORME

DESIGN = SPF

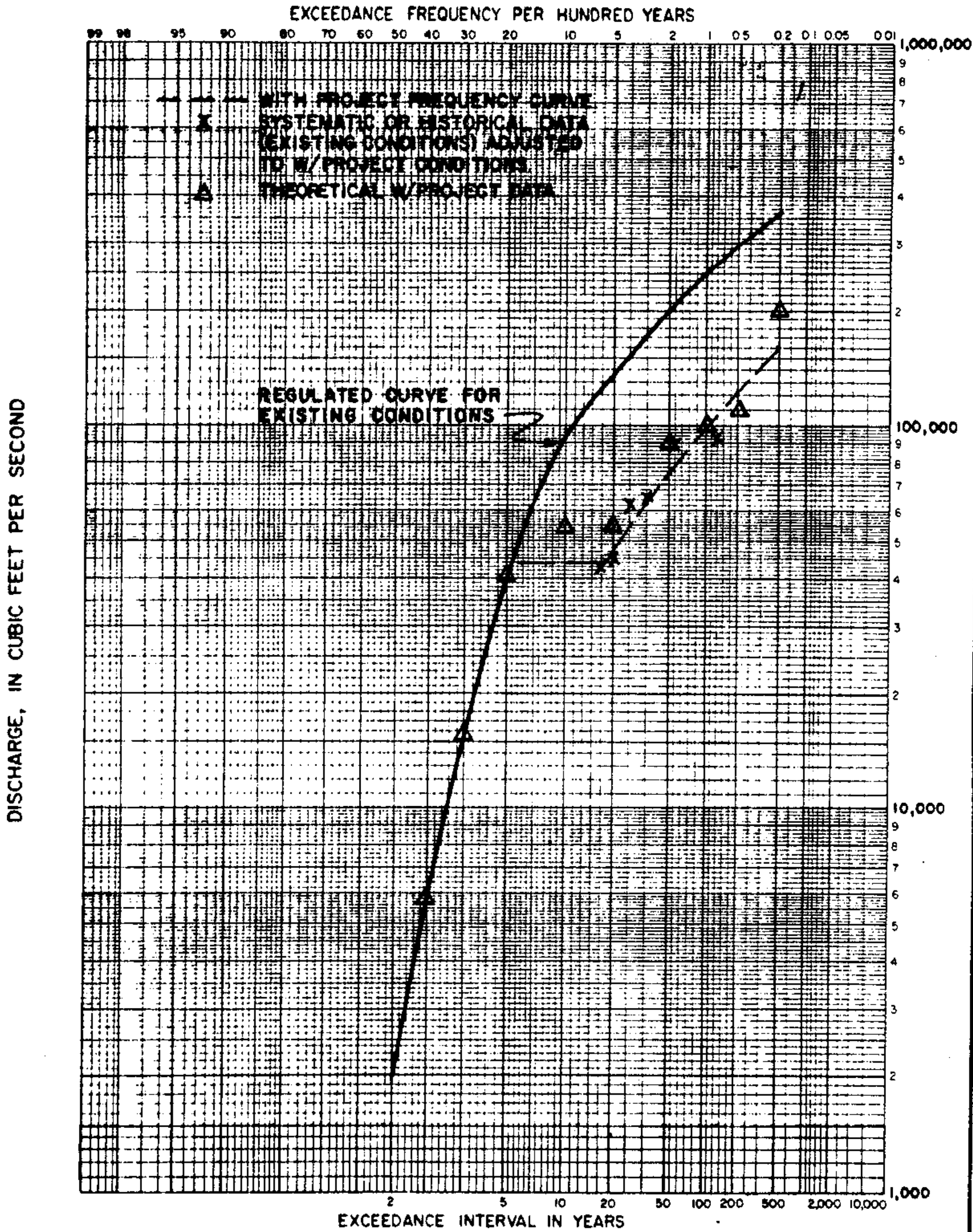
TARGET = 50,000 CFS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE - FREQUENCY CURVES

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP 113)

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



ALTERNATIVE = ORME

DESIGN = SPF

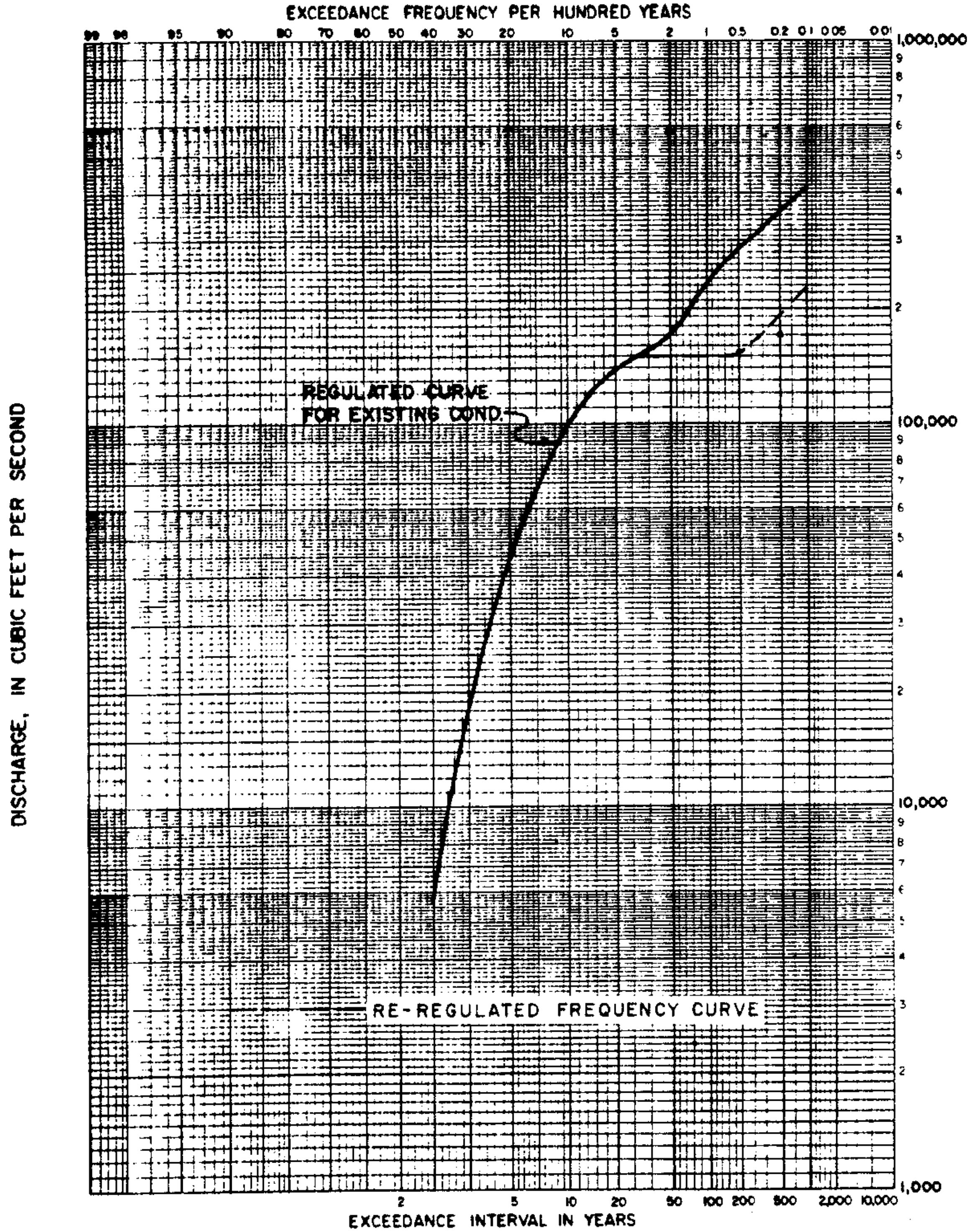
TARGET = 50,000 CFS

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE - FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED



ALTERNATIVE = RE-REGULATION OF SRP

DESIGN = SPF

TARGET = 150,000 CFS

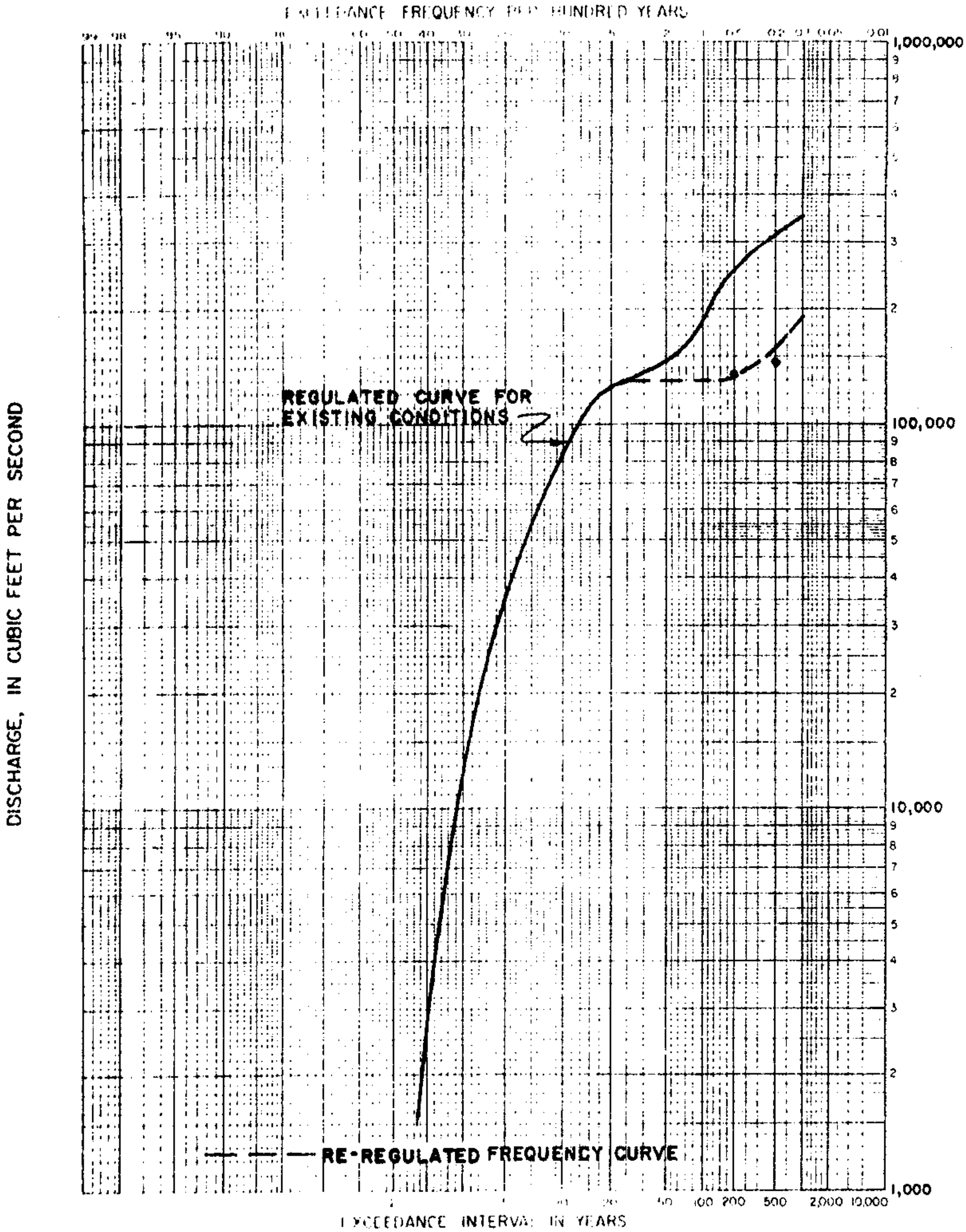
(W/F.C. OUTLETS)

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED



ALTERNATIVE = RE-REGULATION OF SRP

DESIGN = SPF

TARGET = 150,000 CFS

(W/F.C. OUTLETS)

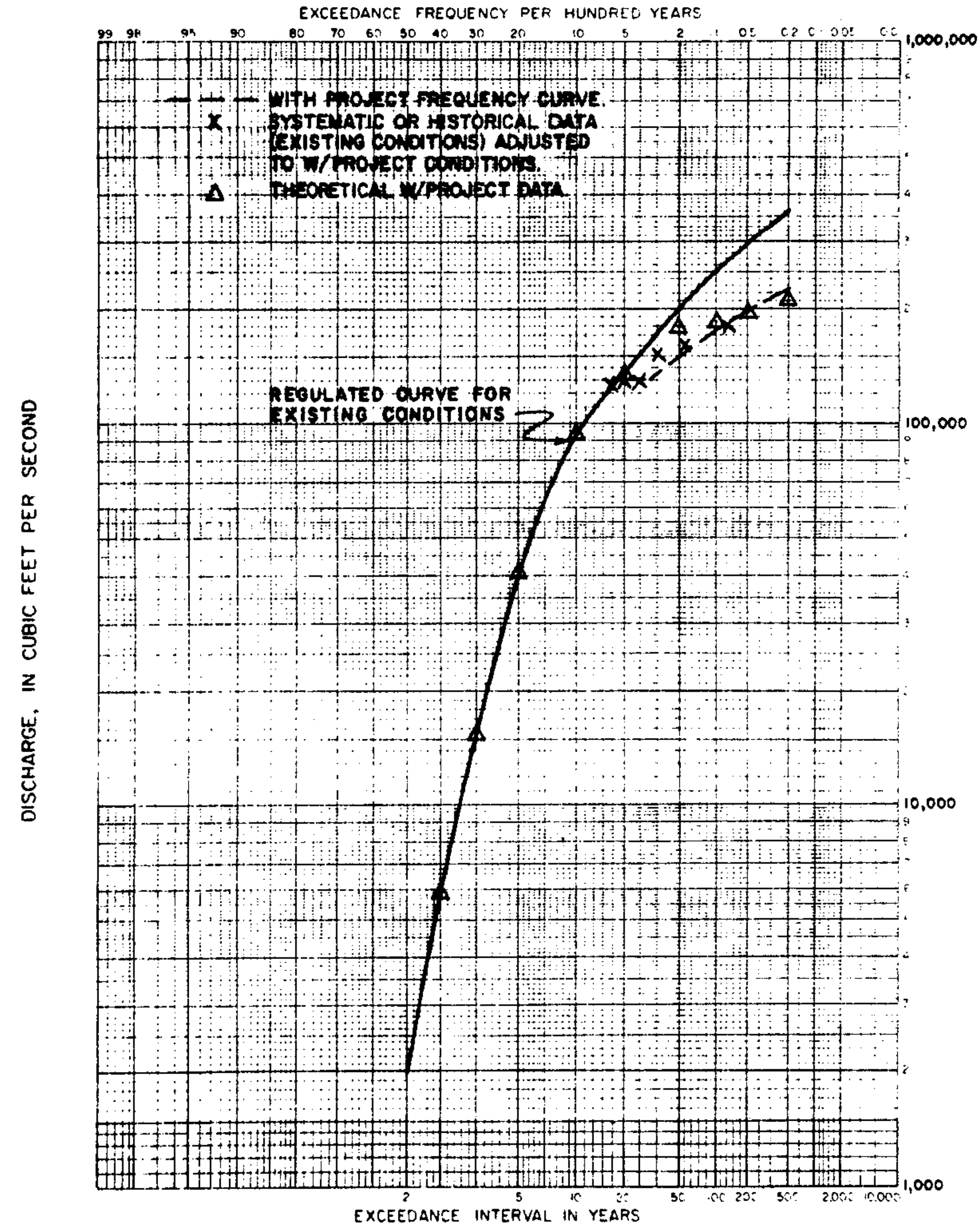
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE - FREQUENCY CURVES

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP 113)

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED

PLATE 2-3b



ALTERNATIVE = RE-REGULATION OF SRP

DESIGN = SPF

TARGET = 150,000 CFS

(W/F.C. OUTLETS)

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

DISCHARGE-FREQUENCY CURVES

GILA RIVER BELOW CONFLUENCE WITH SALT RIVER

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED

GILA RIVER AND TRIBUTARIES

CENTRAL ARIZONA WATER CONTROL STUDY

APPENDIX 3

BASIC DATA

FOR DEVELOPMENT OF EXISTING

CONDITIONS DISCHARGE FREQUENCY RELATIONSHIPS

FOR THE SALT-GILA RIVER SYSTEM

U.S. Army Engineer District, Los Angeles

Corps of Engineers

May 1982

Appendix 3. Basic Data for Development of Existing Discharge Frequency Relationship for the Salt-Gila River System

Table of Contents

Tables

Plates

Table 3-1

Monthly Inflow to Horseshoe Dam
Period: 1888-1980

Flow in CFS

YEAR	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL
1888	162	158	157	397	3159	2357	1106	3218	750	186	133	196
89	192	236	208	272	2682	1974	4287	2367	347	164	144	208
90	1796	1059	1306	2225	2953	1354	16481	1819	504	432	378	296
91	262	373	243	242	309	268	181	151	83	86	49	143
92	187	155	210	242	266	218	634	1128	274	142	64	212
93	757	500	355	279	297	230	244	500	161	64	75	112
94	414	275	228	217	417	3808	1592	3509	708	243	144	137
95	339	166	448	437	369	307	290	267	208	162	110	813
96	801	525	426	464	332	2017	824	1419	1122	254	142	123
97	414	936	288	247	252	239	468	603	301	174	131	305
98	377	319	159	184	286	330	325	245	193	143	143	344
99	409	337	518	209	205	178	188	151	83	86	49	49
1900	142	114	173	406	211	331	1754	844	175	132	99	198
01	592	88	126	231	253	213	235	232	208	174	110	81
02	451	997	136	196	416	236	342	1392	2590	133	128	219
03	310	484	300	195	214	224	213	174	112	119	59	688
04	1532	455	177	198	227	1339	7273	8283	4931	785	267	231
05	535	727	513	3238	825	766	1134	5152	971	233	142	221
06	701	199	171	294	2491	2292	2471	3554	791	237	197	205
07	406	381	579	354	305	289	1861	1316	284	418	138	436
08	830	336	250	265	2952	1660	1376	1914	1187	189	127	358
09	1184	448	151	208	334	3361	484	1215	792	132	61	118
10	295	208	183	308	287	2740	2372	2238	406	453	268	419
11	231	521	642	481	557	283	283	1382	1994	248	216	293
12	459	191	649	242	232	216	643	1699	1331	108	126	144
13	208	341	192	320	289	903	2873	676	237	145	108	192
14	221	217	308	255	615	1171	2309	3388	2057	2512	195	313
15	329	163	167	234	371	7765	3553	4891	657	219	150	190
16	479	1223	685	308	321	1153	1411	1660	5665	1182	221	393
17	686	367	233	229	251	372	853	4352	335	151	129	180
18	514	179	182	326	432	325	899	1472	1255	163	111	2006
19	855	444	699	2689	2104	2108	8449	1776	982	288	197	170
20	430	215	227	437	323	297	316	492	223	158	119	279
21	1599	346	417	272	1356	2447	2593	3093	1009	242	154	194
22	314	226	175	267	1159	327	1153	2082	749	182	109	188
23	240	1820	255	913	3302	938	338	493	1553	162	93	176
24	99	191	228	212	433	298	329	413	452	204	124	204
25	354	1128	779	341	368	298	293	720	4423	408	117	193
26	248	347	246	215	432	320	7080	2029	809	195	202	259
27	474	1829	279	297	424	480	1421	995	241	169	112	133
28	482	189	263	275	303	320	412	1368	2140	149	116	164
29	581	390	175	217	234	291	443	1341	545	175	99	312
30	624	283	225	514	356	229	3019	639	252	246	97	161
31	779	376	213	600	948	478	6454	3640	777	199	137	224
32	247	134	292	208	294	382	364	726	299	344	148	182
33	164	231	233	206	253	260	269	226	257	120	106	124
34	452	215	166	267	317	1046	2236	1971	1000	183	114	120

Table 3-1 (Continued)

Monthly Inflow to Horseshoe Dam
Period: 1888-1980

Flow in CFS

YEAR	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL
1935	592	485	238	247	278	265	866	969	692	150	93	223
36	434	318	211	312	291	461	6333	4076	1420	188	142	216
37	163	187	205	215	262	272	406	4715	223	128	99	138
38	290	203	167	205	375	291	336	861	322	124	89	89
39	309	1391	207	232	253	333	930	386	267	152	133	106
40	356	321	735	430	2167	1372	3387	4873	4671	665	207	239
41	268	328	525	392	429	540	371	990	654	246	124	132
42	173	148	186	230	274	429	640	1757	253	136	103	98
43	430	197	230	212	308	298	773	2621	1717	347	126	122
44	140	201	216	275	283	315	546	2150	1735	239	117	150
45	362	147	228	224	346	303	265	279	787	140	102	183
46	397	309	193	442	517	370	324	229	174	147	103	119
47	381	270	201	239	299	260	341	912	830	139	108	147
48	312	116	168	230	349	966	1190	2687	1642	195	173	185
49	210	283	550	262	300	308	974	509	188	134	101	310
50	194	152	155	205	227	249	262	261	207	263	95	95
51	1184	280	222	280	1730	2255	416	1865	2474	313	108	127
52	212	270	188	299	327	412	244	315	176	161	101	430
53	418	182	164	201	241	253	254	1949	625	130	100	299
54	340	253	198	215	239	285	268	362	179	135	316	332
55	841	142	183	221	284	250	243	205	186	117	87	205
56	189	99	158	211	231	1351	2239	589	175	201	175	194
57	322	136	208	973	269	232	900	2689	1181	156	129	76
58	254	604	269	244	239	227	378	331	170	133	94	228
59	546	130	640	315	1367	859	411	1873	246	155	109	90
60	175	237	224	229	232	224	225	232	412	122	85	133
61	269	340	161	220	296	336	1711	1250	841	125	92	90
62	127	179	190	192	246	250	249	223	155	113	83	85
63	793	405	177	255	244	240	220	315	1059	130	92	155
64	896	231	156	210	256	1563	941	1491	4328	265	130	163
65	228	337	166	1384	4613	1048	397	1715	276	167	121	147
66	317	261	251	264	2805	310	261	230	267	167	164	176
67	327	230	180	204	988	1200	2433	1230	600	271	144	160
68	337	154	230	265	286	2383	730	1862	863	200	127	148
69	210	269	183	269	247	260	236	843	300	166	126	207
70	309	1463	225	256	278	284	248	260	199	149	108	109
71	443	173	305	287	1350	384	247	194	180	139	146	160
72	244	158	4194	1089	1367	587	1614	3894	5638	1322	216	204
73	217	164	198	268	290	394	261	340	232	159	113	183
74	196	159	206	375	272	264	283	901	1329	223	127	175
75	143	189	177	226	274	254	2819	719	1113	274	127	202
76	167	262	246	249	275	314	258	226	228	168	125	128
77	245	207	262	212	243	624	2053	10420	646	210	128	120
78	216	167	198	1378	6322	2374	1479	3898	1866	321	198	168
1979	307	154	220	263	285	2706	10951					

Table 3-2

Monthly Inflow to Roosevelt Dam
Period: 1888-1980

Flow in CFS

YEAR	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL
1888	161	161	146	379	3014	3092	1303	4897	2862	790	296	257
89	192	240	194	259	2561	2591	5049	3596	1325	695	322	272
90	1787	1076	1218	2123	2817	1777	19410	2768	1922	1834	842	388
91	261	378	227	230	295	352	221	230	315	365	110	189
92	186	157	196	231	253	286	747	1718	1043	602	143	279
93	753	508	331	266	283	303	288	760	616	271	166	148
94	412	280	213	207	397	5390	1373	1738	1711	673	309	160
95	440	242	857	764	603	447	393	844	941	485	204	779
96	797	534	398	443	317	2647	970	2160	4281	1114	358	175
97	410	673	549	273	270	338	587	688	757	448	237	408
98	385	338	156	202	300	356	386	480	536	308	204	444
99	671	298	253	203	195	234	221	230	315	365	110	64
1900	142	116	161	387	202	454	2414	1423	1050	735	288	346
01	529	301	152	195	190	189	207	201	268	167	106	78
02	478	1057	131	189	441	207	318	600	909	352	285	142
03	411	316	253	211	208	221	215	217	148	132	80	356
04	1514	460	281	164	172	1611	8213	15300	12560	4606	1405	529
05	600	722	342	6395	1684	1474	1432	7770	5083	1694	667	514
06	869	466	300	275	4952	3259	2549	3709	1938	748	514	428
07	1300	1131	1322	880	466	388	3753	3677	1578	903	430	780
08	2066	1082	369	354	3615	1135	3417	2882	3722	513	642	442
09	1151	2368	458	1063	322	1606	604	1196	997	491	136	155
10	294	212	170	294	274	2158	2896	4357	1114	564	322	616
11	348	245	906	395	234	228	233	1898	2258	1139	415	368
12	548	448	423	288	293	274	559	1348	1859	592	228	231
13	270	369	230	386	449	577	2087	1260	1171	473	277	657
14	1163	804	713	561	3129	3068	4808	4476	6492	4204	1365	1584
15	659	421	344	405	437	19662	5017	9050	4683	1970	912	500
16	845	1107	1569	502	353	1876	1824	1362	3523	1366	552	576
17	555	354	260	283	285	339	600	2490	794	407	316	432
18	504	216	195	299	434	325	1717	2084	4078	1346	407	3711
19	1952	886	543	2663	5730	3251	11240	3502	2650	1824	705	338
20	725	337	339	606	375	364	428	455	359	317	241	835
21	4045	1172	371	276	488	1011	1845	3200	2263	1057	427	401
22	581	273	207	261	723	328	926	2378	1409	709	268	462
23	1031	2075	436	1216	4751	1681	612	1146	3566	1205	414	272
24	284	218	194	213	325	261	265	1184	729	287	217	301
25	672	1101	529	357	307	272	301	1511	6338	2534	507	369
26	399	419	293	275	608	392	6440	2707	2460	1450	734	413
27	555	973	244	243	293	273	985	903	703	553	273	341
28	483	284	286	325	289	330	447	899	2338	493	228	330
29	1231	1108	445	284	241	341	624	2116	1859	649	302	704
30	1075	289	176	434	310	212	3567	940	1569	1159	284	345
31	1167	1318	1182	974	1706	1068	8402	4110	3496	1309	474	549
32	939	550	341	251	339	395	696	1839	1273	1142	561	529
33	502	445	515	316	339	272	321	456	346	214	134	194
34	863	515	197	252	239	1226	2822	3217	3540	1204	710	247
35	687	611	236	268	270	246	2392	2291	3626	1221	382	265

Table 3-2 (Continued)

Monthly Inflow to Roosevelt Dam
Period: 1888-1980

Flow in CFS

YEAR	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL
1936	480	550	259	338	394	616	6019	4734	3855	1342	422	339
37	305	267	217	208	251	232	282	2772	909	475	212	249
38	682	471	165	173	278	273	523	1600	2108	557	167	139
39	420	321	202	239	219	276	699	987	1004	467	244	241
40	391	478	565	540	4158	4400	4096	10740	6301	5978	1529	766
41	705	645	807	562	897	1349	741	1529	2583	1063	332	240
42	456	338	296	284	406	1253	1147	3515	1841	675	233	180
43	450	370	279	235	248	251	716	1913	1670	780	288	211
44	288	414	315	293	299	359	706	2445	3003	1308	300	227
45	611	218	295	197	319	321	286	555	649	265	119	192
46	735	2157	343	670	809	456	532	667	544	378	139	113
47	528	751	774	300	410	320	445	1280	3485	886	229	233
48	325	132	148	200	629	2759	1575	2948	3129	1377	519	603
49	681	275	259	226	265	273	517	708	642	259	129	276
50	241	195	129	151	175	205	235	302	381	444	132	123
51	2327	335	186	262	1580	8124	875	3383	6211	2587	756	305
52	534	268	178	340	370	375	272	1620	690	401	227	364
53	287	114	111	161	178	197	216	2832	1131	410	145	429
54	764	340	159	158	171	207	205	278	243	174	242	467
55	2374	301	159	180	264	270	503	839	724	410	132	164
56	218	85	100	140	148	1882	1489	951	706	570	359	215
57	1207	386	293	497	288	220	972	4243	4546	2009	450	164
58	359	756	791	271	243	215	262	281	280	145	86	273
59	1704	260	1000	1578	3988	3860	1318	3766	1859	764	321	149
60	204	160	281	229	210	214	225	406	623	220	120	152
61	348	354	162	306	670	1349	2610	2352	4470	1307	343	210
62	173	267	291	271	253	293	1489	877	993	330	114	86
63	1821	1085	373	363	238	196	198	289	1134	417	154	349
64	773	919	328	217	264	2653	1829	2396	4373	1679	616	446
65	583	274	180	695	9345	2234	880	3649	2771	972	300	224
66	525	661	273	260	754	262	271	348	359	199	146	450
67	1512	764	238	222	2047	2983	4132	3531	3188	1507	514	344
68	845	262	265	246	271	2064	1016	1669	2689	1156	352	255
69	403	557	264	348	304	272	267	779	1046	741	226	208
70	398	1572	295	205	226	264	269	311	293	204	111	136
71	1072	604	2596	839	1853	756	381	546	287	167	224	167
72	202	239	6248	1800	2272	1290	3991	7777	7456	6715	1483	638
73	434	229	210	267	272	538	295	685	592	327	131	209
74	352	191	646	605	265	260	577	2594	3630	1938	528	380
75	203	377	169	194	269	223	2260	724	1498	994	271	342
76	328	310	228	210	199	275	260	282	719	362	157	246
77	462	361	267	224	185	660	3085	15713	3525	1164	360	228
78	409	209	221	2468	7887	5648	4153	7103	7477	3306	1432	458
79	478	258	235	335	302	2157	13653					

Table 3-3

Monthly Inflow to Coolidge Dam
Period: 1903-1928

Flow in CFS

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1903	169.	53.	36.	51.	2.	107.	53.	962.	223.	106.	55.	35.
04	32.	33.	11.	5.	9.	0.	143.	953	232.	1602.	852.	108.
05	306.	29.	26.	27.	2927.	255.	100.	441.	544.	130.	27.	3903.
06	1789.	2521.	1301.	4201.	2683.	208.	0.	1301.	17.	0.	27.	2033.
07	26.	29.	26.	27.	26.	39.	37.	260.	1277.	358.	27.	26.
08	26.	5042.	602.	27.	26.	39.	37.	2602.	319.	37.	27.	2846.
09	26.	29.	138.	27.	26.	39.	1138.	553.	193.	37.	27.	26.
10	1057.	29.	26.	27.	26.	39.	37.	37.	39.	37.	27.	26.
11	1203.	414.	1447.	239.	13.	39.	1464.	99.	207.	472.	302.	146.
12	146.	82.	2846.	471.	55.	0.	992.	293.	87.	26.	57.	125.
13	179.	549.	618.	244.	54.	1.	46.	91.	38.	50.	412.	350.
14	268.	266.	125.	29.	8.	72.	968.	1079.	606.	1172.	841.	8421.
15	3381.	5135.	3573.	3874.	1131.	193.	907.	500.	267.	67.	71.	222.
16	12632.	3288.	2866.	1077.	403.	57.	88.	788.	720.	3240.	442.	347.
17	1849.	964.	773.	482.	151.	35.	187.	221.	48.	28.	46.	98.
18	196.	224.	273.	38.	15.	32.	75.	330.	25.	35.	87.	350.
19	296.	652.	1099.	1791.	417.	28.	1812.	1641.	401.	343.	480.	1597.
20	1117.	3448.	898.	480.	145.	49.	48.	258.	70.	52.	178.	181.
21	196.	87.	57.	42.	13.	1.	1016.	2532.	295.	54.	58.	110.
22	279.	93.	60.	54.	14.	5.	78.	259.	18.	20.	25.	144.
23	69.	62.	179.	35.	5.	1.	575.	1987.	786.	62.	683.	1719.
24	1002.	268.	269.	1378.	207.	6.	11.	33.	1.	4.	10.	55.
25	76.	51.	51.	18.	8.	14.	42.	307.	1830.	283.	155.	248.
26	261.	93.	480.	2025.	659.	20.	56.	57.	99.	118.	210.	334.
27	215.	1251.	581.	114.	45.	11.	94.	229.	720.	63.	64.	193.
28	161.	15.	176.	49.	23.	1.	102.	532.	61.	36.	0.	

TABLE 3-4

CENTRAL ARIZONA WATER CONTROL STUDY
SALT RIVER AT ORME SITE, CP 40 (BELOW CONFLUENCE WITH VERDE RIVER)
REGULATED FLOWS (SPILL EVENTS) COMPUTED USING HEC-5 SIMULATION
(SRFOC MODEL), DISCHARGES IN CFS

STA	YEAR	PEAK	1-DAY	2-DAY	3-DAY	5-DAY	10-DAY	1-MONTH
40	1889	39630.	31100.	28400.	26000.	21600.	16500.	6820.
40	1890	145500.	95100.	94730.	65500.	39600.	20400.	6850.
40	1891	277100.	215600.	168500.	145600.	98400.	96400.	35260.
40	1895	7525.	7500.	6300.	5910.	5600.	4850.	2050.
40	1905	115000.	80750.	71800.	61750.	53000.	42800.	15900.
40	1906	131700.	96000.	80600.	62500.	38100.	20700.	11600.
40	1907	51790.	36700.	33160.	25840.	18940.	12750.	5970.
40	1908	38040.	26000.	25800.	18800.	13300.	9060.	3700.
40	1909	67200.	51700.	41100.	34460.	22060.	11800.	4160.
40	1914	22310.	15500.	14300.	13900.	10940.	7010.	2260.
40	1915	16550.	16550.	16150.	15700.	15100.	13400.	7000.
40	1916	145900.	123900.	117400.	104700.	79940.	47900.	24500.
40	1917	58680.	40900.	35800.	30600.	22800.	18800.	7640.
40	1918	37080.	24800.	21500.	17400.	12700.	11980.	3020.
40	1919	11350.	6020.	5910.	5770.	5060.	3540.	860.
40	1920	138900.	112300.	112160.	89500.	66700.	39200.	19100.
40	1922	30870.	22200.	19700.	16475.	13400.	11100.	3660.
40	1923	5420.	5420.	5200.	4380.	4160.	3650.	860.
40	1924	91180.	61940.	53700.	41270.	32400.	20900.	4210.
40	1926	10510.	8040.	7870.	7450.	5960.	5030.	460.
40	1927	83160.	61600.	61200.	51400.	37900.	21900.	74100.
40	1928	10910.	7200.	6425.	5760.	4460.	3000.	810.
40	1931	10460.	10420.	9330.	7480.	4960.	4260.	850.
40	1932	86720.	81450.	63960.	54460.	40200.	24100.	10350.
40	1936	6180.	6180.	5450.	5530.	5450.	4835.	3000.
40	1937	54250.	39000.	37200.	31600.	25400.	18900.	7510.
40	1938	74650.	48600.	44900.	32800.	21600.	11100.	2640.
40	1941	131700.	110000.	95400.	80600.	55300.	33000.	12700.
40	1942	9000.	9000.	6160.	5200.	4200.	4000.	2420.
40	1943	5130.	5100.	3230.	3330.	3040.	2820.	1680.
40	1952	12600.	12600.	10800.	7400.	5300.	4930.	1410.
40	1965	9040.	9040.	8260.	7510.	6560.	5300.	280.
40	1966	49320.	36940.	28540.	24900.	16600.	13300.	4210.
40	1967	33530.	13100.	12500.	11300.	8990.	5240.	1150.
40	1968	16900.	12600.	12060.	10240.	8690.	7070.	2960.
40	1969	8300.	8300.	7650.	6430.	5700.	5230.	2080.
40	1973	27700.	23300.	22300.	20760.	19100.	16450.	11400.
40	1978	116000.	61450.	51200.	42900.	35300.	22800.	10250.
40	1979	158700.	140500.	107600.	91300.	58300.	31900.	9570.
40	1980	201200.	167280.	124370.	93910.	79790.	65580.	25580.

TABLE 3-5

CENTRAL ARIZONA WATER CONTROL STUDY
SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER CP 113
REGULATED FLOWS (SPILL EVENTS) COMPUTED HEC-5 SIMULATION
(SRFOC MODEL), DISCHARGES IN CFS

STA	YEAR	PEAK	1-DAY	2-DAY	3-DAY	5-DAY	10-DAY
113	1889	32950.	27000.	24500.	22300.	18145.	12880.
113	1890	123900.	99200.	77250.	65300.	39700.	20000.
113	1891	255500.	181140.	168100.	132400.	92560.	92445.
113	1895	3800.	3800.	3200.	2600.	2270.	1500.
113	1905	104700.	73100.	67600.	57200.	47150.	39400.
113	1906	112400.	81600.	74700.	58660.	35900	18500.
113	1907	42070.	29800.	27700.	22400.	15300.	9355.
113	1908	29230.	23500.	19740.	15750.	9770.	5470.
113	1909	59610.	39300.	38500.	29000.	18800.	9650.
113	1914	16940.	11600.	11400.	9870.	7450.	4110.
113	1915	13000.	13000.	12500.	12300.	11750.	9700.
113	1916	129700.	117500.	110000.	99000.	75900.	44200.
113	1917	47960.	35400.	30800.	26750.	18800.	15100.
113	1918	29180.	19100.	17000.	14400.	9200.	8500.
113	1919	6365.	2670.	2630.	2400.	1680.	840.
113	1920	125400.	118600.	97600.	89750.	61700.	35740.
113	1922	25360.	17850.	15255.	13300.	10100.	7590.
113	1923	2040.	2040.	1375.	1110.	1075.	630.
113	1924	76260.	56250.	45100.	37940.	26900.	16200.
113	1926	7050.	4800.	4570.	3750.	2630.	1810.
113	1927	75130.	62500.	53300.	49200.	34400.	18350.
113	1928	6035.	3360.	3100.	2070.	1240.	830.
113	1931	7030.	7010.	5740.	3945.	2490.	1540.
113	1932	81830.	69100.	62800.	50050.	34800.	20200.
113	1936	2650.	2650.	2200.	2090.	2050.	1620.
113	1937	43760.	35000.	32000.	28600.	22100.	15500.
113	1938	59710.	43300.	37540.	29800.	18100.	9260.
113	1941	117900.	97100.	90700.	74000.	51500.	29600.
113	1942	3620.	3620.	3040.	2030.	1700.	1260.
113	1943	1760.	1760.	995.	660.	400.	200.
113	1952	7300.	7300.	4400.	2930.	1900.	1610.
113	1965	5760.	5760.	4560.	4130.	3210.	2000.
113	1966	41490.	24200.	19200.	16000.	10900.	8400.
113	1967	21220.	9900.	8885.	7900.	5500.	2840.
113	1968	12170.	9140.	8180.	6320.	5060.	3670.
113	1969	3940.	3940.	3330.	2820.	2300.	1820.
113	1973	21800.	19200.	18600.	17400.	15600.	13000.
113	1978	94510.	53500.	46800.	39800.	32300.	19400.
113	1979	137500.	122240.	108900.	83300.	54600.	29400.
113	1980	194910.	161900.	119910.	89810.	75830.	61870.

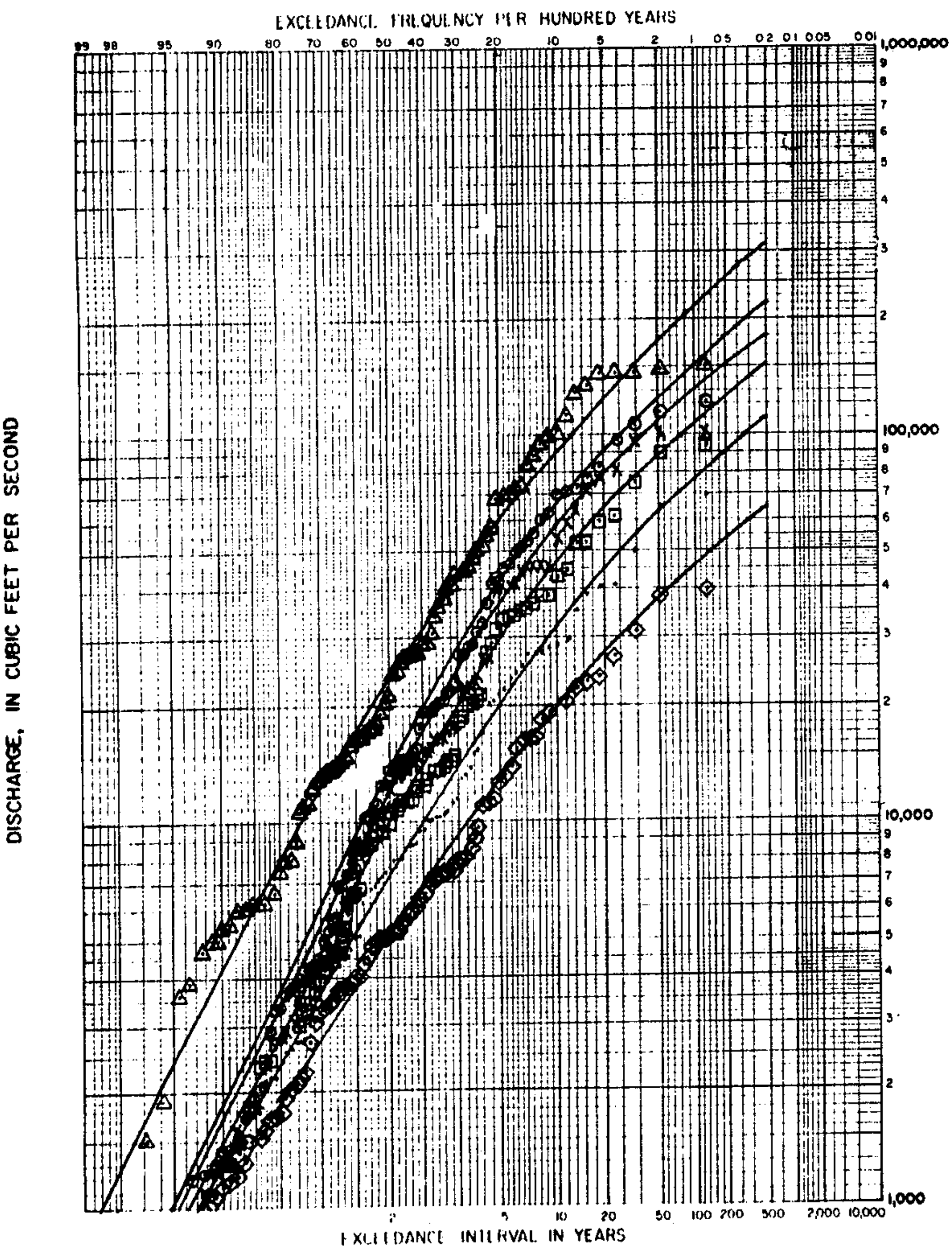
TABLE 3-6

Period of Record
Gila River

Date	Below Confl w/Salt CP 1310	Below Confl w/Waterman CP 1216	Below Confl w/Hassayampa CP 1217	At Gillespie Dam CP 1218	Midway from Gillespie to Painted Rock CP 12191	At Painted Rock Dam CP 1219
3-18-1889	32300.	29100.	26700.	25500.	24200.	22600.
2-23-1890	122900.	110800.	99700.	96200.	96500.	83600.
2-24-1891	300000.	295000.	290000.	285000.	280000.	27500.
March 1895	3800.	3500.	3200.	2900.	2600.	2200.
4-13-1905	103800.	96300.	89700.	88200.	85600.	77900.
11-28-1905	160000.	155000.	150000.	145000.	140000.	135000.
3-7-1907	41400.	33500.	28400.	27200.	25600.	22700.
2-5-1908	30600.	27500.	25700.	24600.	23300.	22500.
12-17-1908	78000.	73800.	71500.	70500.	69200.	67600.
2-23-1914	16300.	13600.	11600.	10400.	9200.	7100.
1-29-1915	22200.	46700.	26100.	25200.	22200.	17200.
1-20-1916	150800.	184100.	178900.	177900.	176000.	171300.
4-19-1917	47300.	40100.	35600.	34500.	32800.	30500.
3-10-1918	28500.	23300.	19200.	18300.	16800.	14600.
2-23-1920	126300.	121700.	118600.	117300.	115300.	111200.
8-22-1921	15500.	27800.	27000.	26800.	25800.	24500.
3-19-1922	24700.	21300.	18400.	17200.	15900.	13900.
9-20-1923	1800.	14200.	13400.	56400.	12100.	10800.
12-29-1923	75500.	65200.	57500.	13100.	53800.	46600.
9-20-1925	3600.	16300.	15500.	15200.	14200.	12900.
9-30-1926	36600.	39400.	38600.	38300.	37300.	36000.
2-18-1927	74400.	71300.	69900.	68700.	67200.	63800.
8-3-1928	5400.	6700.	5900.	5600.	4600.	3300.
9-26-1929	5400.	6300.	5500.	5210.	4200.	2900.
8-10-1930	18400.	15000.	14200.	13900.	12900.	11600.
8-12-1931	6400.	8600.	7800.	7530.	6500.	5200.
2-11-1932	81000.	76700.	71900.	71000.	70100.	63300.
10-9-1932	300.	3300.	2500.	2180.	1200.	600.
8-30-1934	1400.	4200.	3400.	3100.	2100.	800.
8-25-1935	4800.	3500.	2700.	2380.	1400.	100.
7-29-1936	2600.	4300.	3500.	3240.	2600.	2600.
3-18-1937	43100.	37800.	33500.	31700.	29400.	22300.
3-4-1938	59000.	45800.	37100.	35800.	34400.	29400.
9-13-1939	900.	4300.	3500.	3240.	2200.	1000.
8-19-1940	11100.	3700.	2900.	2620.	1600.	400.
3-15-1941	117000.	111000.	105900.	104800.	103600.	96300.
April 1942	3500.	3500.	3400.	3400.	3300.	3200.
8-5-1943	1900.	3300.	2500.	2200.	1400.	1300.
8-14-1945	5200.	2400.	1600.	1350.	400.	200.
9-19-1946	500.	5400.	4600.	4290.	3300.	2100.
8-9-1947	2600.	5500.	4700.	4390.	3400.	2200.
10-19-1949	200.	2600.	1800.	1460.	500.	200.

Table 3-6 (Continued)

Date	Below Confl w/Salt CP 1310	Below Confl w/Waterman CP 1216	Below Confl w/Hassayampa CP 1217	At Guillespie Dam CP 1218	Midway from Gillespie to Painted Rock CP 12191	At Painted Rock Dam CP 1219
8-28-1951	2000.	17700.	16900.	16600.	15600.	14400.
April 1952	6500.	5700.	4900.	4100.	3300.	2600.
8-12-1954	800.	2900.	2100.	1760.	800.	400.
8-28-1955	2300.	4800.	4000.	3660.	2700.	1500.
April 1965	5200.	4700.	4100.	3600.	3100.	2500.
12-31-1966	40800.	35400.	31600.	30400.	28700.	26400.
12-8-1967	20600.	14500.	11600.	10300.	9100.	7500.
3-11-1968	11500.	4200.	7900.	6900.	5600.	4900.
April 1969	3700.	3500.	3300.	3100.	2900.	2700.
9-6-70	1200.	7300.	6500.	6180.	5200.	4000.
8-27-1971	200.	2200.	1400.	1090.	100.	50.
3-18-1973	21100.	19200.	18000.	16900.	15600.	14900.
9-27-1976	400.	3000.	2200.	1920.	900.	400.
3-3-1978	93900.	89400.	87400.	85600.	83400.	81100.
12-19-1978	136046.	133500.	129500.	127000.	125500.	117300.
2-15-1980	193700.	202000.	183000.	180700.	175300.	157100.



LEGEND

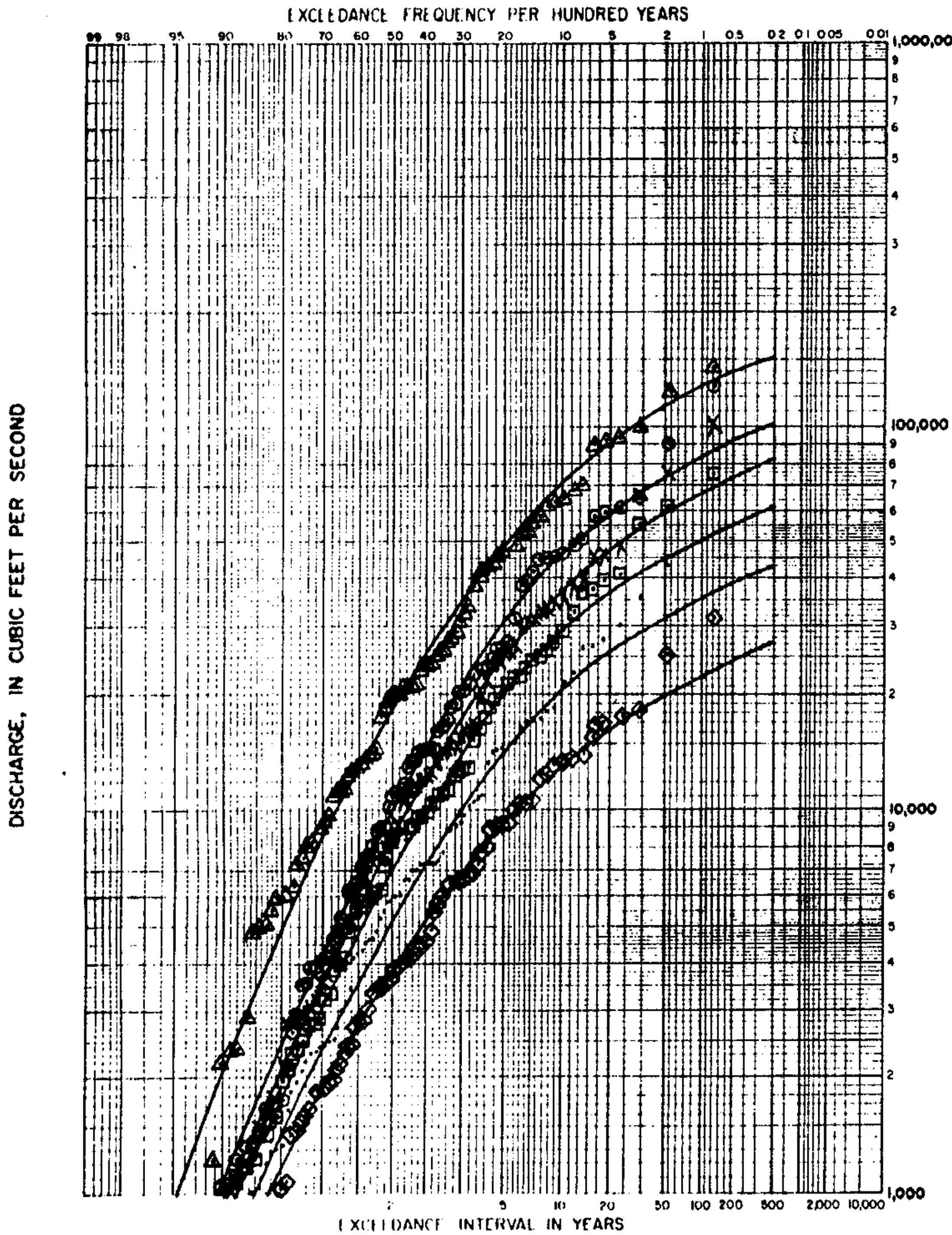
- △ PEAK
- ⊙ 1-DAY
- X 2-DAY
- ⊡ 3-DAY
- • 5-DAY
- ◈ 10-DAY

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY
(EXISTING CONDITIONS)

SALT RIVER COINCIDENT COMPONENT
INFLOW TO ROOSEVELT DAM

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



LEGEND

- △ PEAK
- ⊙ 1-DAY
- X 2-DAY
- ⊡ 3-DAY
- • 5-DAY
- ◇ 10-DAY

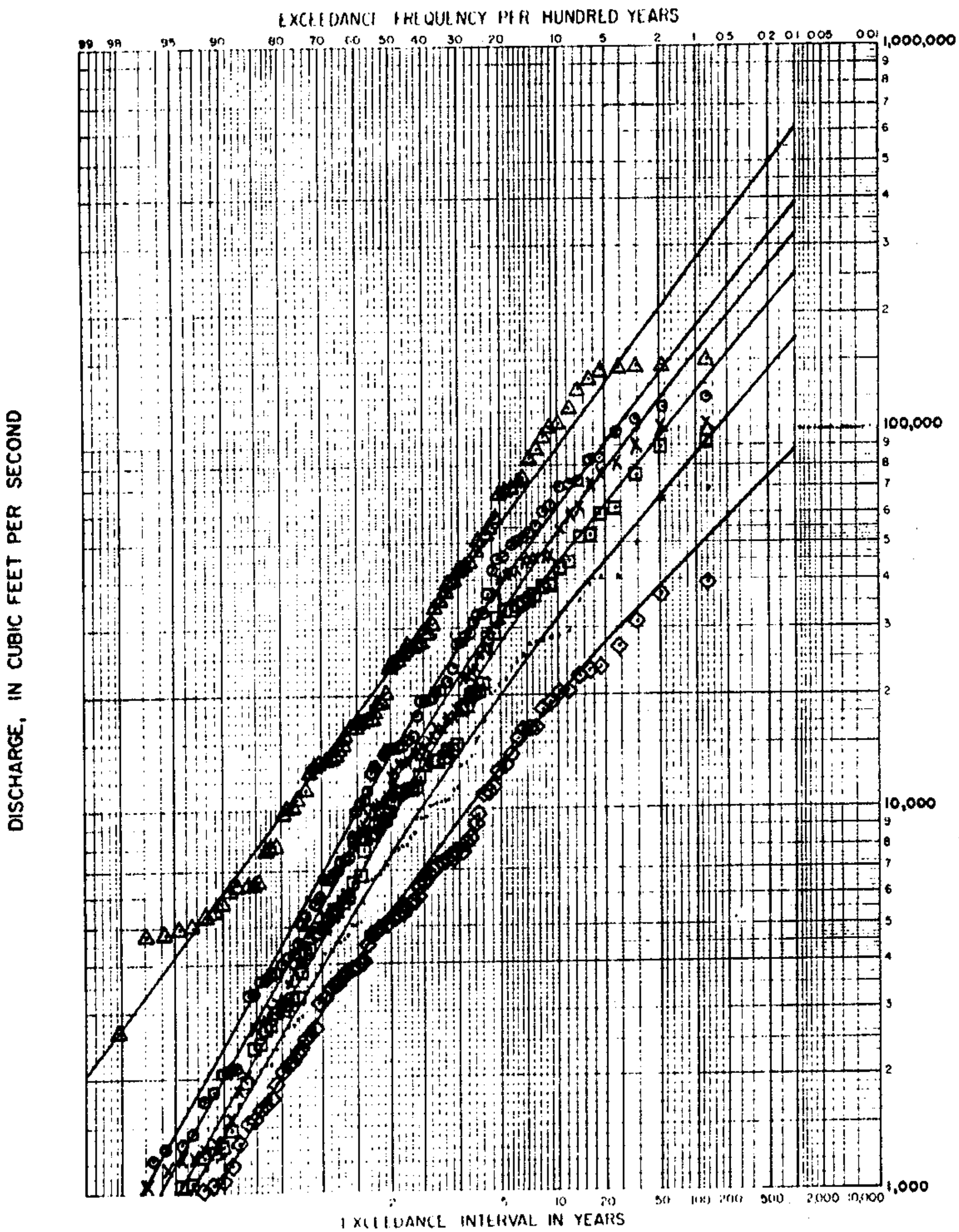
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY
(EXISTING CONDITIONS)

VERDE RIVER COINCIDENT COMPONENT INFLOW TO HORSESHOE DAM

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

PLATE 3-2



LEGEND

- △ PEAK
- ⊙ 1-DAY
- X 2-DAY
- ⊡ 3-DAY
- • 5-DAY
- ◇ 10-DAY

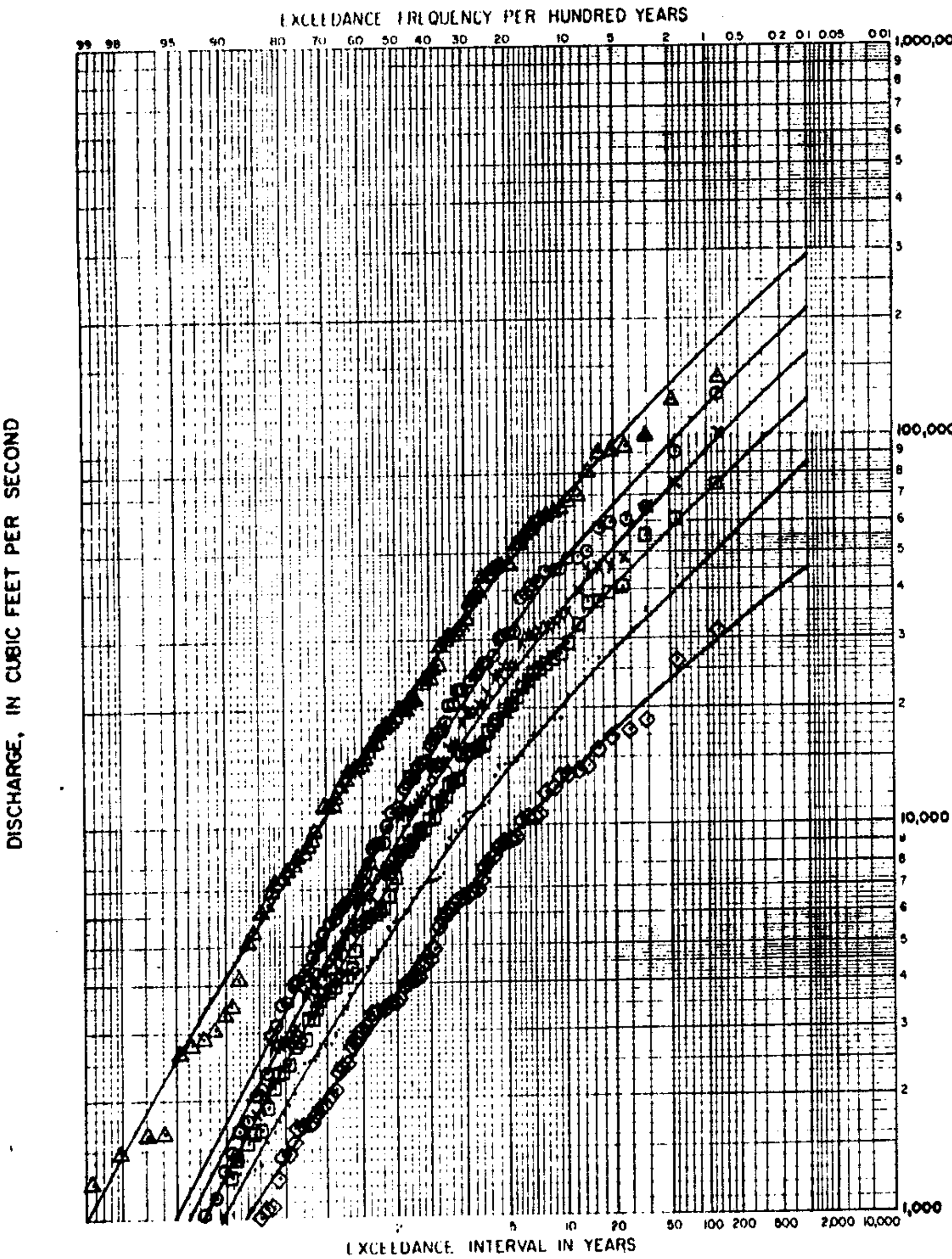
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY

SALT RIVER INFLOW TO ROOSEVELT DAM

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED

PLATE 3-3



LEGEND

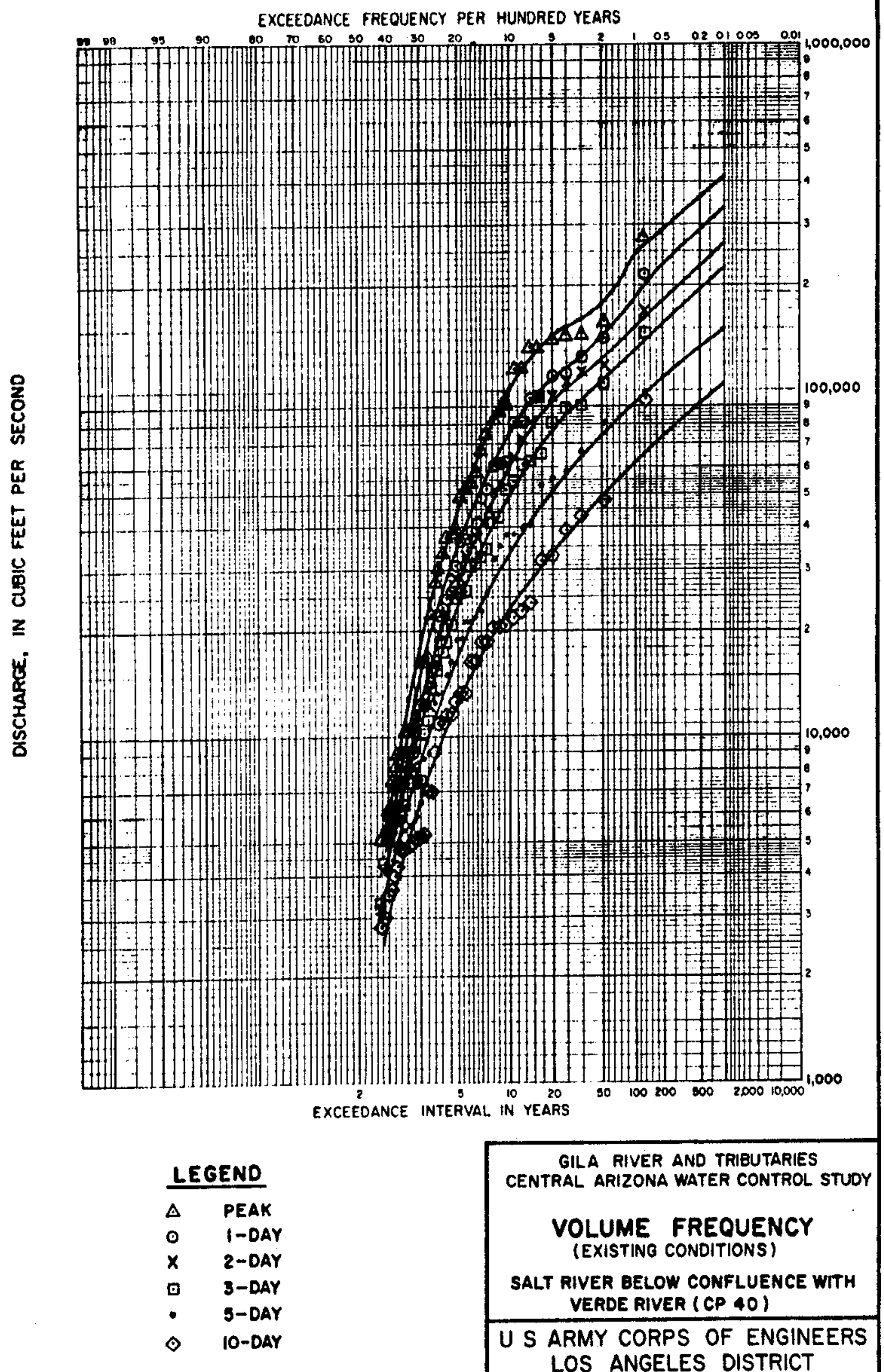
- △ PEAK
- ⊙ 1-DAY
- X 2-DAY
- ⊡ 3-DAY
- • 5-DAY
- ◇ 10-DAY

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY

VERDE RIVER INFLOW TO HORSESHOE DAM

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

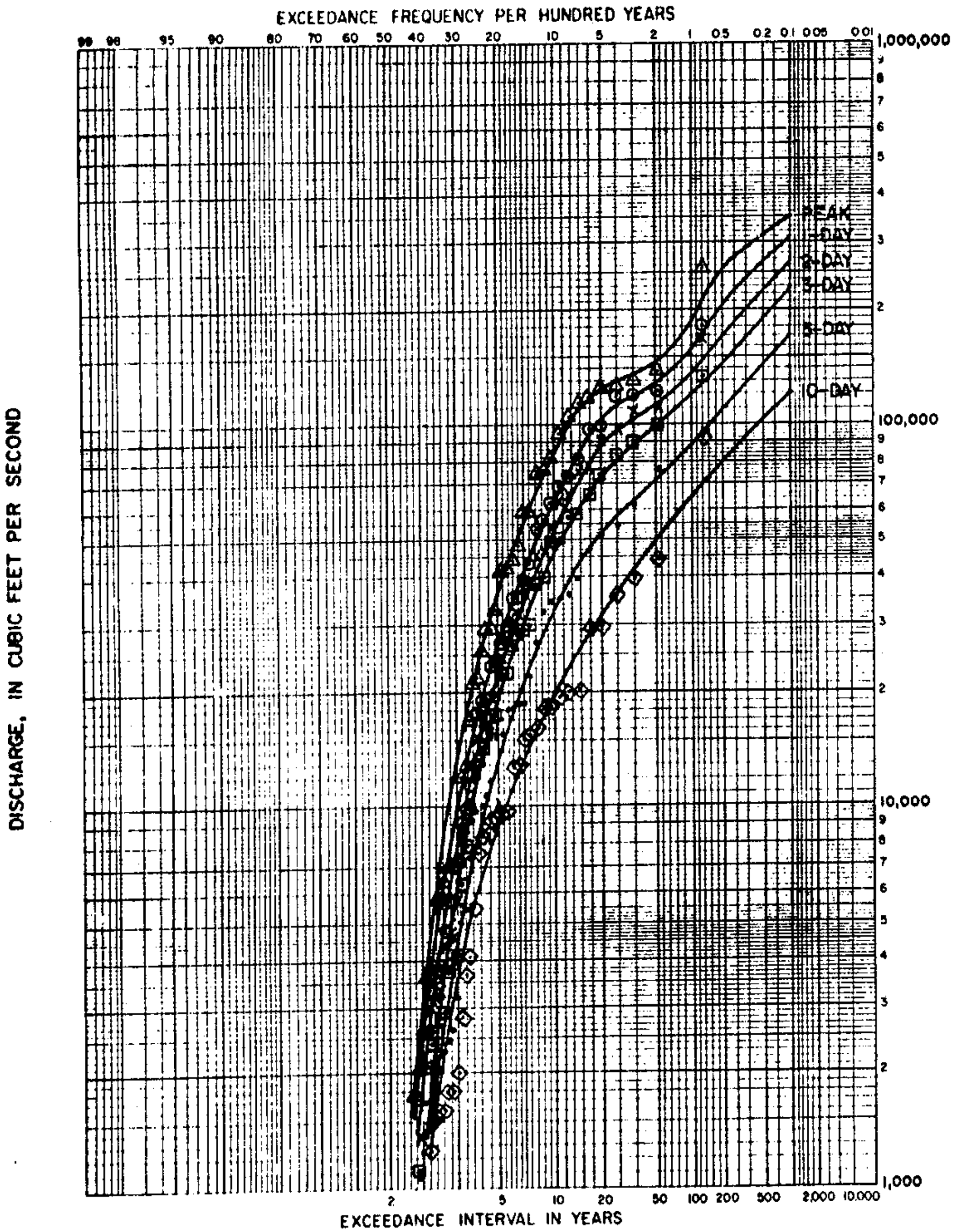


GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY
(EXISTING CONDITIONS)

SALT RIVER BELOW CONFLUENCE WITH VERDE RIVER (CP 40)

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



LEGEND

- Δ PEAK
- ⊙ 1-DAY
- X 2-DAY
- ⊡ 3-DAY
- • 5-DAY
- ◇ 10-DAY

GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

VOLUME FREQUENCY
(EXISTING CONDITIONS)

SALT RIVER ABOVE CONFLUENCE WITH GILA RIVER (CP 113)

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