OLD CROSS CUT
PHOENIX, ARIZONA

HYDROLOGY FOR FEASIBILITY STUDIES FOR FLOOD CONTROL AND ALLIED PURPOSES

U.S. Army Corps of Engineers

Los Angeles District

June 1987

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CESPD-ED-W (29 Jun 87/CESPL-ED-H) (1110-2-1403a) Verke/dh/6-6957
SUBJECT: Old Cross Cut Canal Feasibility Study

DA, South Pacific Division, Corps of Engineers, 630 Sansome St., Room 720, San Francisco, CA 94111-2206

03 SEP 1987

FOR: Commander, Los Angeles District, ATTN: CESPL-ED-H

The hydrology for Feasibility Studies for Flood Control and Allied Purpose, Old Cross Cut Canal, Phoenix, Arizona is approved. Plates 20 through 26 are out of sequence with the text and should be corrected for the final report.

FOR THE COMMANDER:

wd encls

A. E. WANKET
Chief, Engineering Division

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT CORPS OF ENGINEERS

CESPL-ED-H (1110-2-1403a) June 29, 1987

MEMORANDUM FOR: Commander, South Pacific Division, ATTN: CESPD-ED-W

SUBJECT: Old Cross Cut Canal Feasibility Study

1. Request approval of discharge-frequency values, (tables 1,6,7 and 8) in enclosed report titled, "Old Cross Cut Phoenix, Arizona, Hydrology for Feasibility Studies", by 13 July 1987 to meet Feasibility Milestone F3 requirements. Milestone F3 Conference #1 is currently scheduled for 14 July 1987. The hydrology report previously submitted for approval in September 1986 is superseded by enclosure 1 and should be disregarded.

2. Additional supporting data for Milestone F3 Conference #1 are also provided as follows:

a) Hydrology (encl. 2) and Hydraulics (encl. 3) responses to pertinent comments made at the 17 November 1986, In Progress Review meeting.

b) The without-project overflow map (encl. 4)

3. For further information please contact Joseph Evelyn, 8-798-5520.

FOR THE COMMANDER:

Robert C. Koper

4 Encls

CARL F. ENSON
Chief, Engineering Division

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1. INTRODUCTION

1.01 Purpose and Scope.

General. This report presents development of present and future condition hydrology for the Old Cross Cut Canal study area. N-year peak discharges are presented in Table 1. It also discusses seven alternatives to the flood problem and their hydrologic implications. This study area (plate 1), is between the existing Indian Bend Wash project and reach 4 of the Arizona Canal Diversion Channel.

Flood Problem. The Old Cross Cut Canal is part of the Salt River Project's (SRP's) canal system. It was originally built to transfer water from the Arizona Canal to the Grand Canal. Today it serves primarily to drain floodwaters intercepted by the Arizona Canal and local runoff downstream from the Arizona Canal, to the Salt River, plate 1. Runoff from Camelback Mountain results in sheet-flow-type flooding and ponding behind the north levee of the Arizona Canal. For the floods considered in this report (10-year floods and larger) the runoff overtops the north levee at the various locations and is partially intercepted by the Arizona Canal. During such events the Old Cross Cut Canal is used to drain the floodflows from the Arizona Canal. Up to 1000 ft^3/s can be diverted into the Old Cross Cut Canal at 48th Street. Flood waters eventually fill the Arizona Canal and overflow its south bank. Floodflows then disperse into sheet-flow, traveling through developed areas. At downstream locations some of the sheet-flow will be intercepted by the Old Cross Cut Canal. The rest will continue to flow southwestward toward the Grand Canal.

Planning Studies. Modifications of the existing drainage system are under consideration as a means to reduce ponding and sheet flow in the Phoenix area. This report analyzes the no-project conditions for the study area as well as seven alternative projects. When planning studies develop specific plans of action, additional hydrology studies may be required to analyze project conditions in more detail. For this study, peak discharges and total storm volumes were computed for various flood frequencies and the standard project flood.

1.02 Coordination With Local Interests.

The canals in this project area may be emptied at any time through the use of gates, wasteways, and diversion canals which are operated by the Salt River Project (SRP). Several alternatives in this study consist of such operations by SRP and as such would require their action to implement them. They have participated in the formulation of study alternatives and are willing to consider implementing one of these alternatives as it may benefit their interests.

2. GENERAL DESCRIPTION OF DRAINAGE AREA

2.01 Physiography and Topography.

This study area of approximately 17 square miles is located in the Phoenix region. About 20 percent of the area is mountainous with the remaining area being valley. Camelback Mountain, 2700 feet at its peak, is rugged and steep with a slope of about 60 percent. Papago Park Mountain is also rugged but has a slope of about 5 percent. The valley region, which dominates this area, is densely populated and very flat, about 1 percent slope. Land in the valley areas originally covered by natural vegetation, as well as that used for agriculture, is almost all now urbanized. Camelback Mountain is too steep for intensive development but is experiencing limited residential building.

2.02 Runoff Characteristics.

Phoenix is located within a desert region of central Arizona. Most of the areas addressed in this report are subject to flooding from two distinct types of topography: gently sloping valley areas, and steep hills. Runoff tends to not concentrate but rather flow downhill at somewhat equal depths across an entire valley area. Valley slopes range from 30 to 50 feet per mile in most cases. Some of the basin is in a state of rapid transition from natural land to residential, commercial and industrial development. Most of it has already been developed.

The study area also includes steep terrain. Camelback Mountain peak elevation is about 2700 feet with a slope of about 3000 feet per mile. Camelback Mountain runoff concentrates in numerous small gullies rather than one major water course. Upon reaching the valley area, runoff again disperses into sheet flow.

Flow paths in the valley area are controlled by the slope of the land and manmade obstructions. When the path of flow is interrupted by embankments such as those for railroads, highways and canals, ponding and diversion may occur. Drainage boundaries at several locations for this study are defined by such embankments.

2.03 Vegetation.

This study area is mainly urbanized flat valley. The residential areas have either grass or rock as ground cover in their yards while the business areas are mostly asphalt or concrete with a high concentration of buildings. Most streets are paved while the residential alleys are not. About 20 percent of the study area is unurbanized. Camelback and Papago Mountain, where natural vegetation is sparse, are rugged, steep and undeveloped. Cacti and other desert shrubs as well as a few stunted trees, including juniper, paloverde, mesquite, ironwood and scrub oak, exist among the shrubs. Growth tends to be thicker along and adjacent to the small gullies and washes. Perennial grasses form a very small portion of the vegetation, but a good cover of annual grasses occur after the winter rains.

2.04 Geology and Soils.

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

2.05 Land Use.

Much of the land in the Old Cross Cut-Arcadia area is presently devoted to residential use. The Papago Mountains have been designated as a regional park (Papago Park) and future development is not anticipated. Present condition land use was estimated from 1982 photo revised US Geological Survey (USGS) quadrangle sheets (1:24,000 scale) for each drainage area. This information was supplemented by field surveys and photographs. Projected future development was based on the General Plan for Phoenix - 1885/2000 map prepared by the City of Phoenix to show ultimate development.

2.06 Climatology.

The climate of Phoenix and the study area is arid. Annual precipitation is about 8 inches in the study area. Most of the precipitation occurs in two distinct seasons, summer (June through September) and winter (December through March), and is about equally divided between them. Monthly, seasonal, and annual precipitation amounts vary considerably from year to year. During any season there may be many successive rainless days. Three basic types of storms can affect the Phoenix area, although some may consist of a combination of types.

General Winter Storms. Storms of this type normally move inland from the north Pacific Ocean, spreading general light to moderate precipitation over large areas. Although they can occur any time from late October through May, they are most common and generally heaviest from December through early March. These storms frequently last several days and may occur in series with only slight breaks between storms. They usually reflect orographic effects to a great degree, so the mountains of central Arizona often receive from four to ten times as much precipitation from winter storms as do the desert areas near Phoenix. Snow frequently falls in the mountains above 6000 feet and occasionally falls at elevations below 3000 feet. Despite the normal low intensities of precipitation during general winter storms, the large areal extent and the relatively long duration of these storms, sometimes combined with snowmelt from the mountains, can produce substantial volumes of runoff and high peak discharges on the larger rivers of the region.

General Summer Storms. Storms of this type normally result from a flow of warm and very moist tropical air into the region from the southeast or south, including the Gulf of California (Sea of Cortez), the tropical Pacific Ocean south of Baja California, and, to a slight extent, the Gulf of Mexico. Such storms over Arizona are often associated with tropical storms or hurricanes. General summer storms can occur any time from late June through mid-October, but are most frequent from August through early October. They usually last from 1 to 3 days and generally consist of numerous locally heavy storm cells embedded in more widespread, general light to moderate rain. Like their general winter counterparts they usually reflect orographic influence, with higher mountains often receiving from three to eight times as much precipitation as do most of the desert areas. Some of the late September and October general storms can show characteristics of both the summer and winter types. The areal extent and duration of general summer storms are usually somewhat less than those of general winter storms, but intensities may be higher.

Because infiltration rates are normally higher during summer than during winter, runoff volumes are usually lower than from winter events, but the peak flows on intermediate-sized streams may be higher.

Local Storms. Local storms consist of heavy downpours of rain over relatively small areas (up to about 300 square miles) for short periods of time (up to about 7 hours). They are usually accompanied by lightning and thunder, and are often referred to as "thunderstorms" or "cloudbursts." They can occur any time of the year, but are most prevalent and most intense during the summer months, July to September, when tropical moisture frequently invades Arizona from out of the south or southeast. During the latter part of the summer season they are often larger, of longer duration, and more apt to be associated with general summer storms. Runoff from local storms is usually of a high-peak, low-volume type, affecting mostly the smaller creeks and washes, and is characterized by a rapidly rising and receding hydrograph. They can result in serious flash floods, sometimes with loss of life and serious local property damage.

2.07 Existing Structures Affecting Runoff.

General. Several existing bridges and canals alter flow characteristics in the study area and were considered in this study. Pertinent information on major existing and proposed structures affecting runoff is provided in this section.

Arizona Canal. The Arizona Canal is a partially entrenched water supply canal which carries water between Granite Reef Dam and Skunk Creek. During storms, water ponds behind the north bank causing flooding. If flows in the canal exceed capacity, the south bank breaks causing flooding downstream of it. For this reason it has several wasteway structures, such as 40th Street spillway and 48th Street gates, to allow for water disposal. This canal also has diversion structures to provide water to customers. One such diversion is the New (or Arizona) Cross Cut Canal. Starting at the Arizona Canal and Invergordon Road, it brings water to the Penstock treatment plant at a rate of up to 625 ft^3/s. These structures were accounted for in the without and with project analyses.

Old Cross Cut Canal. The Old Cross Cut Canal connects the Arizona Canal to the Grand Canal. This entrenched canal runs parallel to 48th Street with a westward jog at McDowell Road (plate 1). Its capacity is 1000 ft^3/s at the Arizona Canal, and gradually increases throughout its 3.5 mile long reach to 2,000 ft^3/s at the Grand Canal. Although SRP reserves 1000 ft^3/s capacity it only uses this canal for wasting water to the Grand Canal. The remaining capacity is available for downstream storm runoff. The Old Cross Cut Canal has six box culverts, three foot bridges, and three other bridges, all of which may constrict the Canal flow during a flood.

Grand Canal. The Grand Canal runs parallel to and has the same function as the Arizona Canal. It receives flow from the New Cross Cut Canal, distributes it and wastes the excess flow into the New River. It receives flow from the Old Cross Cut Canal during floods and either brings it westward toward the New River or passes it to the Salt River through waste gates. The Grand Canal affects floodflows similarly to the Arizona Canal since it is also partially entrenched. However, the north bank does not cause the same ponding problem because the banks are generally less than one foot and the canal is mainly entrenched.

3. RAINFALL PRECIPITATION AND RUNOFF

3.01 Precipitation and Streamflow Records.

The City of Phoenix and the Maricopa County Flood Control District each have had precipitation gages installed since 1972 and 1980, respectively, but not enough data has been collected to determine a trend. No runoff gages, other than in controlled channels, exist in the area. Since there are no usable rainfall or runoff gages in this study area and the Phoenix Arizona and Vicinity, and Indian Bend Wash Projects are adjacent to it, much of the pertinent hydrology generated in the Phoenix Arizona and Vicinity studies (ref. 2 and 3, hereafter referred to as the Part 1 or 2 hydrology report) was directly adopted for this study. This provides for more consistancy between studies in the Phoenix area.

The USGS and the National Weather Service (NWS) operate a network of stream and precipitation gages in the Phoenix area. This was discussed in detail in reference 2. They were used to determine rainfall-runoff relationships and standard project flood frequencies in the Phoenix area.

3.02 Storms and Floods of Record.

General. Little is known about floods in the Phoenix area, or Arizona in general, during the early-to-mid-1800's. Rainfall records and/or historical accounts indicate that sizable floods have occurred on numerous occasions. Several events for which data are available were described in the Part 1 hydrology report. A brief description of significant storms are given in the following paragraphs.

Storm and Flood of 26-29 August 1951. The storm of 26-29 August 1951 was one of the heaviest on record at many Arizona locations. The storm developed from the remnants of an old Gulf of Mexico hurricane that crossed the Mexican mainland and turned northward towards Arizona on 26 August, combining with moisture outflow from a tropical storm west of Baja California. General, moderate rainfall, with heavy thunderstorms embedded, spread northward through Arizona on 26 and 27 August. At most stations, the maximum 24-hour rainfall, which accounted for about 65 percent of the total storm precipitation, occurred between approximately midday of the 27th and midday of the 28th. Precipitation generally tapered off during the afternoon of the 28h and ended on the 29th, although a few locations experienced a secondary burst of rain during the morning of the 29th. The total 26-29 August precipitation in and near the study area ranged from 3.85 inches at Phoenix and 3.95 inches at Prescott to 13.55 inches at Crown King. A total of 6.94 inches was observed at Waddell Dam. Total storm isohyets for 26-29 August are shown on plate 2. Because antecedent precipitation during August 1951 was relatively abundant, the ground in most areas was partially saturated at the beginning of the 26-29 August storm. Thus, the high precipitation intensities on 27 and 28 August produced heavy runoff in many areas, and caused significant flooding in some locations north and west of Phoenix. While the maximum mean daily inflow at Waddell Dam on August 29 was 23,144 ft^3/s, the peak discharge was probably considerably higher. Based on high water marks at numerous breaks on the Beardsley Canal in the Trilby Wash basin, the U.S. Soil Conservation Service estimated that a peak discharge of 35,000 ft^3/s may have occurred on Trilby Wash, assuming that all the numerous flood peaks along the canal had occurred at the same time. The peak discharge on the Hassayampa River at Box Damsite, near Wickenburg, is estimated by the USGS to have been 27,000 ft^3/s on 29 August. This storm was selected as the general type standard project storm for the Phoenix area.

Storm and Flood of 19 August 1954. Although there was no widespread general precipitation in Arizona during August 1954, one large and very intense thunderstrom occurred over the Queen Creek drainage area, approximately 50 miles east of Phoenix. The storm and flood were the most severe on record in the Queen Creek Basin. Precipitation intensities were very high during portions of the storm, especially between 5:00 and 9:00 a.m. on the 19th. The smelter at Ray (about 11 miles southeast of Superior) measured 4.05 inches of rain in less than 2 hours, while the Boyce Thompson Arboretum (about 4 miles west of Superior) measured a total of 5.3 inches for the storm, most of which fell within 3 hours. An estimated 140 square miles of area miles had over 5 inches of precipitation in the storm, and approximately 850 square miles had over 1 inch. Total storm isohyets for August 19 are shown on plate 3. This storm was selected as the local type standard project storm.

Storm and Flood of 22 June 1972. The heavy thunderstorm that occurred over northeastern Phoenix and adjacent communities on the morning of 22 June 1972 was a part of a series of early summer thunderstorms over the entire southwestern United States from 20 through 23 June 1972 that resulted from a deep flow of very moist, tropical air into the region from off the west coast of Mexico. In Phoenix the unofficial maximum rainfall was 5.25 inches during an estimated 2 hours near 4th Street and Camelback Road. Bucket survey amounts of 4.87 inches at 24th Street and Indianola Avenue and 4.8 inches at 28th Street and Indian School Road were confirmed by the National Weather

Service. The maximum recording-gage intensity was 3.85 inches in 80 minutes at 18th Street and Turney Avenue. Large hail also fell in the area. The storm was highly localized, with only 10 square miles having greater than 4 inches of rainfall and only 200 square miles with more than 2 inches. Total storm isonyets for 21-22 June are shown on plate 4. Estimates of peak discharges for 22 June made by the USGS include: Shea Wash at Shea Boulevard (1.79 square miles), 945 ft^3/s; Cudia City Wash 1000 feet upstream from McDonald Drive (2.16 square miles), 4200 ft^3/s; Dreamy Draw at 16th Street (1.62 square miles), 860 ft^3/s; Indian Bend Wash (at Indian Bend Road) near Scottsdale (142 square miles), 21,000 ft^3/s.

Ponding north of the Arizona Canal from one to four houses away occurred throughout this area. SRP shows no records of the Arizona Canal breaking in this study area other than 40th Street. However, at 56th Street, residents were flooded south of the Arizona Canal from water crossing the Canal at the depressed intersection of 56th Street and Mitchell Drive. Outside of this study area extensive flooding on Cudia City Wash south of the Arizona Canal was caused when the canal was overtopped at 32nd and 40th Streets. The Arizona Canal also broke at other locations outside this study area but the inundation was small relative to that caused by Cudia City Wash.

The U.S. Geological Survey 1972 Surface Water Records show a Cudia City Wash peak flow of 3000 ft^3/s at a location 1000 feet upstream from McDonald Drive, with a contributing drainage area of 2.16 square miles. The synthesized 50-year flood at this location is about 2700 ft^3/sec. The Salt River Project estimated a peak discharge of 3375 ft^3/s on Cudia City Wash just upstream from the Arizona Canal. The synthesized 50-year flood on Cudia City

Wash upstream from the Arizona Canal is 4400 ft^3/s derived from a 4.9 square mile contributing drainage area. This indicates that the 1972 peak flow on Cudia City Wash was approximately a 50-year frequency flood. The peak flow in Dreamy Draw at 16th Street, with a contributing drainage area of 1.62 square miles, was estimated to be 860 ft^3/s. This is approximately a 25-year event with no regulation upstream. (Dreamy Draw Dam was constructed in 1973.)

4. SYNTHESIS OF STANDARD PROJECT FLOOD

4.01 General.

The standard project flood (SPF) represents the flood that would result from the most severe combination of meteorologic and hydrologic conditions that are considered reasonably characteristic of the region. Normally larger than any past recorded flood in the area, it can be expected to be exceeded in magnitude only on rare occasions and thus constitutes a design standard that will provide a high degree of flood protection. The SPF was determined using a stream system analysis approach, which requires dividing the study area into subbasins that are hydrologically and meteorologically homogeneous. Subdividing a watershed permits more accurate modeling of the runoff process, as variations in topography and urbanization, as well as changes in channel characteristics, may be incorporated into the hydrologic description of the basin. The standard project storm was then centered over the watershed in the most critical flood producing manner. Application of the rainfall loss rate function enabled determination of the rainfall excess, which was then applied to the subbasin unit hydrograph to produce the subbasin flood hydrograph. Combining and routing of subbasin flood hydrographs to the desired concentration point, while removing percolation loss as appropriate, completed the computation. The elements involved in the computation are described below.

4.02 Standard Project Storm (Local Type).

The 19 August 1954 thunderstorm that was centered generally in the Queen Creek drainage east of Phoenix was determined to be the storm with the most severe flood-peak-producing rainfall that can reasonably be expected to occur over central Arizona. This storm was therefore used to determine the standard project flood for smaller drainage areas. The methods used to determine the total precipitation amounts, the duration of the storm, the intensity-duration relationships, and the time-distribution of the precipitation are explained in the following subparagraphs.

Total Precipitation. Total precipitation amounts for the standard project local storm were obtained from the isohyets (plate 3) of the 19 August 1954 Queen Creek thunderstorm, transposed and critically centered over the various drainage basins within the study area. Because the heaviest precipitation of this storm (7.5 inches maximum) occurred in the mountain and foothill areas where orographic influences are significant, the total storm depth was adjusted as it was transposed to the study area by means of 10-year, 6-hour precipitation values published by the National Weather Service in NOAA Atlas 2 (plate 5). The average total-storm precipitation over each basin of interest was determined by reducing the transposed maximum point precipitation by means of a family of depth-area curves (plate 6). These were constructed from the original depth-area curve developed from the isohyets of the original 1954 storm, adjusted for orographic influences. They are labeled according to the

10-year, 6-hour precipitation statistic. The depth-area curves in the higher mountain regions (where the 10-year, 6-hour precipitation is greater) decrease less rapidly with increasing area than do the curves in the deserts (where the 10-year, 6-hour precipitation is less).

Storm Duration. In the original August 1954 storm nearly all the precipitation fell within a 7-hour period according to local observations, and at many stations most of the rainfall occurred within 3 hours or less. Thus, a duration of 7 hours was used in the development of the standard project storm, with large portions of the total precipitation occurring within 1 to 3 hours.

Intensity-Duration Relationships. A time-distribution curve (mass curve) of precipitation was synthesized for each point within the August 1954 Queen Creek storm for which a total-storm precipitation measurement was made. These curves were based on the total precipitation at that location and available measurements or estimates of precipitation intensities for various durations within the storm. The curves at nearby locations within the storm were compared for consistency, and portions of the curves that were not based on firm observational data were adjusted to conform to patterns at nearby stations that were based on firm data. Maximum intensity-duration relationships for durations of approximately 2 to 7 hours were obtained from these August 1954 time-distribution curves. No extremely intense precipitation rates for durations of less than 1 hour were measured in this 1954 storm because of the lack of properly functioning recording rain gages in the area. Such high intensities have, however, been measured on a number of other occasions in central Arizona. Those rates are considered to be reasonably characteristic of the heaviest local storms in this part of the State. Therefore, maximum intensity-duration relationships for durations of less than 2 hours were obtained from all available intense local historical storms in central Arizona and were transposed to the Queen Creek area by means of the corresponding 10-year, t-hour precipitation statistic obtained from NOAA Atlas 2. Synthesized composite values of the intensity-duration relationship for the standard project storm in the Queen Creek area were thus obtained from the August 1954 storm and from other historical storms (plate 7, intensity-duration curve no. 7). These intensity-duration values were transposed to the study area by means of the 10-year precipitation statistic for each duration from 5 minutes to 7 hours.

Time-Distribution Patterns. From the standard project intensity-duration relationship (plate 7) and the synthesized precipitation mass curves drawn for the various observation points within the 1954 Queen Creek storm, a time-distribution curve for the point-value precipitation at the center of the standard project storm in the Queen Creek area was constructed. Time-distribution curves for areal averages of standard project storm precipitation in the Queen Creek area were derived from examination of various combinations of the synthesized Queen Creek mass curves. These central-value and areal-average time-distribution curves, expressed as a percent of the total storm precipitation, are shown on plate 8. In addition to variations according to areal extent, a time-distribution pattern of an intense storm (expressed as a percent of the total storm precipitation) can become significantly smoothed in mountainous regions, where the total rainfall of a storm can become augmented by the addition of a semi-steady orographic rainfall component. Therefore, for a given drainage area, the time-distribution of precipitation in a local thunderstorm will frequently become smoothed if the storm ascends a mountain slope. This factor was incorporated in a diagram (plate 9) that relates the time-distribution of precipitation in the local type standard project storm to both drainage area and 10-year, 6-hour precipitation. It can be seen from this diagram that patterns 1 and 2 (as a percent of the total storm) apply primarily to small drainage areas in the lower desert valleys, while patterns 4 and 5 apply to higher mountain regions (regions having higher 10-year, 6-hour precipitation), as well as to larger drainage areas.

Antecedent Rainfall. Ground conditions characteristic of standard project flood conditions are assumed to be established by 0.5 inch of precipitation occurring within a 24-hour period immediately prior to the local type standard project storm. This assumption has some basis in that a secondary storm cell formed in the same general area on the day following the Queen Creek storm of August 19, 1954. Meteorologically, this secondary storm cell could have occurred prior to the main Queen Creek storm. Therefore, when computing the SPF, the loss rate function, discussed in paragraph 4.03, was reduced to account for the antecedent rainfall.

4.03 Determination of Rainfall-Runoff Relationships.

General. Regional unit hydrograph and loss rate studies for the general Phoenix region are described in detail in the Part 1 hydrology report, (ref. 2). Twenty-two observed floods were reconstituted during these studies to derive relationships between rainfall and runoff applicable to most subbasins in the study area. Adopted rainfall-runoff relationships are discussed briefly below.

S-Graph Unit Hydrograph. A unit hydrograph is the runoff hydrograph that results from one inch of rainfall excess occurring uniformly over a watershed in specified period of time. The Los Angeles District's normal unit hydrograph procedure utilizes the S-graph, which is a summation graph of discharge in percent of ultimate discharge versus time in percent of lag time. Lag time is defined as the time required for 50 percent of the total volume (ultimate discharge) of the unit hydrograph to occur. The basin lag time for ungaged watersheds can be approximated by the use of the lag relationship presented on plate 10. The basin n-value is a variable in the lag equation that permits adjustment of the lag time depending on the type of ground cover and other characteristics for the subareas shown on plate 1 are given in table 2.

S-graph. The Phoenix Valley and the Phoenix Mountain S-graphs shown on plates 11 and 12, respectively, were used to describe the time distribution of runoff for most basins in the study area. The Phoenix Valley S-graph was derived from reconstitutions at New River at Bell Road, Skunk Creek at Phoenix, Cave Creek near Phoenix, Aqua Fria Tributary at Youngtown, and Queen Creek Tributary at Apache Junction. Similarly, the Phoenix Mountain S-Graph was derived from the New River near Rock Springs and New River at New River reconstitutions.

Basin n-Value. Basin n-values derived from the reconstituted unit hydrographs were used as a guide in establishing the following SPF basin n-values. Adjustments, based on judgement, were made to include the influence of basin characteristics that affect the lag time of the watershed.

For the highly urbanized valley area south of Arizona Canal and west of Old Cross Cut Canal, the terrain is flat and a majority of the rainfall does not concentrate. There is no storm drain system, so a majority of the flow is in streets and alleys. An appropriate method to model this overland flow across a frontal concentration point is the sheet flow unit hydrograph method described in reference 3. However, because of the complexity and many alternatives in this study, an SPF basin "n" of 0.15 which creates similar results to this method was determined and used with the Phoenix-Valley S-Graph to simplify computations.

Type of area	n-value
Mountain	0.040 to 0.045
Foothill	0.035
Valley	0.030
Highly urbanized valley	0.15

Rainfall Loss Rate Function. The variables in the H.E.C. loss rate function, which were used in this study and are shown graphically on plate 13, are: DLTKR--initial accumulated loss during which loss rate coefficient is increased; STRKR-- starting value of loss coefficient on exponential loss curve; RTIOL--ratio of loss coefficient on exponential loss curve to that corresponding to 10 inches more of accumulated loss; ERAIN-- exponent of precipitation in loss rate equation. Values for these variables to be used with both the local and the general standard project storms were taken from the Part 1 hydrology report and are reproduced on plate 14.

Baseflow and Snowmelt. Baseflow is considered negligible for this study area because runoff occurs only as a direct response to relatively high intensity rainfall. Snowmelt is not a significant contributing factor to runoff.

4.04 Flood Routing.

General. Reservoir routing was performed using the Modified Puls routing procedure. Channel routing was accomplished by the Muskingum method.

Muskingum Routing. The Muskingum coefficient, K, which can be approximated by the flood wave travel time in a reach, was determined by dividing reach length by average peak flow velocity. For channel routing (Arizona and Old Cross Cut Canals), a velocity of 2 and 4 feet per second (ft/s) respectively was determined by backwater computations for Arizona Canal, and normal depth computations for Old Cross Cut Canal. For overland routing of Arizona Canal breakouts through the swale east of Old Cross Cut (breaks from CP's 501 and 502 routed to CP401), an average velocity of 3 ft/s was used for the SPF as determined by backwater computations. For overland flow routing through the area west of the Old Cross Cut Canal and south of the Arizona Canal, a rating curve of average flow velocity versus discharge per unit cross section width were computed during hydraulic studies. The average flow velocity was weighted according to the proportion of the total discharge conveyed within the street right-of-way to the discharge conveyed beyond it. Averaged velocities were computed for SPF and 100-year floods, and ranged from 1.1 to 5.0 ft/s. The number of reaches between concentration points was determined by dividing the travel time between concentration points by the hydrograph computation time interval. Muskingum X values, which range from 0 to 0.5, were based on judgement. For improved channels, X values of 0.3 to

0.4 were used, depending on the type of improvement. For natural channels, X values used ranged from 0 to 0.3 depending on the amount of overbank flow encountered. Muskingum coefficients used in this study are given in tables 3 and 4. It should be noted that the computed peak discharges were often quite sensitive to changes in routing velocity, especially on the Old Cross Cut Canal. A schematic flow diagram is shown on plate 15 for without project routings, and on plates 16-19 for with project alternatives.

Modified Puls. The Modified Puls routing procedure was used in the Arizona Canal for breakout routing. Seven breakout locations were determined and are shown on plate 15. Elevation-storage relationships for each break were developed from September 1966, 2-foot contour maps provided by the Maricopa County Flood Control District and field inspection. The elevation-storage and elevation-spillway discharge relationships tabulated in table 5 were taken from HEC-2 runs used in hydraulic studies.

4.05 Generation of SPF Hydrograph in Highly Urbanized Areas. Section 4.03, Basin n-Value, discussed the characteristics of the highly urbanized valley area which is south of the Arizona Canal and west of the Old Cross Cut Canal. The total flood hydrograph below the Arizona Canal is comprised of the Arizona Canal breakout flow and of local runoff generated by subareas below the canal. Since flows do not concentrate, but travel mainly through very flat streets, the approach used to compute the peak discharges varied from the above paragraphs as described in the following paragraphs.

Breakout locations shown on plate 15, and hydrographs were calculated and routed to each concentration point or frontal flow line as described in section 4.04. They were then combined with the local runoff flood hydrographs.

To determine the local flow for locations below the Arizona Canal, four SPF hydrographs were computed at the Grand Canal (CP 207, 206, 205, and 204) using the unit hydrograph procedure for drainage areas between the Arizona Canal and the Grand Canal defined by combined areas (407+307+207), (406+306+206), (305+205), and (204). Next these hydrographs were ratioed proportionally by drainage area size and the slope of the peak discharge enveloping curve (plate 20) to obtain hydrographs at each upstream frontal flow line. The lag time of the hydrographs at each upstream frontal flow line was judged to be a portion of the total lag time determined by the ratio of the length of each subarea flow path to the total flow path length. The general shape of the ratioed local flood hydrograph obtained in this manner was the same as the overall computed hydrograph.

Next, the breakout flood hydrographs were routed and combined with the computed local flood hydrographs to obtain the total peak discharge at each frontal flow line (407, 406, 305). This procedure was repeated for each successive reach to the Grand Canal (307, 306, 205, and then 207, and 206).

The flow at CP 205, presented on table 1, was generated by assuming that breakouts from CP's 401B and 302B remain within the subarea contributing to CP 205 (plate 1). These two breaks actually disperse into the adjacent subareas as well as contributing to CP 205. This dispersing effect was accounted for in determining overflow depths during hydraulic studies.

4.06 Standard Project Flood Results.

Standard project flood results, computed as described above, were determined for present conditions without alternative plans. SPF peak discharges without project are presented in table 1. Future condition results were approximately the same as present. The standard project local storm (August 1954 Queen Creek) produced the maximum peak runoff rate at all project sites (ref. 2).

5. DISCHARGE FREQUENCY ANALYSIS FOR URBANIZED AREAS

5.01 General.

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

5.02 Results.

Since this study area is bounded by two other projects, Phoenix Arizona Vicinity and Indian Bend Wash, the same discharge frequency relationships used in them was adopted for this study. As stated in refernece 2, the graphical method was best suited for the Phoenix area. For this reason no expected probability adjustment was performed.

Recorded and historical floodflows were plotted during Phase 2 of the Phoenix studies for stream gages based on the median plotting positions in Beard's "Statistical Methods in Hydrology." Four long record stream gages (San Carlos River near Peridot, Gila River near Solomon, Salt River near Roosevelt, and Hassayampa River at Box Damsite) were compared for record consistency in order to estimate SPF exceedence frequency in the Phoenix Arizona and Vicinity Study. Resulting SPF exceedence percentages varied as follows: San Carlos and Hassayampa--0.2 to 0.5 percent, Solomon--0.3 percent. SPF for Roosevelt is not available. Variations were dependent on graphical or analytical curve fitting of the data. All stations show consistency through similar standard deviations. This analysis indicated that an SPF exceedence frequency of 0.2 to 0.5 percent is reasonable for areas in this study (ref. 2).

Two stream gages located in southern Arizona on catchments with significant percentages of impervious cover were used to determine the adopted n-year to SPF frequency ratios in the Part 1 hydrology report. The gages were Agua Fria Tributary at Youngtown (USGS Gage No. 9-5137) and Tucson Arroyo at Vine Avenue (USGS No. 9-4830). The discharge frequency curve for the Youngtown stream gage (plate 21) is representative of a valley watershed in Phoenix with approxi-mately 40 percent impervious cover. The data collected was from 1962 to 1968, a total of 7 points. The gage was discontinued after this. The discharge frequency curve for the Tucson stream gage (plate 22) is indicative of a more highly urbanized watershed, (60 percent impervious cover); however, the normal annual precipitation in and around Tucson is higher than the Phoenix area. The average of the ratios of the n-year flood to standard project flood for these two watersheds was used for determining discharge frequency curves for urbanized basins in the Phoenix region (ref. 2). Since the Part 1 hydrology report, several years of data have become available for the Tucson gage. During this study, the same gage was plotted using a continuous record from 1944 to 1981 of 38 events. No data is available beyond 1981. Plate 23 presents these points superimposed on the Part 1 hydrology Tucson frequency curve. The plotted points fit the earlier frequency curve sufficiently, so no revisions were made to the frequency relationships. These relationships are as follows:

n-Year Flood	Percent of SPF for an urbanized watershed
SPF	100
100	45
50	32
40	26
25	21
20	19
10	12

Because routing velocities of breakout flows vary with the quantity of flow, the 100-year discharges were computed by multiplying the SPF hydrographs of each subarea by 45 percent (ref. above table), and routed using the 100-year peak flows to determine new routing velocities. The combined 100-year flows confirmed that the above percentage of SPF table is appropriate for this study area. Therefore, the 50- and 25-year peak discharges were determined by using the n-year to 100-year ratio of the 100-year peak discharge. This results in discharge-frequency values that plot in the same shape as the frequency curves developed from recorded runoff data.

5.03 Risk Analysis.

For any design frequency there is a corresponding risk which represents the likelihood that the design flow will be exceeded at least once in a certain number of years.

This section addresses the risk of the design flood being exceeded in an amount of time called the project life. The project life is defined as the number of years a project will last, and was assumed to be 100 years in each alternative. The risk of any one alternative being exceeded was determined by using the binomial equation, $Risk = 1 - (1 - p)^n$, where p is the exceedance frequency and n is the project life. The relationships of the design exceedance frequency to risk are as follows:

Exceedance Frequency of Design (years)	Project Life (years)	Risk of being exceeded (percent)
10	100	100.0[1]
25	100	98.4
40	100	92.0
50	100	86.5
100	100	63.4

This information will be useful in determining the proper alternative and level of protection.

[1] Note: This risk is actually rounded from 99.997%.

6. PROJECT ALTERNATIVES

6.01 General.

Seven alternative plans were formulated for further study. Some require little or no construction but offer relatively small protection while those offering greater flood protection are also more expensive to implement. All alternative plans concentrate on reducing ponding behind or breakouts over the Arizona Canal. They also utilize an improved Old Cross Cut Canal to convey flood waters to the Salt River. Alternatives 1 through 4 involve starting evacuation of the Arizona Canal prior to flood waters reaching it thus enabling the canal to provide flood protection. A sensitivity analysis which consisted of starting the canal evacuation at different times in the storm, was also completed.

6.02 Alternatives.

Alternative 1 requires no structural modifications to the system. It incorporates closing the existing radial gates at Camelback Road and releasing up to 625 ft^3/s into the Arizona Cross Cut and up to 1000 ft^3/s into the Old Cross-Cut Canals. In doing this, the canal flow decreases as floodflows into the canal increase.

Alternative 2 is the same as alternative 1 except that the Old Cross Cut Canal capacity and its gate capacity at the Arizona Canal are increased to 1200 ft^3/s, 1500 ft^3/s, and then 2000 ft^3/s.

Alternative 3 is similar to alternative 2 plus a radial gate at either 48th, 44th, or 40th Street is added to isolate the Arizona Canal between this Street and Camelback Road. Hydraulic studies determined that the most appropriate gate location for this alternative is 44th Street.

Alternative 4 is similar to alternative 3 except with an invert elevation at 48th Street of 5 feet less than exists. This lower invert will taper back to 56th and 40th Street.

Alternative 5 consists of the optimal gate sizes of alternative 3 with the canal bifurcated. The north half is for flood control and the south half is for water supply.

Alternative 6 consists of a collector channel parallel to and north of the Arizona Canal, and similar to the Arizona Canal Diversion Channel (ACDC). This channel collects water between 39th Street and 1700 feet upstream of 64th Street, and brings it to 48th Street, where it is syphoned under the Arizona Canal and released into an improved Old Cross Cut Canal. This alternative was analyzed for the 25-, 50-, and 100-year capacity.

Alternative 7 consists of a storm drain system north of the Arizona Canal which releases flow into the Old Cross Cut Canal expanded to accept the floodflows. The storm drains are located along Lafayette Boulevard and Camelback Road. These flows are collected at Arcadia Drive upstream of the Arizona Canal and then released to the Old Cross Cut Canal through a syphon. This storm drain system was studied for the 25-, 50-, and 100-year frequency floods.

6.03 Method of Analysis and Results.

Alternatives 1 through 4, which consist of varied operations of existing and proposed canal gates, were studied in order to find the most efficient way of evacuating the canal so that floodflows are captured by it instead of spilled over it. Schematics of each alternative are on plates 16-17. Because of the complexity of analyzing this system, a Hydrologic Engineering Center (HEC) program called USTDY was used to model the Arizona Canal. Using unsteady flow, this program modeled gate operations as well as additional side inflow in order to study the possiblitiy of preventing floods by forecasting them and emptying the Arizona Canal. Two criteria for operating the canal were studied. The first was to begin operating the canal gates when floodwaters reach it, about 1 hour before the peak and 4 hours into the 7-hour Queen Creek storm. The second criteria for operating the canal gates requires flood forecasting from rainfall such that gate operations begin 2 hours prior to the peak runoff, 3 hours into the Queen Creek storm, or at least 1 hour prior to significant flow reaching the Arizona Canal. Plate 24 shows the rainfall-runoff timing for subarea 13 north of the Arizona Canal. Discharges into the Old Cross Cut Canal were used in HEC-1 to determine the Old Cross Cut Canal design capacity for the appropriate level of protection. Alternative 1 provides a 25-year level of protection without damage if the gates are operated by forecasting, as per criteria number 2, and a 20-year frequency if they are operated when flood waters reach the canal, criteria number 1. Alternatives 2, 3, and 4 provide 25, 40, and 100-year levels of protection respectively when the gates are operated by criteria number 1. Design flows are presented in table 6.

Alternatives 5 and 6 have the same hydrologic analysis but different levels of protection as determined through the USTDY program. The modeling program was HEC-1 as in the without project analysis. The flows from the subareas north of Arizona Canal are routed in the proposed canal using the Muskingum method with an X = 0.3. Routing parameters are in table 4, the schematic is on plate 18. Alternative 5 provides 10-year level of protection while alternative 6 provides protection for any frequency depending on its design. The 25, 50, and 100-year frequency discharges for alternative 6 and the 10-year frequency discharges for alternative 5 are presented in table 7.

Alternative 7, the storm drain system, is shown on plate 19. To conform to the level of detail in this study, no routing was performed. Drainage area ratios of subareas 11 through 16 were used to determine the necessary capacity of each length of storm drain. Flow at convergences were directly summed instead of combined as hydrographs. Thus these flows have a more conservative estimate of the necessary capacity of the Old Cross Cut Canal than other alternatives for the same frequency. Design flows are presented in table 8.

N-year peak discharges are presented for each alternative in tables 6, 7 and 8, and 100-year flood hydrographs at CP 401 and 401D for without and with project, respectively, are on plates 25 and 26.

6.04 Flood Forecasting.

General. Flash floods are sudden violent floods caused by heavy rain from which runoff concentrates within minutes. Flash floods can occur in the Phoenix area at any time of the year, but the predominant seasons are summer and early fall. They can occur as the result of isolated thunderstorms, tropical storms, or within general storms.

Local summer thunderstorms causing sudden runoff are common in the Phoenix area. Most intense between July and September, they consist of high-intensity rainfall over relatively small areas for short periods of time. Runoff from local storms is usually characterized by a rapidly rising and receding hydrograph. Runoff from local storms can result in flash floods, sometimes with loss of life and serious local property damage.

Flood Forecasting. Alternatives 1 through 5, discussed previously, require SRP gate operations to utilize the Arizona Canal for flood control. To do this, SRP will need a flood forecasting system which is capable of communicating with their automated operating system. A flood forecasting system for this project area would consist of weather forecasting for the local area, rainfall gages, Arizona Canal flow gages, transmitters, data receiving equipment, an action plan which uses SRP's remotely controlled operating equipment for the Arizona Canal gates, and public involvement.

Available Resources. SRP has indicated a willingness to incorporate a flood control operation plan which includes gate operations in their canals during flood events. Since SRP is the owner/operator and has both vast experience and automated equipment with which to operate the Arizona and Old Cross Cut Canals, it is necessary to use their agency to monitor the system and activate a flood forecasting plan. At present SRP is revamping their automated gate operating system. Telephone cables are being replaced by radio communications, and a new computer system is being installed. They have no plans to install rain gages, and have no rain gages in the project area. However they will have a radio receiver connected to their computer, both of which will be available for flood forecasting as well as for their normal gate operations.

There is an existing event recording precipitation gage in the project area at the fire station near Thomas Road and 48th Street. The City of Phoenix has been receiving good data from it since 1976. Being located south of the Arizona Canal and west of the Old Cross Cut Canal, it does not represent, but may be indicative of, the rainfall which will affect the Arizona Canal. Furthermore, it is an event recording gage that does not provide information as to whether it is operating properly during dry periods.

The usefulness of this gage for forecasting versus the cost of adding a radio transmitter would have to be carefully weighed if it were to be incorporated into SRP's system.

Proposed Forecasting System. A flood forecasting system must be designed in cooperation with SRP. Two to four dependable continuous reporting gages with radio transmitters would be necessary. Redundancy of the equipment at each gage site would depend on the location and ease of access to the sites. The location of each gage is critical in order to get a good estimate of the flood potential at the Arizona Canal where the most damage is done. One gage should be located near the top of Camelback Mountain, and one should be downstream closer to the Arizona Canal. The two others should be located midway between the Arizona Canal and Camelback Mountain, one toward the western boundary and the other toward the eastern boundary. This will provide a good representation of the flood producing rainfall.

To receive and utilize this data, SRP will have available for flood forecasting, a radio receiver which will directly input data to their computer, and the local National Weather Service to aid in predicting the severity of the storm. A schematic of the system is shown on plate 27. Depending on the alternative chosen, a plan of action would be administrated by SRP using their computer system to operate gates as designated by this plan. Essential to this plan is the effective response time, or the time for rainfall over Camelback to cause runoff at the Arizona Canal. SRP has indicated that the effective response time of this area is less than one half hour. The Corps analyses indicate about 20 minutes for the SPF event (plate 24). Therefore this automated computerized system is absolutely necessary to effectively operate the proposed forecasting system.

7. ADEQUACY OF RESULTS

In order to determine the adequacy of the SPF peak discharge, three locations, CP 207, 206, and 302B, were plotted on the Arizona, New Mexico, South West Utah enveloping curve, plate 20. Each plotted point falls short of the enveloping curve to about the same order of magnitude as observed floods from local summer storms presented on this plate. Because the valley area is particularly flat, the mountain runoff is attenuated quickly at the valley's edge, where flooding occurs, and does not contribute significantly to the plotted peak flows of the valley floor. Mountain runoff is also partially diverted by the Arizona and Old Cross Cut Canals, thus causing less flow per square mile at points similar to CP 302B. The standard project flood results are reasonable as compared with the enveloping curve determined from events in the Arizona, New Mexico, South West Utah area.

8. BIBLIOGRAPHY

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TABLE 1

N-YEAR PEAK DISCHARGES WITHOUT PROJECT (PRESENT CONDITIONS)

CP	Location	Drainage Area (mi ^{2})	Storm Centering (mi ^{2})	Future Conditions 100-YR (ft ^{3}/s)	Present Conditions (ft ^{3}/s) SPF	100-YR	50-YR	25-YR
501U	Spur Cir. U/S of A.C.	0.93	3.9	1500	3300	1500	1000	700
501D	Spur Cir. D/S of A.C.	0.93	3.9	0	260	0	0	0
502U	56th St. U/S of A.C.	1.36	3.9	710	1500	680	490	320
502D	56th St. D/S of A.C.	1.36	3.9	360	800	350	230	140
401	Thomas Rd. at O.C.C.	3.09	7.6	2400	4700	2100	1500	980
401U	O.C.C. U/S Thomas Rd	----[1]	7.6	1000	1000	1000	1000	1000
401D	O.C.C. D/S Thomas Rd.	----[1]	7.6	1250	1250	1250	1250	1250
401B	O.C.C. Breakout at Thomas Rd.	3.09	7.6	2100	4500	1900	1300	730
302	McDowell at O.C.C.	.96	11.3	1400	2900	1300	940	620
302D	O.C.C. D/S at McDowell	--[a]	11.3	1450	1450	1450	1450	1450
302B	O.C.C. Breakout at McDowell	4.05	11.3	1200	2700	1100	740	420
203	Above Grand Canal at O.C.C.	1.72	17.2	2200	4400	2000	1400	920
203D	O.C.C. Inflow to Grand Canal	----[1]	17.2	2000	2000	2000	2000	2000
203B	O.C.C. Breakout at Grand Canal	5.77	17.2	1600	3600	1400	850	370
204	Washington & Grand Canal	0.59	17.2	410	900	410	290	190
305	44th St. & Coronado Rd.	3.59	11.3	2400	5700	2200	1600	1000
205	1500ft. west 40th St.	5.12	17.2	2100	5100	1900	1400	890
504U	Heatherbrae U/S of A.C.	0.38	3.9	650	1400	620	440	290
504D	Heatherbrae D/S of A.C.	0.38	3.9	270	750	260	0	0
503U	48th St. U/S of A.C.	1.20	3.9	2100	4400	2000	1400	920
503D	O.C.C. D/S of A.C.	1.58	3.9	1100	1900	1100	1000	680
406	Flower Pl.	2.43	7.6	550	2000	530	230	160
306	38th St. & Yale St.	3.54	11.3	780	2100	780	450	320
206	32nd St & Grand Canal	5.09	17.2	1100	2200	1100	710	460
505U	44th St. U/S of A.C.	0.61	3.9	1100	2300	1000	730	480
505D	44th St. D/S of A.C.	0.61	3.9	540	1700	510	360	320
506U	40th St. U/S of A.C	0.39	3.9	650	1400	620	440	290
506D	40th St. D/S of A.C.	1.00	3.9	210	510	200	120	50
506AD	1000 ft west of 40th St. D/S of A.C.	1.00	3.9	300	650	280	200	140
407	36th St. & Devonshire Ave.	2.15	7.6	1200	3200	1200	850	560
307	30th St. & Mitchel Ave.	3.24	11.3	1300	3100	1300	920	610
207	24th St. & Grand Canal	4.69	17.2	1400	2900	1400	1000	650

1. The first 1000 ft ^{3}/s in the Old Cross Cut Canal are from outside of this study area by the way of the Arizona Canal therefore no drainage area is defined. Flows are channel capacity.

TABLE 2

BASIN CHARACTERISTICS

Subarea	Drainage Area (mi^2)	L (mi)	Lca (mi)	Slope (ft/mi)	Impervious Cover (%)		Basin "n"-Value	
					present	future	present	future
11	0.93	1.63	0.79	380	25	35	.04	.035
12	0.43	1.69	0.85	500	25	35	.04	.035
13	1.20	1.53	0.66	515	25	35	.04	.035
14	0.38	1.49	0.76	500	20	30	.04	.035
15	0.61	1.25	0.61	400	25	35	.04	.035
16	0.39	1.26	0.65	160	25	35	.04	.035
1	1.73	2.12	1.03	66	25	30	.04	.030
2	0.96	1.42	.60	214	25	40	.04	.030
3	1.72	1.99	1.03	196	25	40	.04	.030
204	.59	1.14	.38	30	30	30	.15	.15
205	1.07	3.05	1.26	24	35	35	.15	.15
206	3.51	4.19	2.0	26	35	35	.15	.15
207	3.69	4.19	2.0	27	35	35	.15	.15
406	0.85							
407	1.15							
305	0.50							
306	1.11							
307	1.09							

TABLE 3

PERTINENT ROUTING DATA[1]
WITHOUT PROJECT

Reach[2]	Dist. (ft)	Muskingum X[3]	SPF Vel (fps)	SPF K[3] (hrs)	SPF NRCHS	100-YR Vel (fps)	100-YR K[3] (Hrs)	100-YR NRCHS
501 R 401	9600	0.0	3.0	0.89	11	0.0	--	--
502 R 401	6560	0.0	3.0	0.61	7	3.0	0.61	7
503 R 401	6000	0.3	4.0	0.42	5	4.0	0.42	5
401 R 302	4200	0.3	4.0	0.30	4	4.0	0.30	4
302 R 203	9000	0.3	4.0	0.63	8	4.0	0.63	8
401B R 305	5500	0.0	5.0	0.31	4	3.6	0.42	5
302B R 305	2000	0.0	4.2	0.13	2	3.1	0.18	2
305 R 205	6000	0.0	5.8	0.29	3	4.0	0.41	5
503B R 406	5600	0.0	4.0	0.39	5	1.8	0.85	10
504 R 406	5620	0.0	4.9	0.32	4	3.2	0.48	6
406 R 306	4800	0.0	2.9	0.46	6	1.1	1.21	15
306 R 206	6000	0.0	2.7	0.62	7	1.4	1.19	14
505 R 407	5960	0.0	4.2	0.40	5	2.1	0.61	7
506 R 407	5700	0.0	4.7	0.34	4	3.3	0.48	6
407 R 307	4800	0.0	3.4	0.39	5	2.1	0.65	8
307 R 207	6000	0.0	3.4	0.50	6	2.2	0.76	9

1. Refer to plate 15 for schematic of routing. No routing was performed for 50 and 25-year frequencies.
2. This symbolizes the reach from subarea "A1" routed through subarea "A2" ("A1" R "A2").
3. Muskingum Coefficients, travel time in hours.

TABLE 4

PERTINENT ROUTING DATA[1]
WITH PROJECT ALTERNATIVES

Reach[2]	Used in Alternative	Length (ft)	velocity (fps)	NRCHS	K^3 (hrs)	X^3
501 R 502	5,6	3600	10	1	0.10	0.4
502 R 503	5,6	5400	10	2	0.15	0.4
506 R 505	5,6	3600	10	1	0.10	0.4
505 R 504	5,6	2900	10	1	0.08	0.4
504 R 503	5,6	1300	10	1	0.04	0.4
503 R 401	1-7	6060	22	1	0.08	0.4
401 R 302	1-7	4200	20	1	0.06	0.4
302 R 203	1-7	9000	25	1	0.10	0.4

1. Refer to plates 16-19 for schematics of routing.

2. This symbolizes the reach from subarea "A1" routed through subarea "A2" ("A1" R "A2").

3. Muskingum Coefficients, travel time in hours.

TABLE 5.

ELEVATION-STORAGE-OUTFLOW RELATIONSHIPS

CP	Locations	Elevation (ft)	Storage (ac-ft)	Outflow (ft^3/s)	
501	60th Street Break	1270	37	0	
		1272.88	148	0	
		1273.26	175	39	
		1273.49	193	100	
		1273.83	222	220	
		1274.13	249	344	
		1274.48	284	519	
		1274.99	337	803	
		1276.24	482	1655	
502	56th Street Break	---	---	---	
		1254.8	14	0	
		1255.01	19.2	50	
		1255.13	21.6	100	
		1255.14	26.0	151	
		1255.33	30.9	226	
		1255.49	35.3	337	
		1255.60	38.1	449	
		1255.81	43.4	583	
		1256.0	48.5	712	
		1256.73	69.7	1246	
		1257.0	76.8	1472	
		1257.39	88.5	1809	
503	48th Street Break	---	---	---	
		1253.7	34	0	
		1253.93	58.1	40	
		1254.0	69.1	97	
		1254.06	86.8	231	
		1254.12	105.0	420	
		1254.16	120.9	559	
		1254.19	125.2	668	
		1254.28	144.7	887	
		1254.28	144.7	1105	
		1254.47	161.3	1540	
		1254.76	193.1	1540	
		1255.21	235.3	2499	
		1255.68	295.7	3637	
504	47th Street Break	1252.4	12.0	0	
		1252.64	21.0	1	
		1253.13	23.2	39	
		1253.31	25.3	126	
		1253.46	27.1	215	No erosion for the 50,
		1253.63	29.2	334	25 & 10 yr Flood
		1254.02	34.3	640	
		1254.49	41.4	1086	
		1254.80	46.1	1403	

TABLE 5. Continued

CP	Location	Elevation (ft)	Storage (ac-ft)	Outflow (ft^3/s)
505	44th Street Break	1250.9	20	0
		1251.03	22.4	50
		1251.11	22.9	100
		1251.34	24.6	300
		1251.41	48.8	398
		1251.56	52.7	582
		1251.66	55.5	721
		1251.79	59.1	907
		1251.90	62.6	1085
		1252.04	66.3	1336
		1252.39	75.7	2023
		1252.74	84.8	2703
506	40th Street Break	---	---	----
		1250.0	8	0
		1250.19	14.1	50
		1250.30	15.6	100
		1250.30	17.3	148
		1250.31	19.1	247
		1250.34	20.8	338
		1250.41	22.5	415
		1250.50	24.2	492
		1250.60	25.8	570
		1250.70	27.4	647
506A	Spillway West of 40th Street	1248.5	3	0
		1248.84	4.2	50
		1249.01	4.6	163
		1249.86	5.8	252
		1250.17	6.6	341
		1250.51	7.6	450
		1250.79	8.4	550
		1251.06	9.2	650
		1251.36	10.0	750

TABLE 6

N-YEAR PEAK DISCHARGES (FT^3/S)
WITH PROJECT ALTERNATIVES 1-4
(FUTURE CONDITIONS)

CP	Location	Drainage Area (mi^2)	Storm Centering (mi^2)	Alternative[1] 1	Alternative[1] 1A	Alternative[1] 2	Alternative[1] 3	Alternative[1] 4
	Frequency Criteria[3]			25-Year 2	20-Year 1	25-Year 1	40-Year 1	100-Year 1
501U	Spur Cir. U/S of A.C.	0.93	3.9	720	650	720	890	1500
502U	56th St. U/S of A.C.	0.43	3.9	330	300	330	410	710
503U	48th St. U/S of A.C.	1.20	3.9	980	880	980	1200	2100
504U	Heatherbrae U/S of A.C.	0.38	3.9	300	270	300	380	650
505U	44th St. U/S of A.C.	0.61	3.9	500	450	500	620	1100
506U	40th St. U/S of A.C.	0.39	3.9	300	270	300	370	650
503D	O.C.C. D/S of A.C.	3.94[2]	3.9	840	800	1000	1400	2600
401D	O.C.C. at Thomas Rd.	5.67[2]	5.7	2000	1800	2200	2700	5000
302D	O.C.C. U/S of McDowell	6.63[2]	6.6	2400	2200	2600	3300	6000
203D	O.C.C. at Grand Canal	8.35[2]	8.4	3400	3100	3500	4400	8100
204	Washington & Grand Canal	0.59	17.2	190	170	190	240	410
305	44th St. & Coronado Rd.	0.50	11.3	90	85	90	120	200
205	1500 ft. west of 40th St.	1.07	17.2	180	160	180	220	380
406	Flower Pl.	0.85	7.6	150	140	150	190	320
306	38th St. & Yale St.	1.96	11.3	300	270	300	370	640
206	32nd St. & Grand Canal	3.51	17.2	460	420	460	570	1000
407	36th St. & Devonshire Ave.	1.15	7.6	190	180	190	240	420
307	30th St. & Mitchel Ave.	2.24	11.3	330	300	330	410	710
207	24th St. & Grand Canal	3.69	17.2	490	440	490	610	1100

1. Alternatives are described on pages 27 and 28.

2. Drainage area is dependent on the alternative.

3. Criteria 1 operates canal gates when first flood waters reach the canal.
Criteria 2 operates canal gates as per forecasting, at least 1 hour prior to first flood waters.

TABLE 7

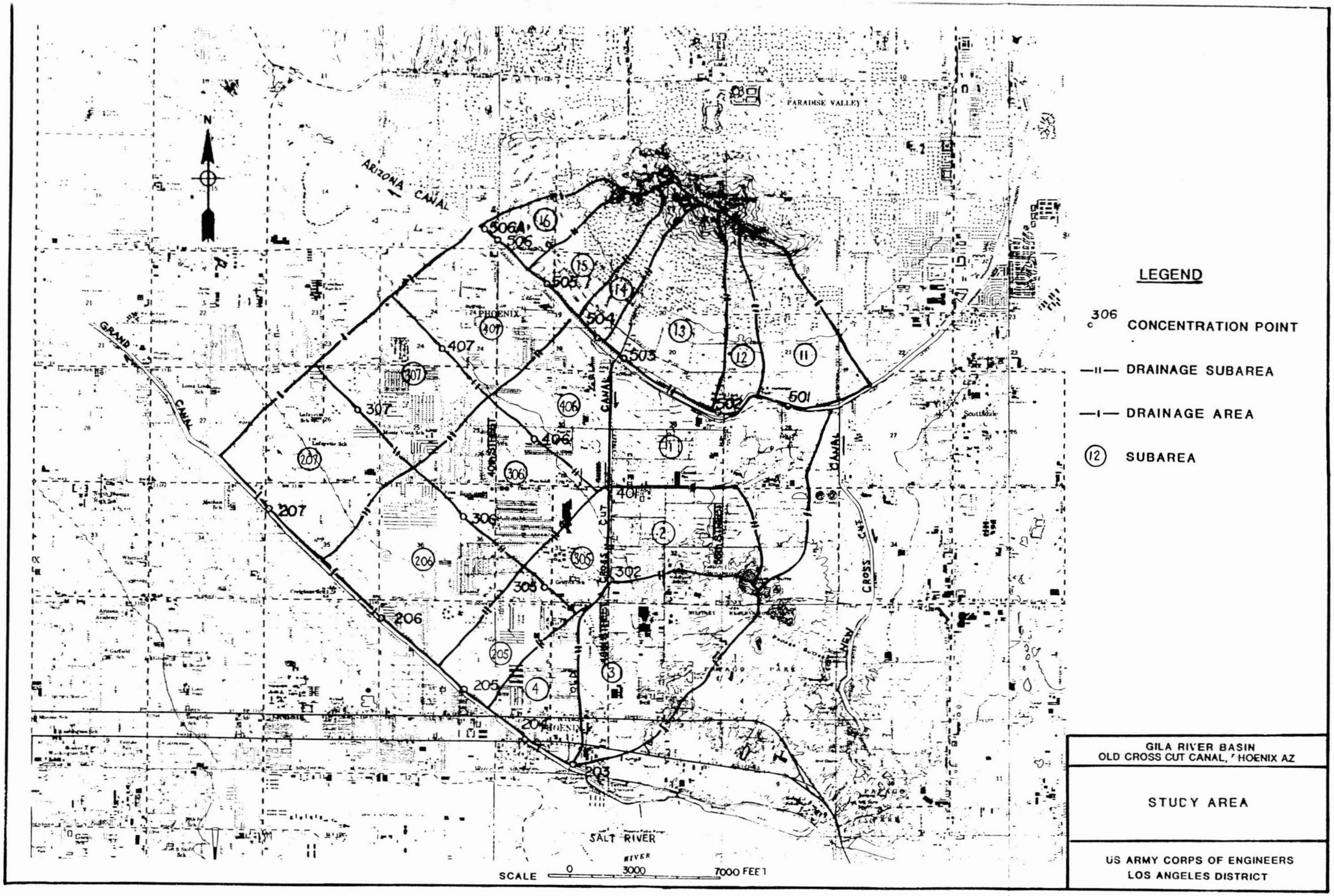
N-YEAR PEAK DISCHARGES (FT^3/S)
WITH PROJECT ALTERNATIVES 5 AND 6
(FUTURE CONDITIONS)

CP	Location	Drainage Area ($mi.^2$)	Storm Centering (mi^2)	Alternative 6 100-YR	Alternative 6 50-YR	Alternative 6 25-YR	Alt. 5 10-YR
501	A.C. at Spur Cir.	0.93	3.9	1500	1100	720	410
502	A.C. at 56th St.	1.36	3.9	2200	1600	1000	580
503	A.C. East of O.C.C.	2.56	3.9	3700	2600	1700	980
506	A.C. at 40th St.	0.39	3.9	650	460	300	170
505	A.C. at 44th St.	1.00	3.9	1600	1100	750	430
504	A.C. at Heatherbrae	1.38	3.9	2200	1600	1000	590
503D	O.C.C. at A.C.	3.94	3.9	5900	4200	2700	1600
401D	O.C.C. at Thomas Rd.	5.67	5.7	8100	5700	3800	2100
302D	O.C.C. U/S of McDowell	6.63	6.6	8800	6200	4100	2300
203D	O.C.C. at Grand Canal	8.35	8.4	10,000	7400	4900	2700
204	Washington & Grand Canal	0.59	17.2	410	290	190	110
305	44th St. & Coronado Rd.	0.50	11.3	200	140	90	50
205	1500 ft. west of 40th St.	1.07	17.2	380	270	180	100
406	Flower Pl.	0.85	7.6	320	230	150	85
306	38th St. & Yale St.	1.96	11.3	640	450	300	170
206	32nd St. & Grand Canal	3.51	17.2	1000	700	460	270
407	36th St. & Devonshire Ave.	1.15	7.6	420	300	190	110
307	30th St. & Mitchel Ave.	2.24	11.3	710	510	330	190
207	24th St. & Grand Canal	3.69	17.2	1000	750	490	280

TABLE 8

N-YEAR PEAK DISCHARGES
WITH PROJECT ALTERNATIVE 7
(FUTURE CONDITIONS)

			50-Year			25-Year			
Pipe No.	CP	Drainage Area mi^2	Flow From U/S Pipes	Inlet Capacity ft^3/s	Pipe Capacity ft^3/s	Flow from U/S Pipes	Inlet Capacity ft^3/s	Pipe Capacity ft^3/s	Pipe Ends Into Pipe no.
1	1	.31		180	180		120	120	
	2	.45		85	270		56	180	
	3	.90		270	540		180	360	9.1
2	4	.18		110	110		73	73	9.1
3	7	.31		190	190		120	120	5
4	8	.37		200	200		130	130	5
5	9	.68	390	---	390	250	---	250	
	10	.81		350	740		230	480	
	11	1.11		260	1000		170	650	
	12	1.16		60	1060		40	690	9.2
6	13	.14		160	160		110	110	
	14	.67		430	590		280	390	
	15	.96		270	860		180	570	
	16	1.84		780	1640		510	1080	9.2
	17	NO ROUTING SO NO CHANGE			1640			1080	
7	19	.25		290	290		190	190	
	20	.36		130	420		90	280	
	21	.59		280	700		180	460	9.3
8	22	.41		240	240		160	160	
	23	.54		160	400		110	270	
	24	.60		70	470		50	320	
	25	.63		40	510		20	340	9.3
9.1	5&16	1.08	650	----	650	430	----	430	9.2
9.2	18	3.00	2700	----	3350	1770	----	2200	9.3
9.3	26	3.85	1210	----	4560	800	----	3000	Old Cross Cut Canal
	401	5.67	----	----	5700	----	----	3800	Old Cross Cut Canal
	302	6.63	----	----	6200	----	----	4100	Old Cross Cut Canal
	203	8.35	----	----	7400	----	----	4900	Old Cross Cut Canal

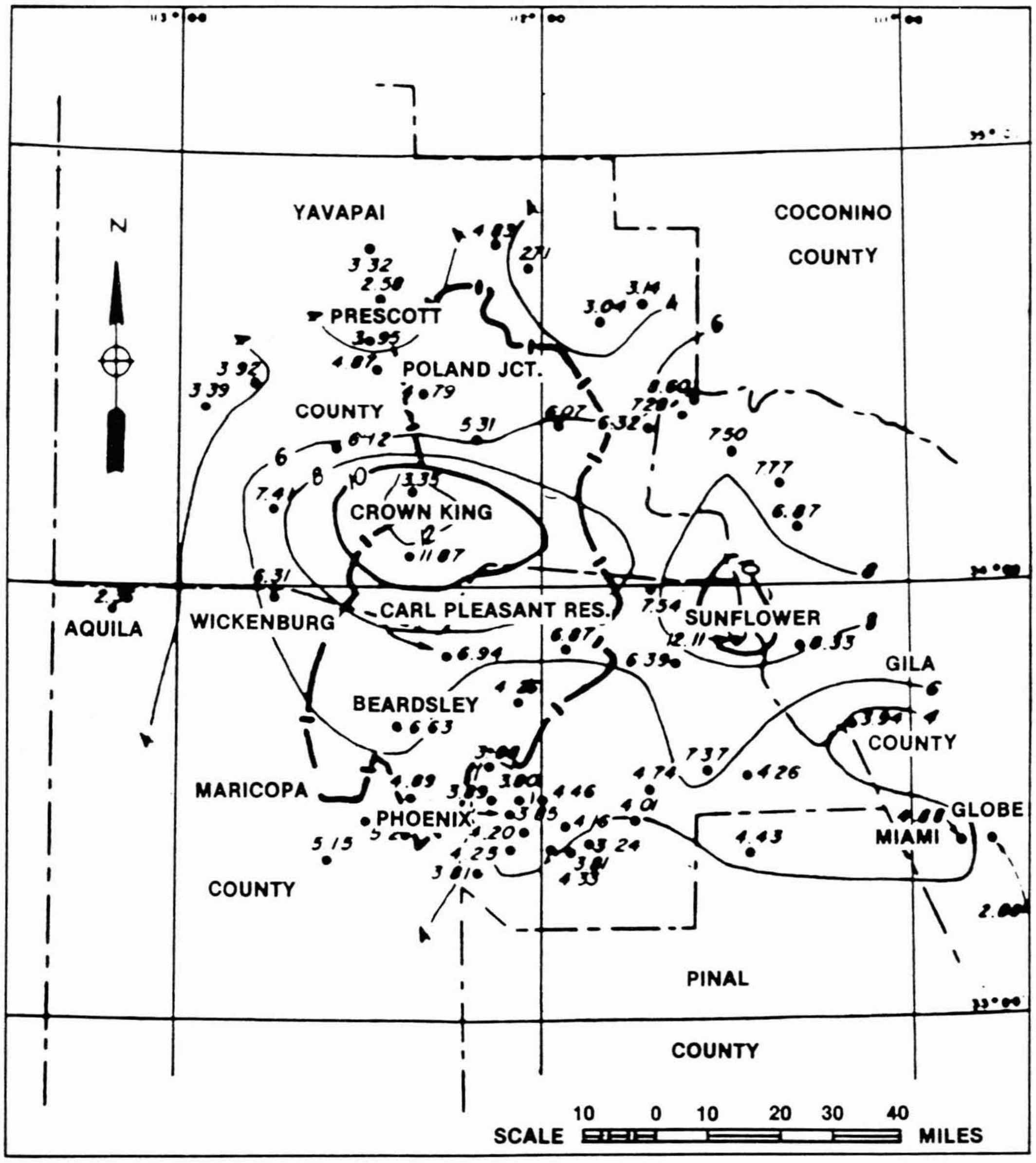


GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

STUDY AREA

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 1



LEGEND

—|— BOUNDARY OF DRAINAGE AREA

—8— LINE OF EQUAL PRECIPITATION IN INCHES

• 6.31 RECORDED PRECIPITATION DEPTH IN INCHES

(SOURCE: REF. 2)

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

ISOHYETAL MAP
STORM OF AUGUST 26-29, 1951

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

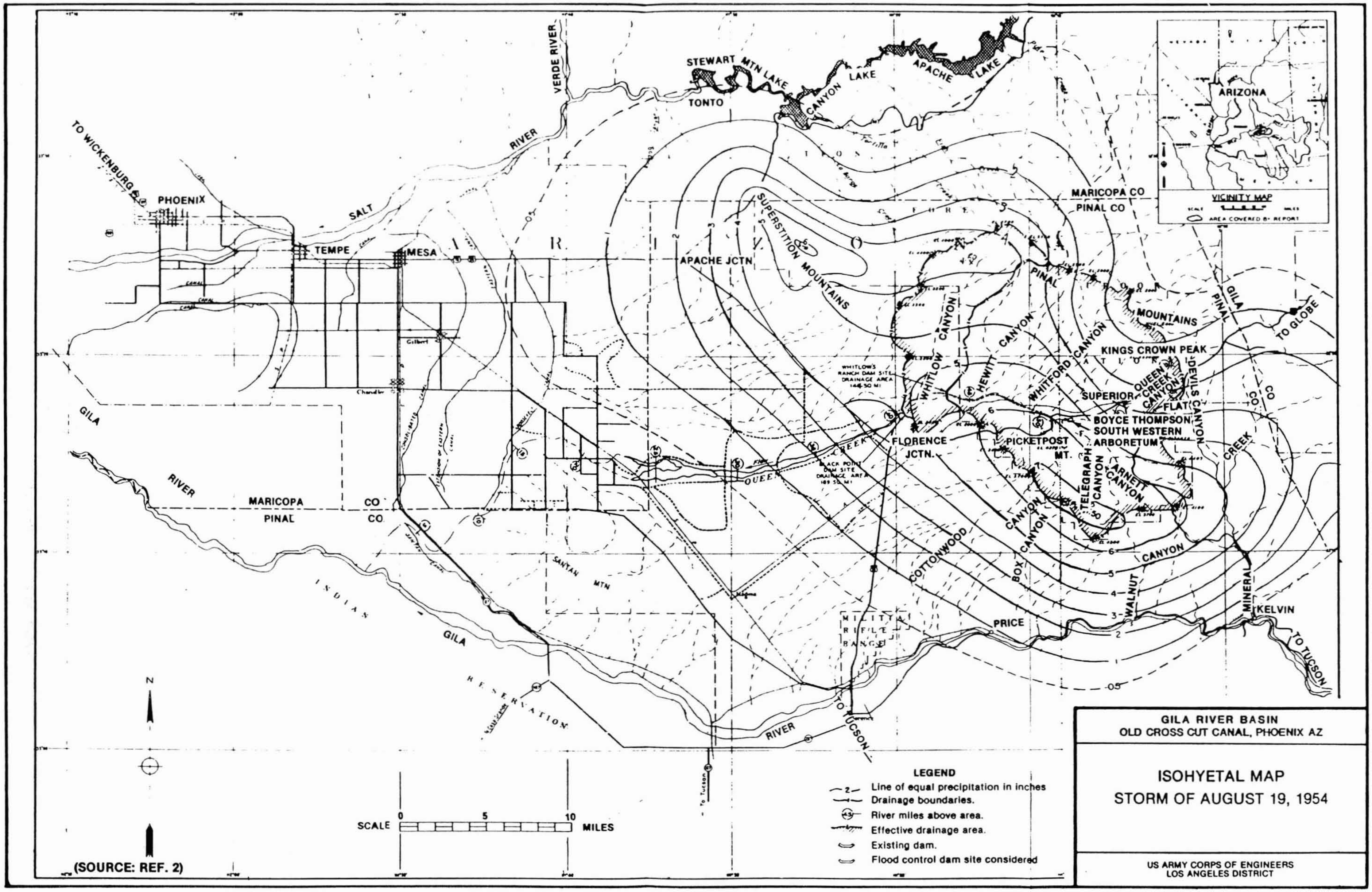
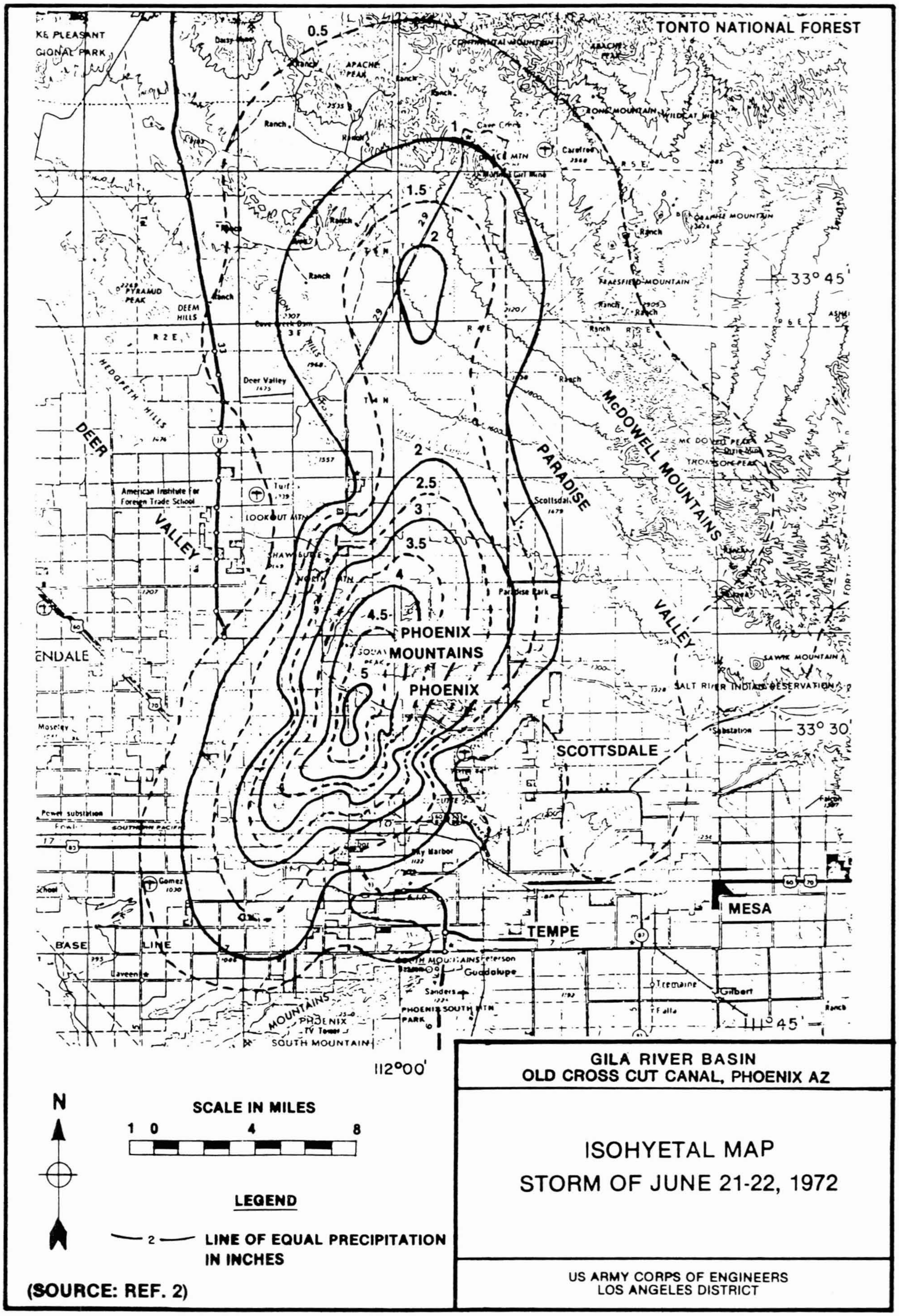

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ
ISOHYETAL MAP
STORM OF AUGUST 19, 1954
US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
LEGEND
Line of equal precipitation in inches
Drainage boundaries.
River miles above area.
Effective drainage area.
Existing dam.
Flood control dam site considered
SCALE 0 5 10 MILES
(SOURCE: REF. 2)
VICINITY MAP
ARIZONA
AREA COVERED BY REPORT
PHOENIX
TEMPE
MESA
TO WICKENBURG
SALT
RIVER
VERDE RIVER
STEWART MTN LAKE
TONTO
CANYON LAKE
APACHE LAKE
MARICOPA CO
PINAL CO
SUPERSTITION MOUNTAINS
APACHE JCTN
PINAL
MOUNTAINS
GILA
PINAL
TO GLOBE
KINGS CROWN PEAK
WHITLOW CANYON
HEWITT CANYON
WHITFORD CANYON
QUEEN CREEK CANYON
DEVILS CANYON
SUPERIOR
FLAT
BOYCE THOMPSON
SOUTH WESTERN
ARBORETUM
PICKETPOST MT.
FLORENCE JCTN.
TELEGRAPH CANYON
ARNETT CANYON
WHITLOWS RANCH DAM SITE DRAINAGE AREA 146.50 MI
BLACK POINT DAM SITE DRAINAGE AREA 189.50 MI
QUEEN CREEK
GILA
RIVER
MARICOPA CO
PINAL CO
SANTAN MTN
INDIAN
GILA
RESERVATION
RIVER
TO TUCSON
MILITARY RIFLE RANGE
COTTONWOOD
BOX CANYON
CANYON
PRICE
WALNUT
CANYON
MINERAL
CREEK
KELVIN
TO TUCSON
CO
Gilbert
Chandler

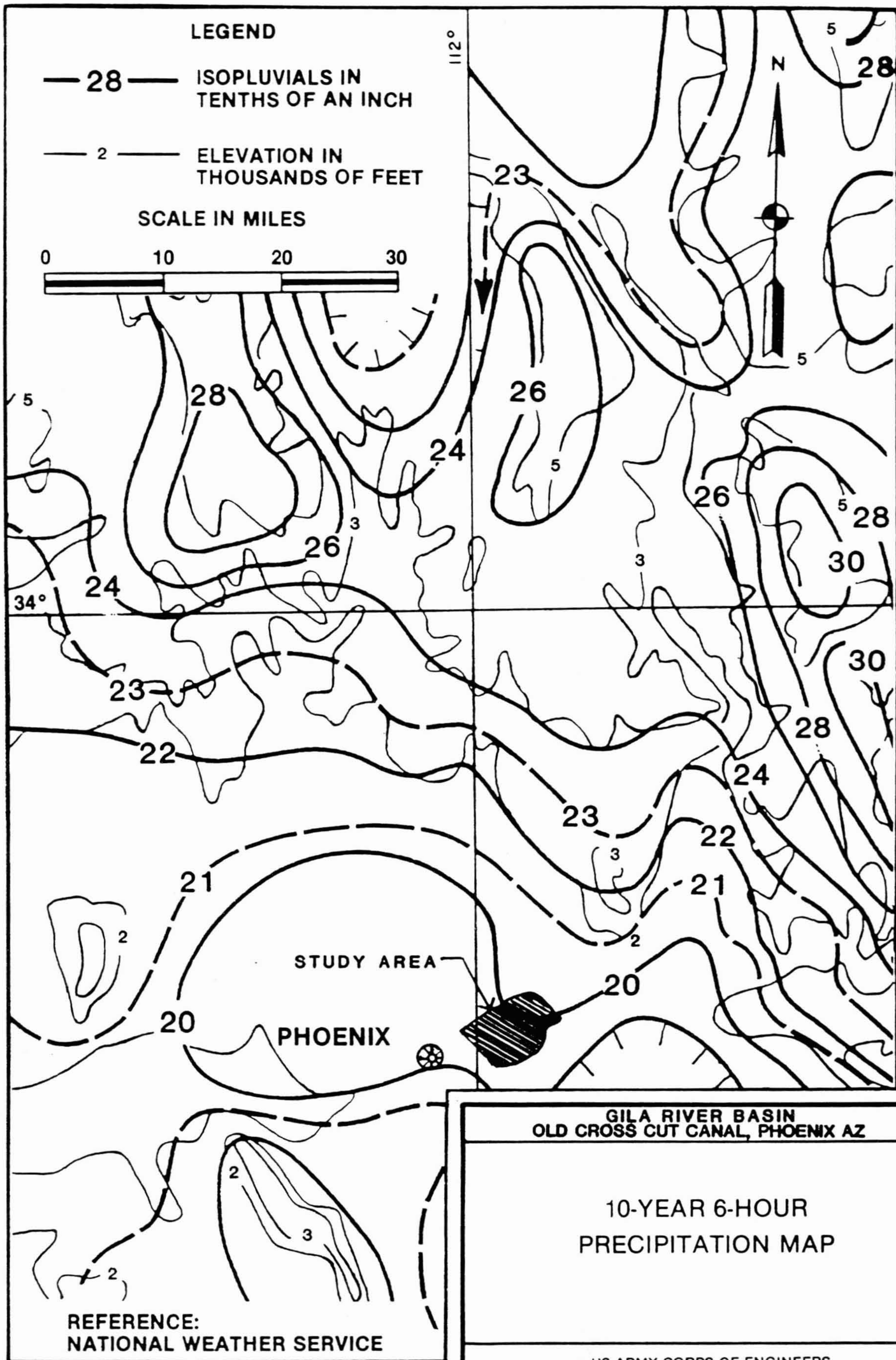
PLATE 3



GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

ISOHYETAL MAP
STORM OF JUNE 21-22, 1972

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



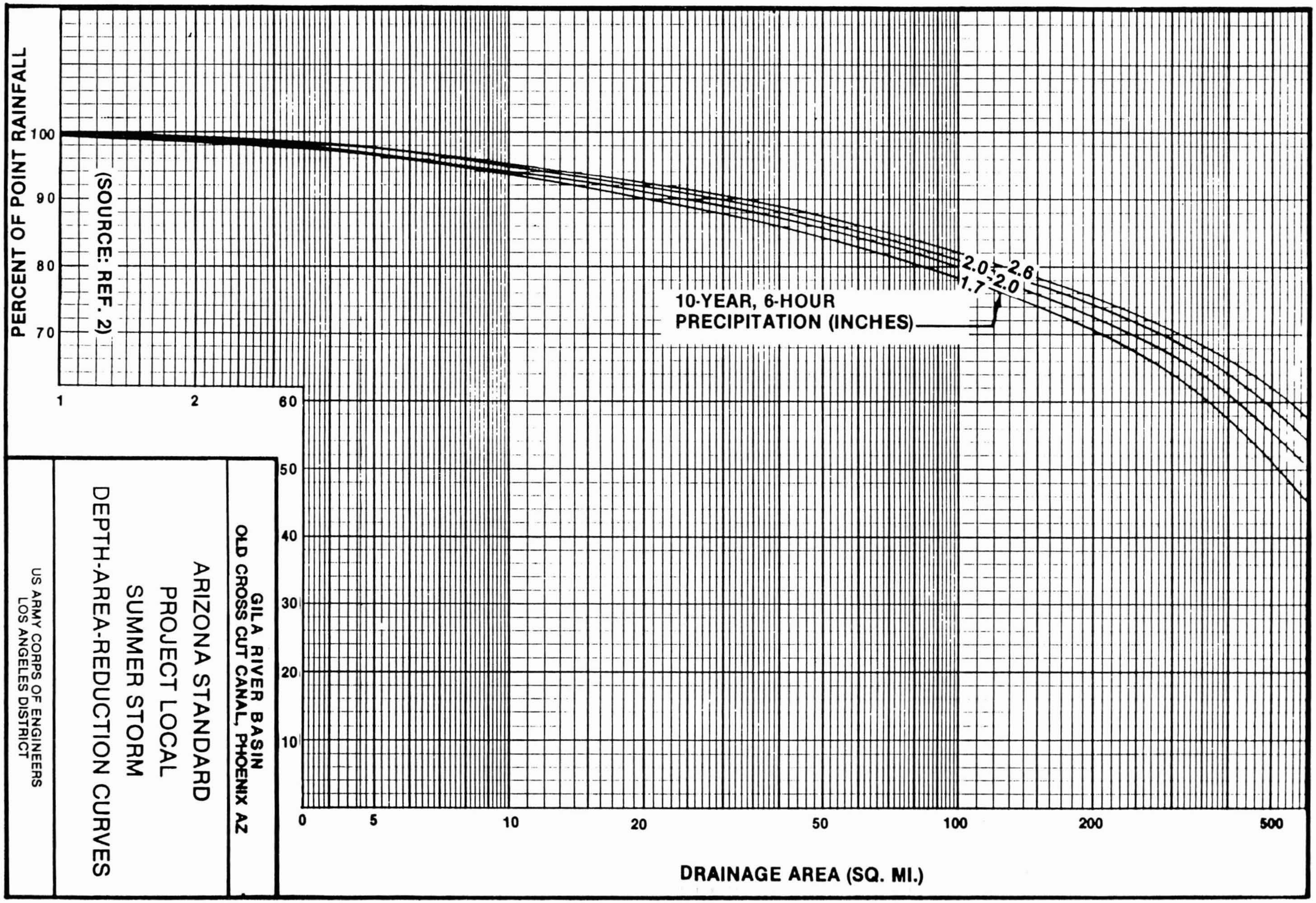
(SOURCE: REF. 2)

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

10-YEAR 6-HOUR
PRECIPITATION MAP

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

(revised 2/83)

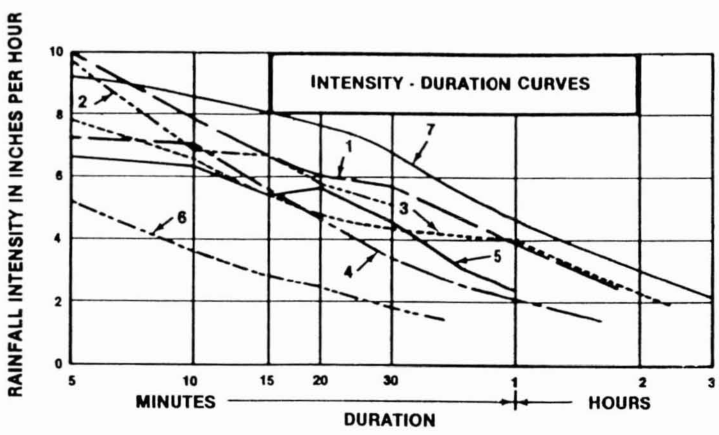


GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

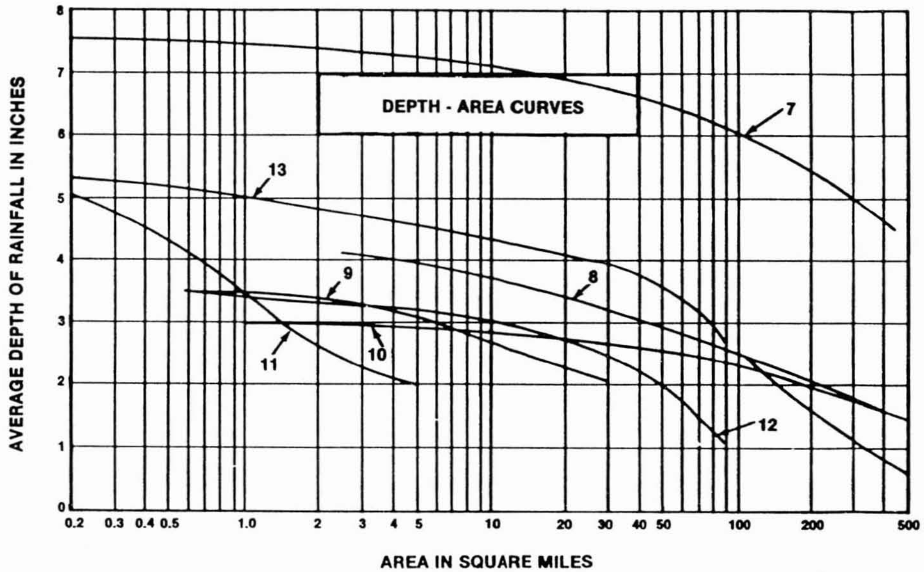
ARIZONA STANDARD
PROJECT LOCAL
SUMMER STORM
DEPTH-AREA-REDUCTION CURVES

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 6



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



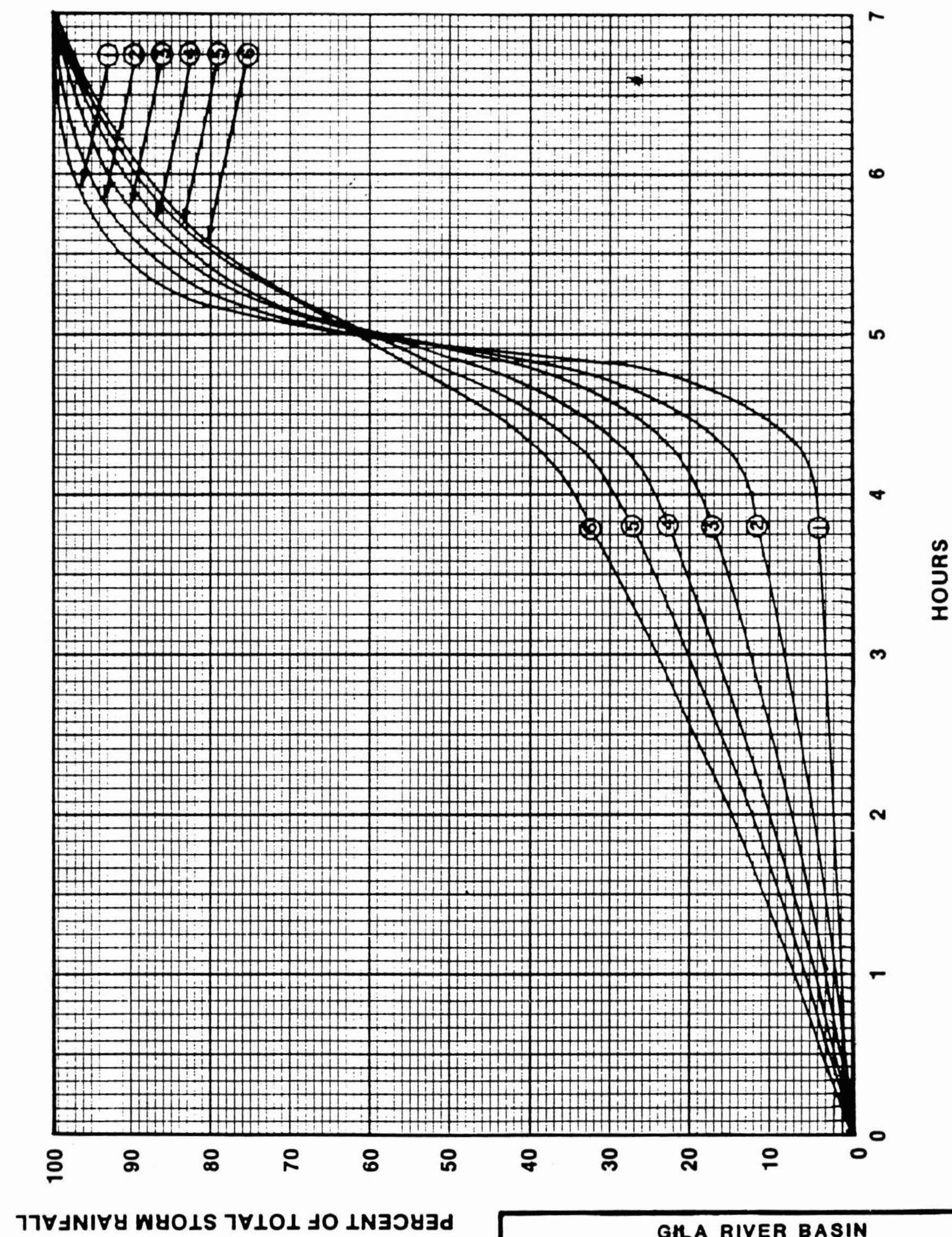
CURVE NO.	STORM			
	LOCATION	DATE	APPROXIMATE DURATION	
			HRS	MIN
1	PARKER CREEK	SEPT. 10, 1933	1	45
2	WALNUT GULCH	OCT. 4-5, 1954	0	30
3	SANTA RITA	JUNE 29, 1959	2	20
4	UNIV. OF ARIZONA	AUG. 13, 1940	1	35
5	TUCSON AIRPORT	SEPT. 24, 1943	1	0
6	PHOENIX	JULY 26, 1936	0	40
7	QUEEN CREEK	AUG. 19, 1954	7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

(SOURCE: REF. 2)

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

INTENSITY-DURATION AND DEPTH-AREA CURVES

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



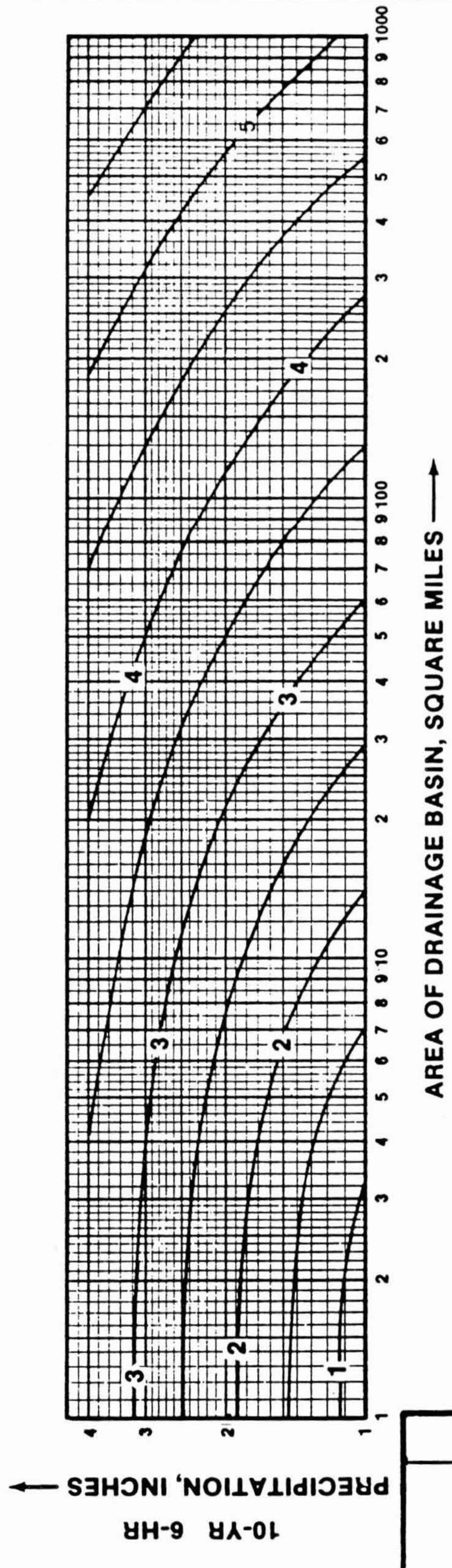
—2— PATTERN NUMBER
MAKE PATTERN NUMBER
SELECTION ON PLATE 20

(SOURCE: REF. 2)

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

ARIZONA
STANDARD PROJECT LOCAL
SUMMER STORM
PRECIPITATION PATTERNS

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



—2— PATTERN NUMBER
REFER TO PLATE 19
FOR ACTUAL PATTERN

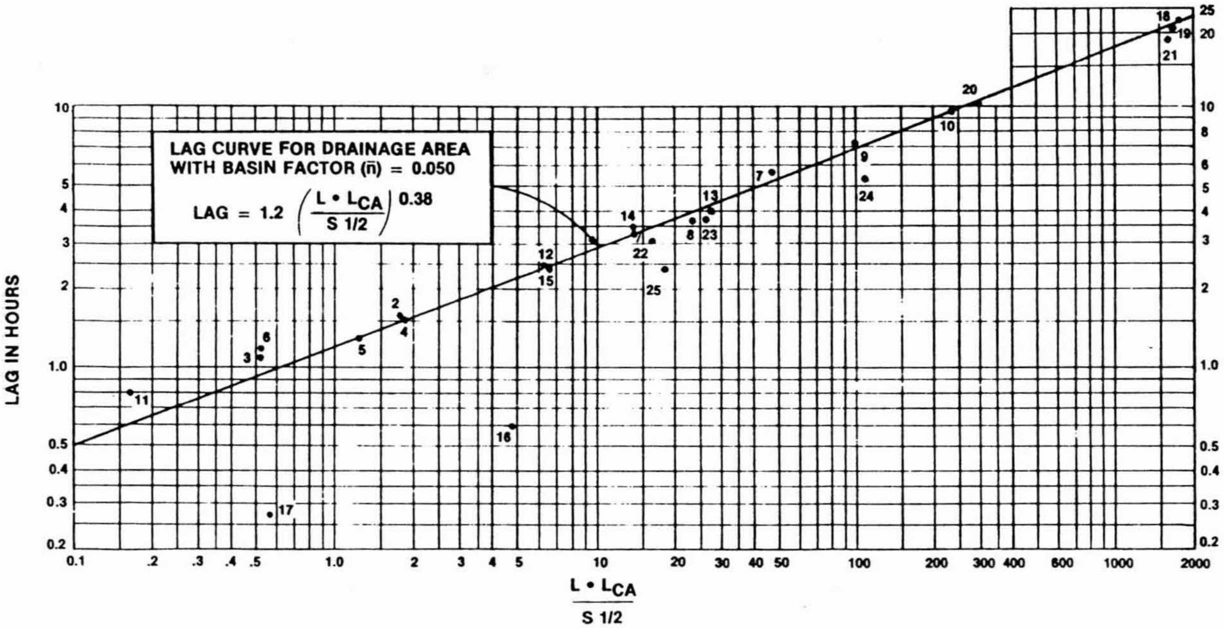
GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

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

(SOURCE: REF. 2)

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

	CONTRIBUTING AREA SQ. MI.	L MILES	L_{ca} MILES	S FT./MI.	LAG HOURS	ESTIMATED $\bar{n}$
1. SAN GABRIEL RIVER AT SAN GABRIEL DAM, CALIF.	162.0	23.2	11.6	350	3.3	0.050
2. WEST FORT SAN GABRIEL RIVER AT COGSWELL DAM, CALIF.	40.4	9.3	4.3	450	1.6	.050
3. SAN ANITA CREEK AT SANTA ANITA DAM, CALIF.	10.8	5.8	2.5	690	1.1	.050
4. SAN DIMAS CREEK AT SAN DIMAS DAM, CALIF.	16.2	8.6	4.8	440	1.5	.050
5. EATON WASH AT EATON WASH DAM, CALIF.	9.5	7.3	4.4	600	1.3	.050
6. SAN ANTONIO CREEK NEAR CLAREMONT, CALIF.	16.9	5.9	3.0	1017	1.2	.055
7. SANTA CLARA RIVER NEAR SAUGUS, CALIF.	355.0	36.0	15.8	140	5.6	.050
8. TEMECULA CREEK AT PAUBA CANYON, CALIF.	168.0	26.0	11.3	150	3.7	.050
9. SANTA MARGARITA RIVER NEAR FALLBROOK, CALIF.	645.0	46.0	22.0	105	7.3	.055
10. SANTA MARGARITA RIVER AT YSIDORA, CALIF.	740.0	61.2	34.3	85	9.5	.055
11. LIVE OAK CREEK AT LIVE OAK DAM, CALIF.	2.3	2.9	1.5	700	0.8	.070
12. TUJUNGA CREEK AT BIG TUJUNGA DAM, CALIF.	81.4	15.1	7.3	290	2.5	.050
13. MURRIETA CREEK AT TEMECULA, CALIF.	220.0	27.2	10.3	95	4.0	.050
14. LOS ANGELES RIVER AT SEPULVEDA DAM, CALIF.	152.0	19.0	9.0	145	3.5	.050
15. PACOIMA WASH AT PACOIMA DAM, CALIF.	27.8	15.0	8.0	315	2.4	.050
16. ALHAMBRA WASH ABOVE SHORT STREET, CALIF.	14.0	9.5	4.6	85	0.6	.015
17. BROADWAY DRAIN ABOVE RAYMOND DIKE, CALIF.	2.5	3.4	1.7	100	0.28	.015
18. GILA RIVER AT CONNOR NO. 4 DAM SITE, ARIZ.	2840.0	131.0	71.0	29	21.5	.050
19. SAN FRANCISCO RIVER AT JUNCTION WITH BLUE RIVER, ARIZ.	2000.0	30.0	74.0	32	20.6	.050
20. BLUE RIVER NEAR CLIFTON, ARIZ.	790.0	77.0	37.0	65	10.3	.050
21. SALT RIVER NEAR ROOSEVELT, ARIZ.	4310.0	160.0	66.0	45	18.6	.050
22. NEW RIVER AT ROCK SPRINGS, ARIZ.	67.3	20.2	9.7	141	3.1	.045
23. NEW RIVER AT NEW RIVER, ARIZ.	85.7	23.2	13.6	145	3.7	.045
24. NEW RIVER AT BELL ROAD, ARIZ.	187.0	47.6	20.7	83	5.3	.037
25. SKUNK CREEK NEAR PHOENIX, ARIZ.	64.6	17.6	10.0	89	2.4	.033



(SOURCE: REF. 2)

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 GENERALY 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 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.

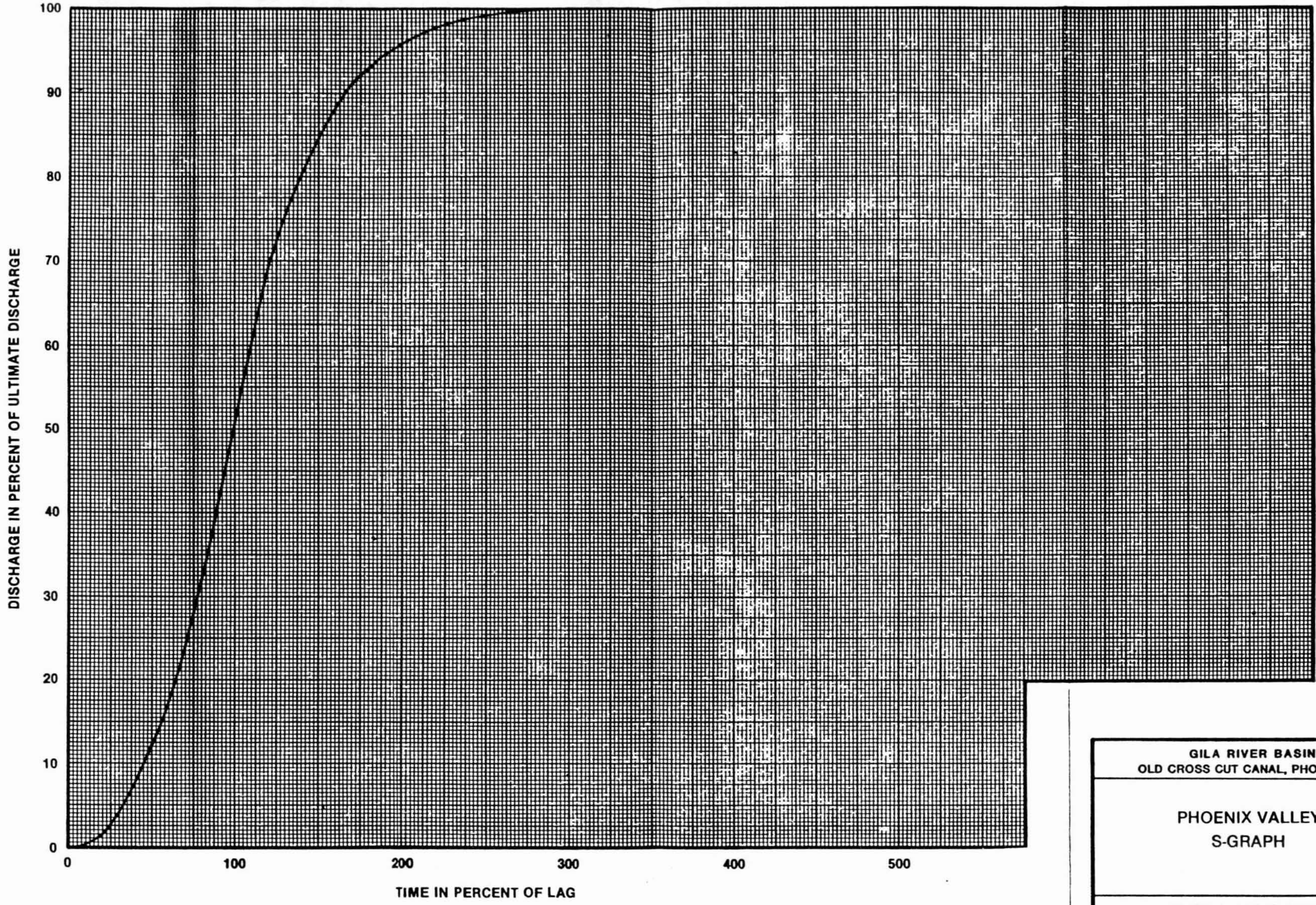
NOTE:
TO OBTAIN THE LAG (IN HOURS) FOR ANY AREA, MULTIPLY THE LAG OBTAINED FROM THE CURVE BY:

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

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIZ AZ

LAG RELATIONSHIPS

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

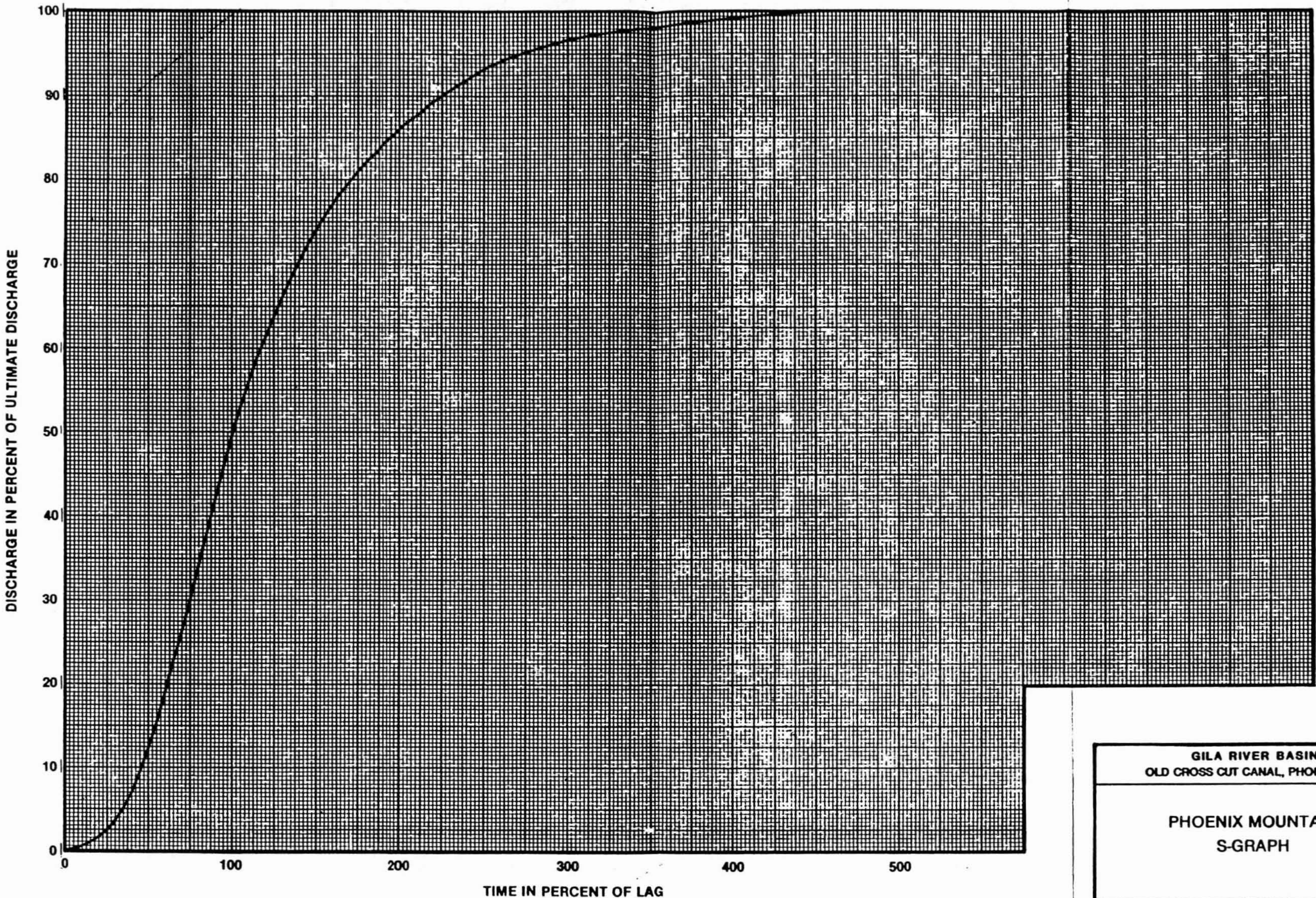


(SOURCE: REF. 2)

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

PHOENIX VALLEY
S-GRAPH

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

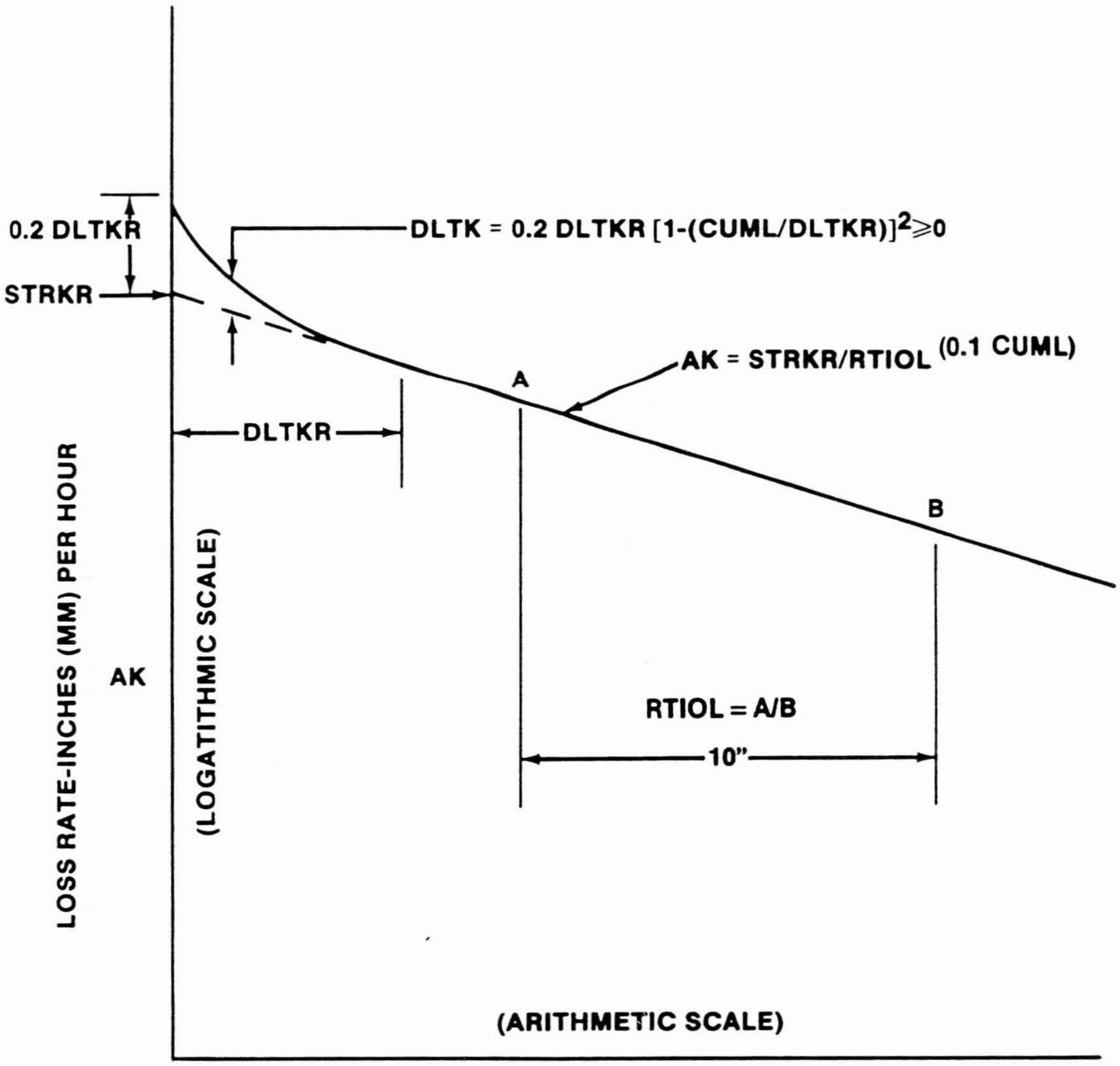


(SOURCE: REF. 2)

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

PHOENIX MOUNTAIN
S-GRAPH

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



ACCUMULATED LOSS (CUML)-INCHES (MM)

ALOSS = (AK + DKTK) PRCP^{ERAIN}

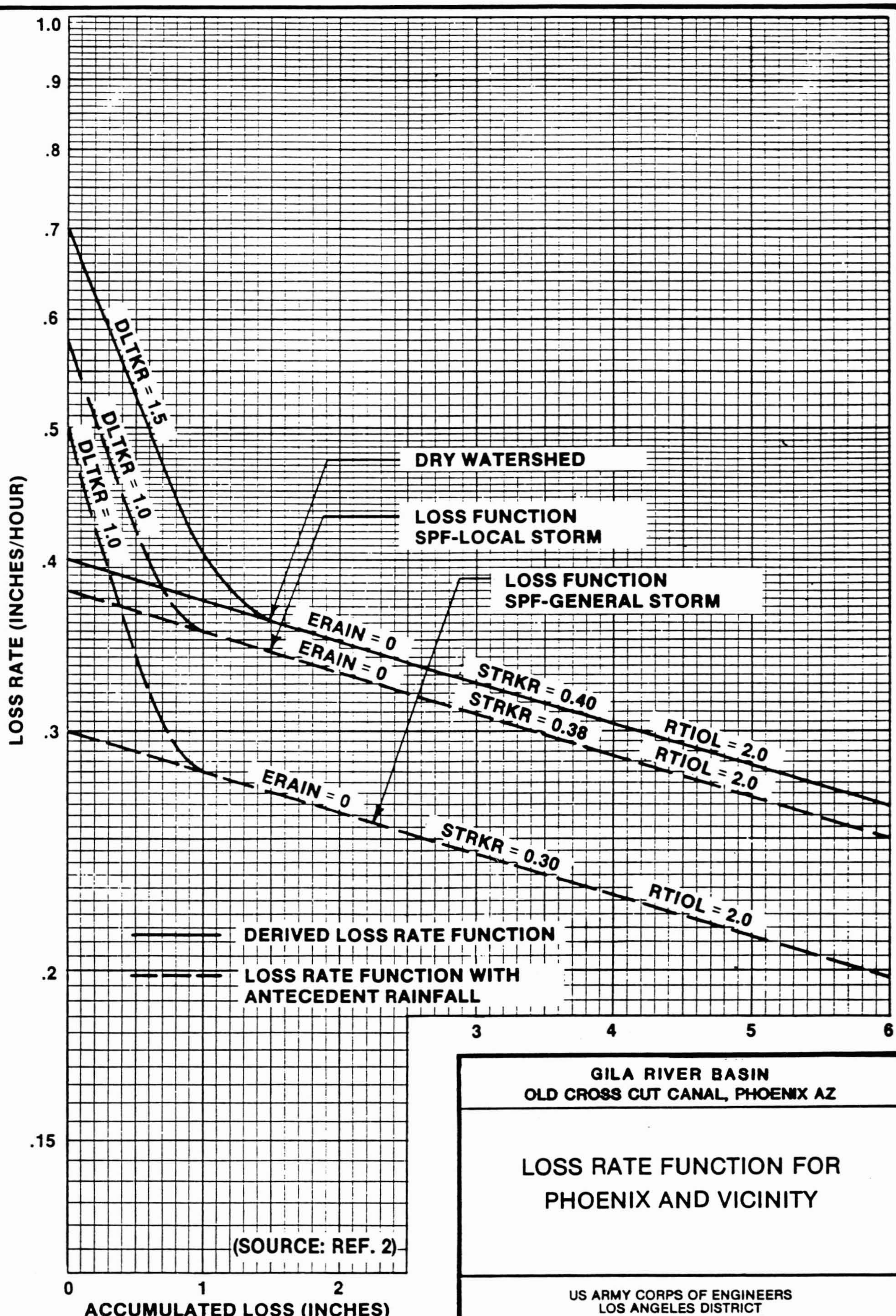
NOTE: VARIABLES ARE DEFINED IN PARA. 4.04 C OF TEXT.

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

H.E.C. LOSS RATE FUNCTION

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

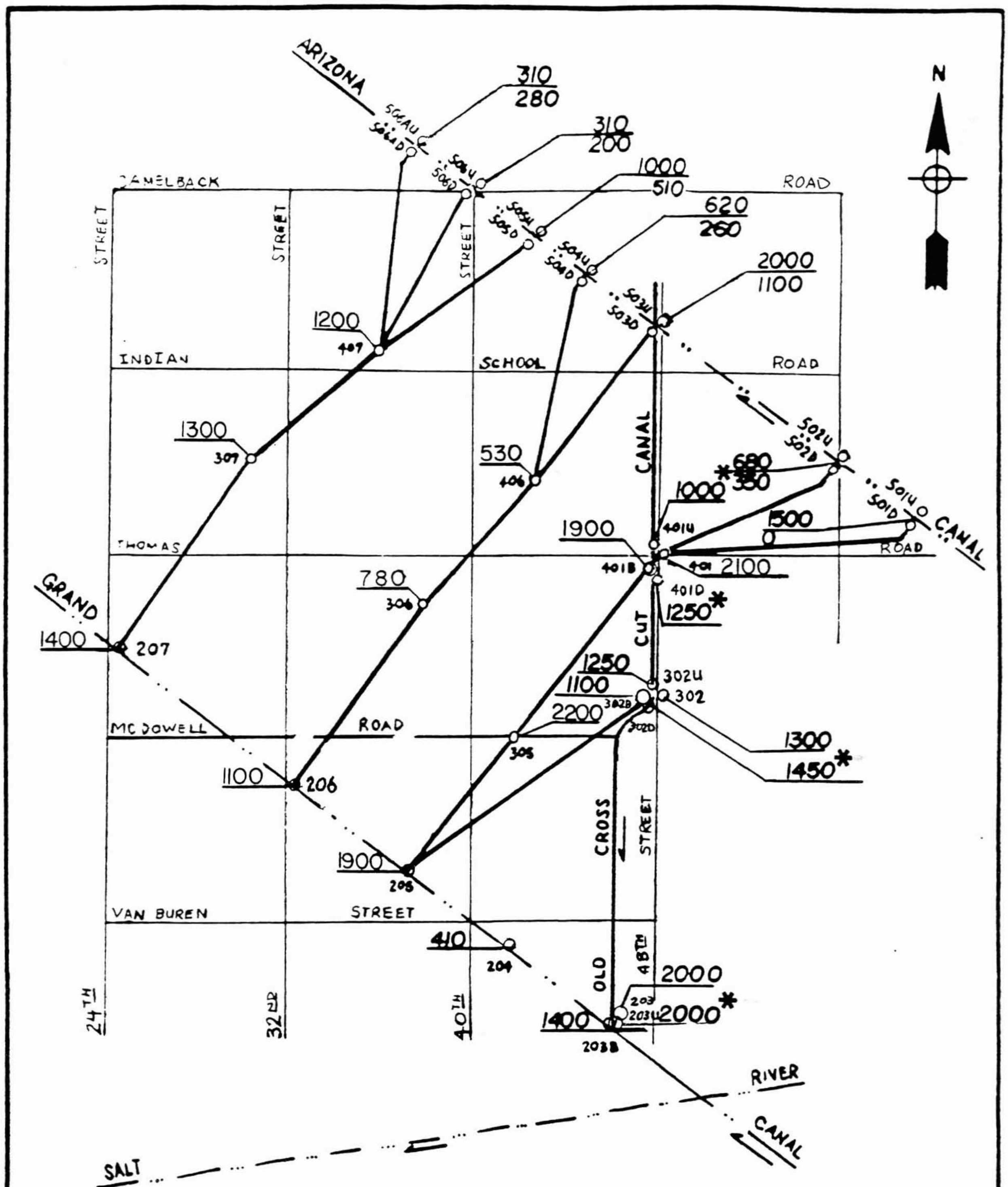
(SOURCE: REF. 2)



GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

LOSS RATE FUNCTION FOR
PHOENIX AND VICINITY

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



-- NOT TO SCALE --

LEGEND

810	100 YEAR FLOOD ft^3/s
203 O	CONCENTRATION POINT
——	STREET
—··—	CANAL
▬▬	ROUTING LINE

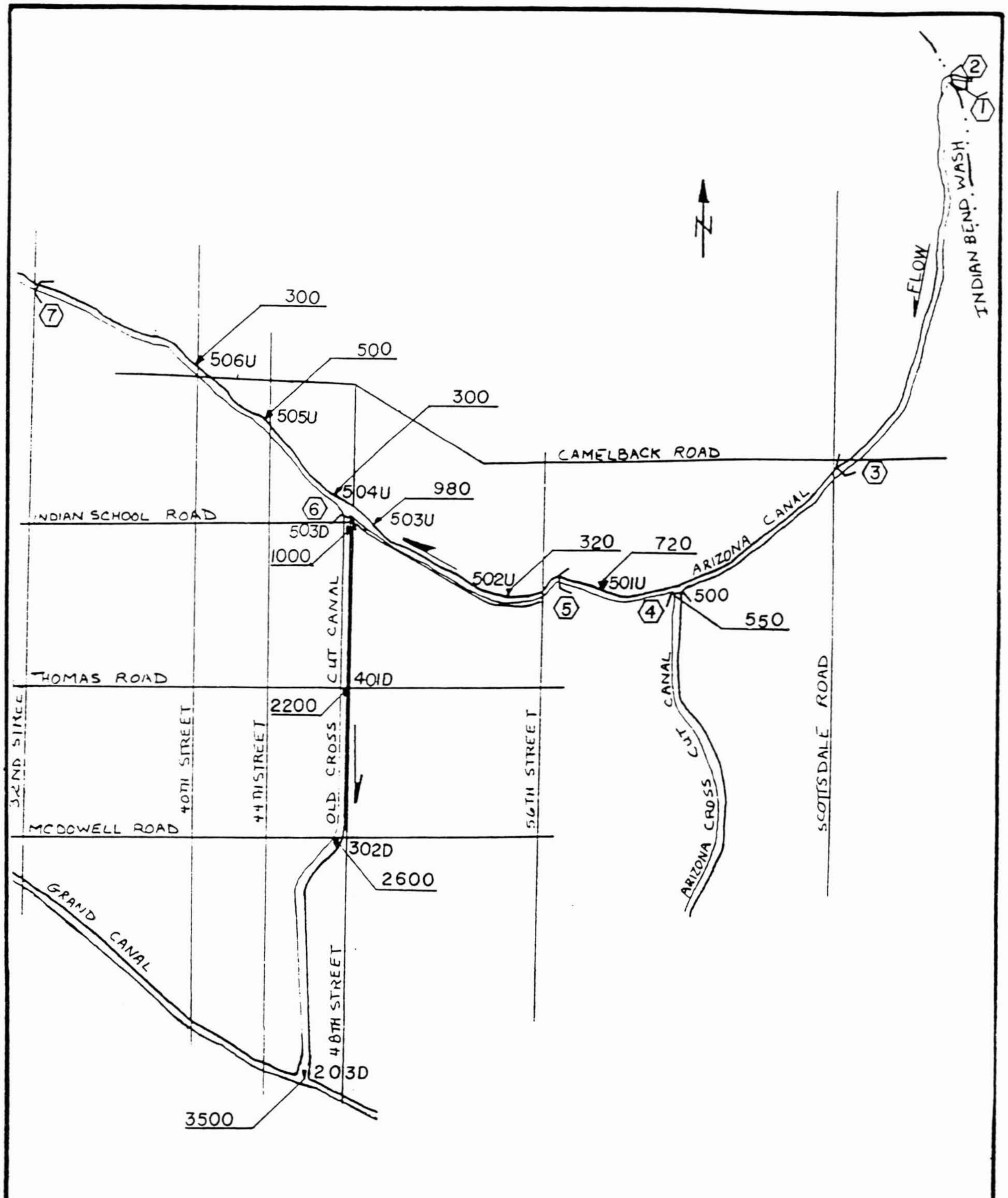
* Old Cross Cut Canal capacity at breakouts.

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

SCHEMATIC OF EXISTING CONDITIONS

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS



LEGEND

300 506U 25 YR, ALT 2, 7HR OPERATION CONCENTRATION POINT

5 CANAL GATE

-- NOT TO SCALE --

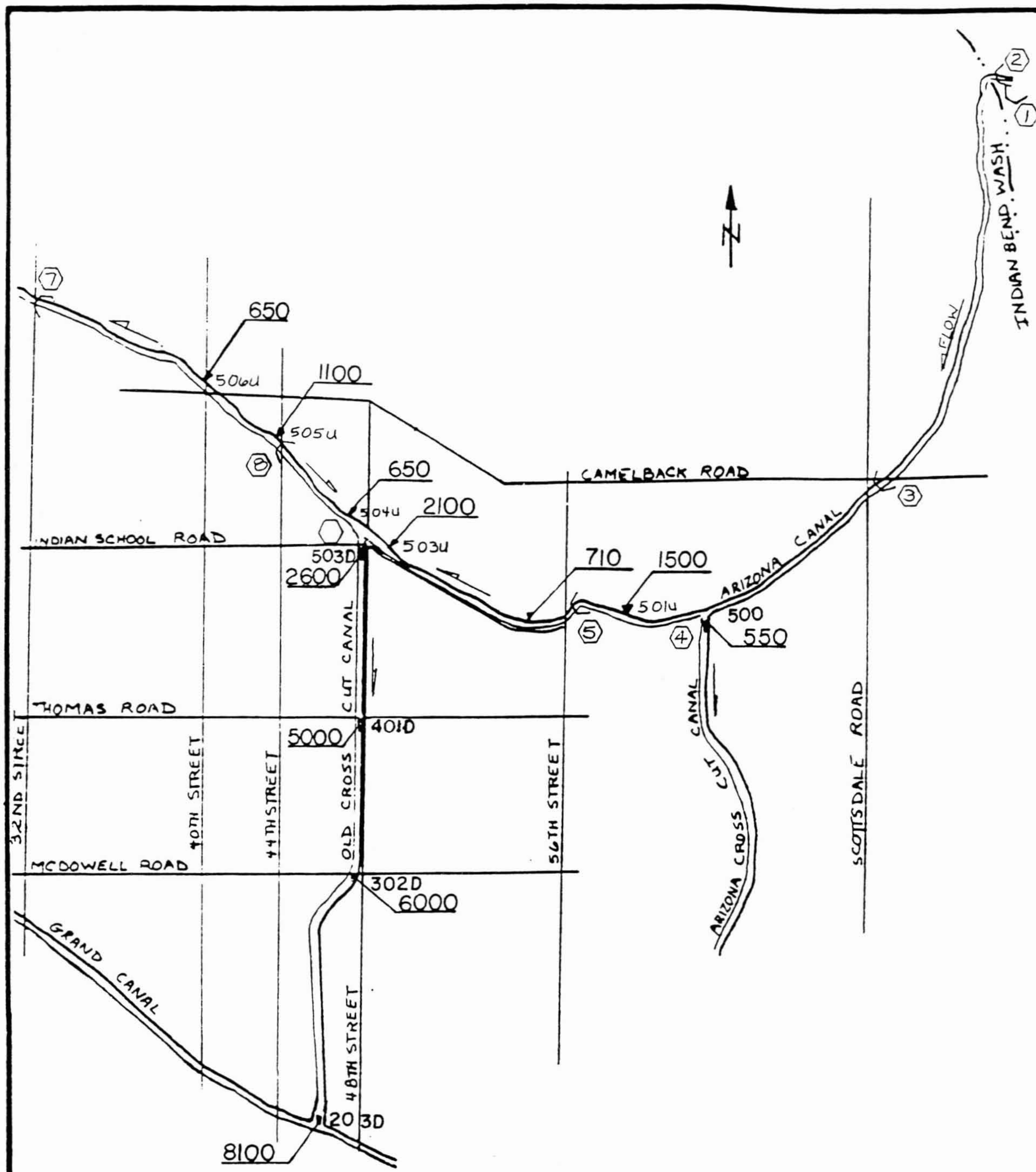
GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

SCHEMATIC OF

ALTERNATIVES 1 AND 2

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

PLATE 16



LEGEND

300
506U — 100 YR, ALT 4 CONCENTRATION POINT

4 — CANAL GATE

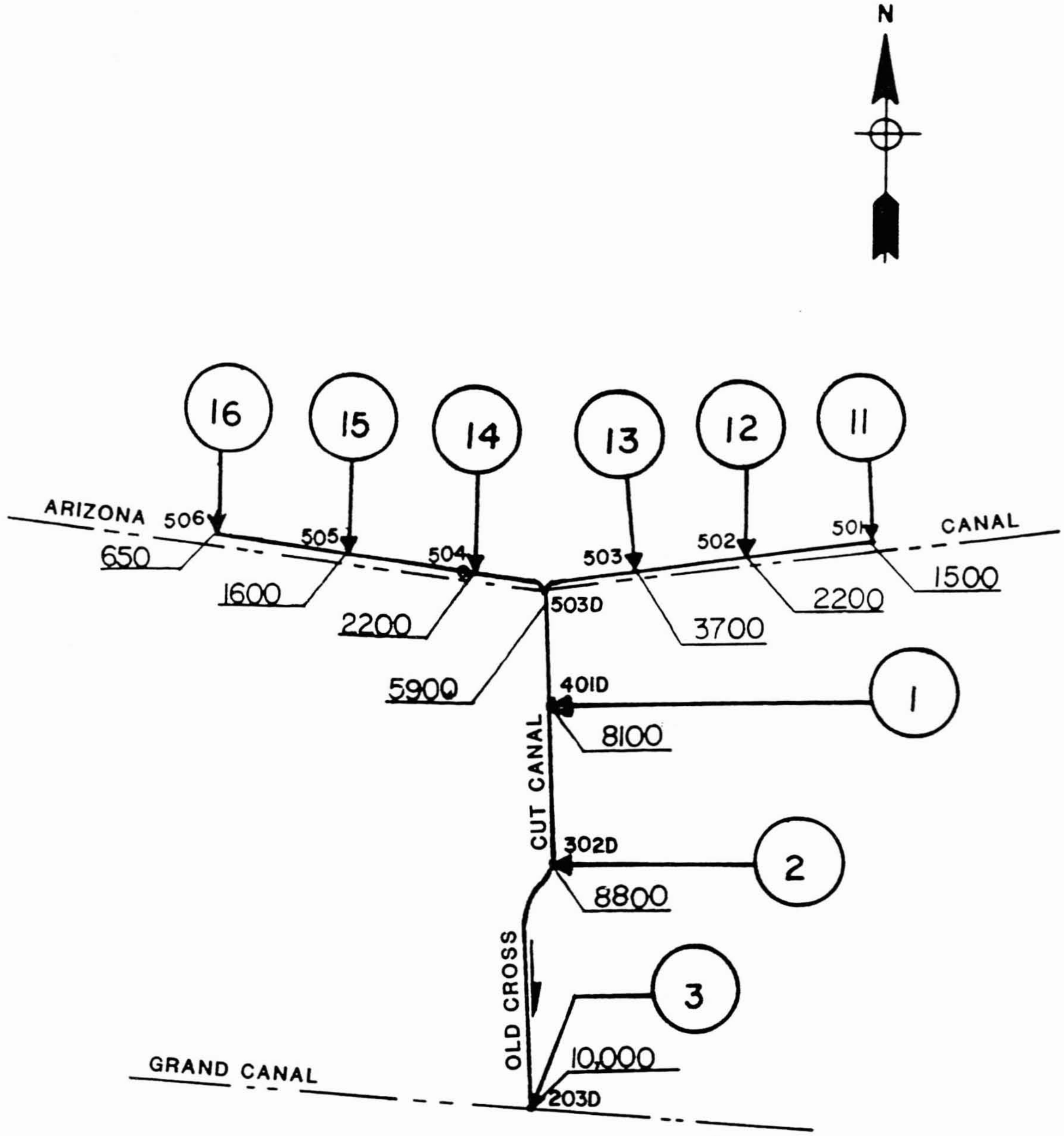
-- NOT TO SCALE --

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

SCHEMATIC OF

ALTERNATIVES 3 AND 4

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



LEGEND

ROUTING LINE

3 CONTRIBUTING SUBAREA

650 ALT. 6 100-YEAR FLOW (FT^3/S)

506 CONCENTRATION POINT

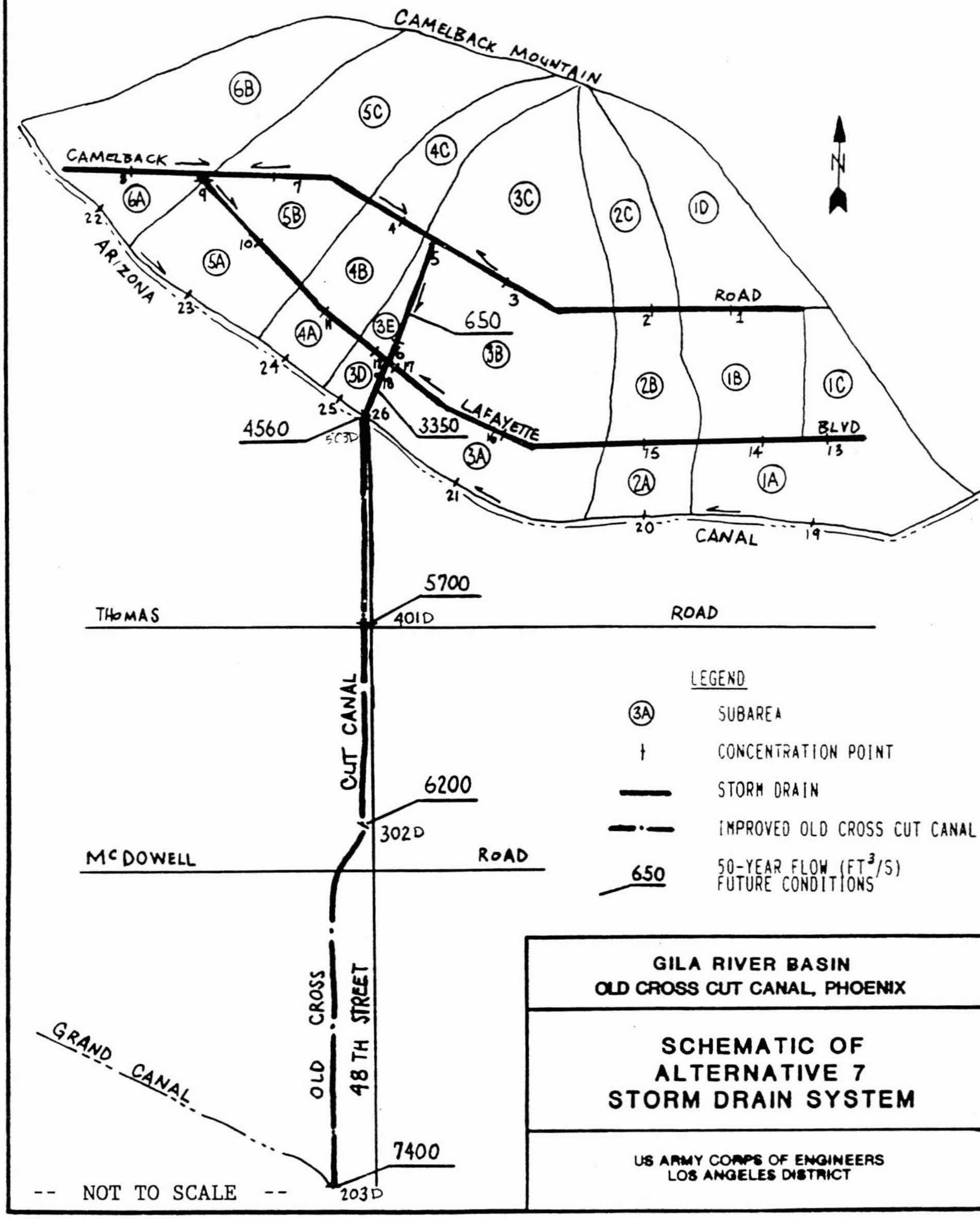
CANAL

-- NOT TO SCALE --

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

SCHEMATIC OF
ALTERNATIVE 5 & 6

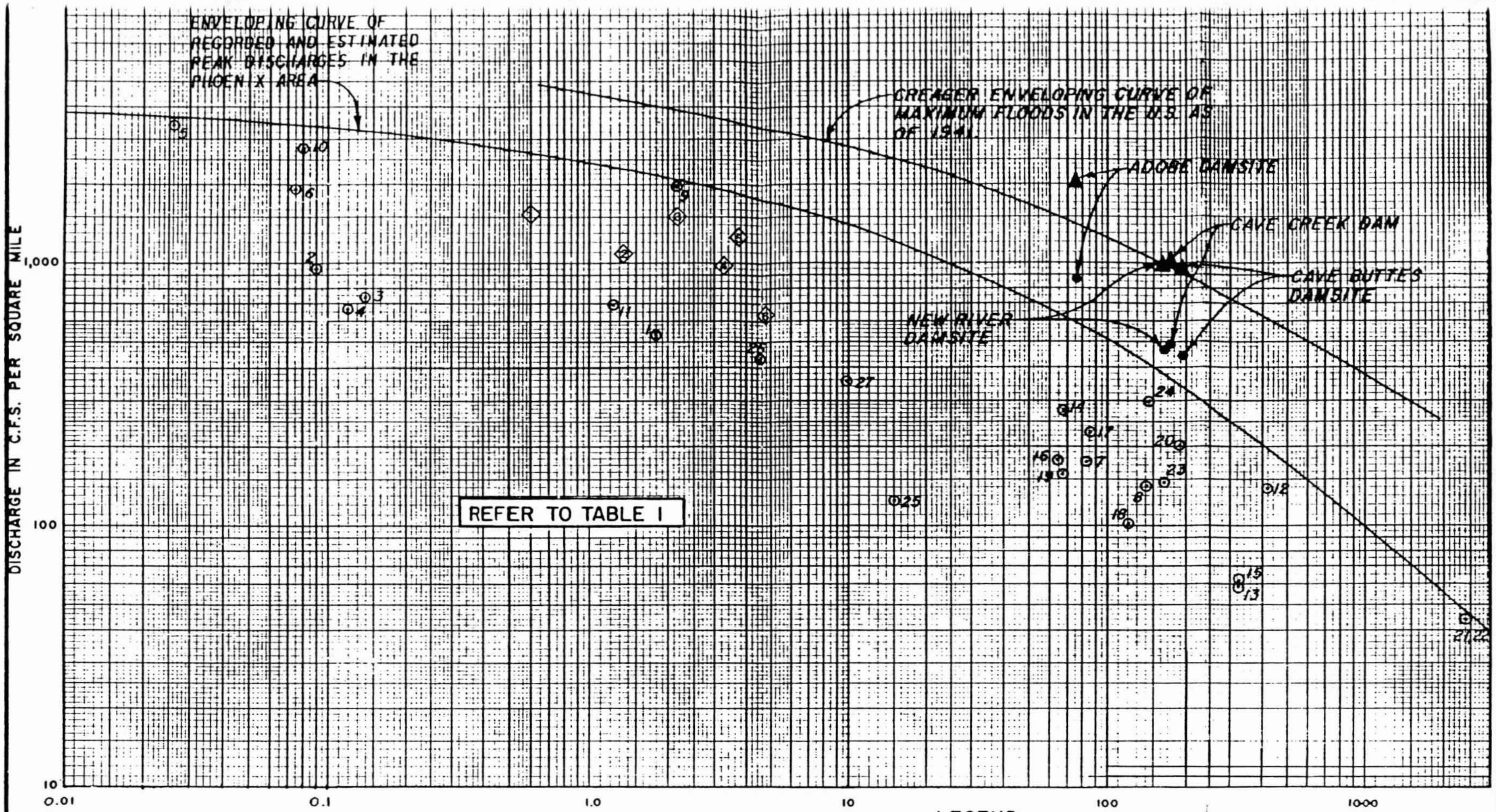
US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX

SCHEMATIC OF ALTERNATIVE 7 STORM DRAIN SYSTEM

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



OLD CROSS CUT CANAL
SPF WITHOUT PROJECT

1 Washington & Grand Canal CP204
2 56th Street u/s of A.C. CP502U
3 36th Street & Devonshire Ave. CP407
4 30th Street & Mitchel Ave. CP 307
5 Thomas Road at O.C.C. CP401
6 24th Street & Grand Canal CP207

LEGEND

⊙1 PEAK DISCHARGE FOR MAJOR RECORDED FLOOD (SEE TABLE 15 REF. 2)

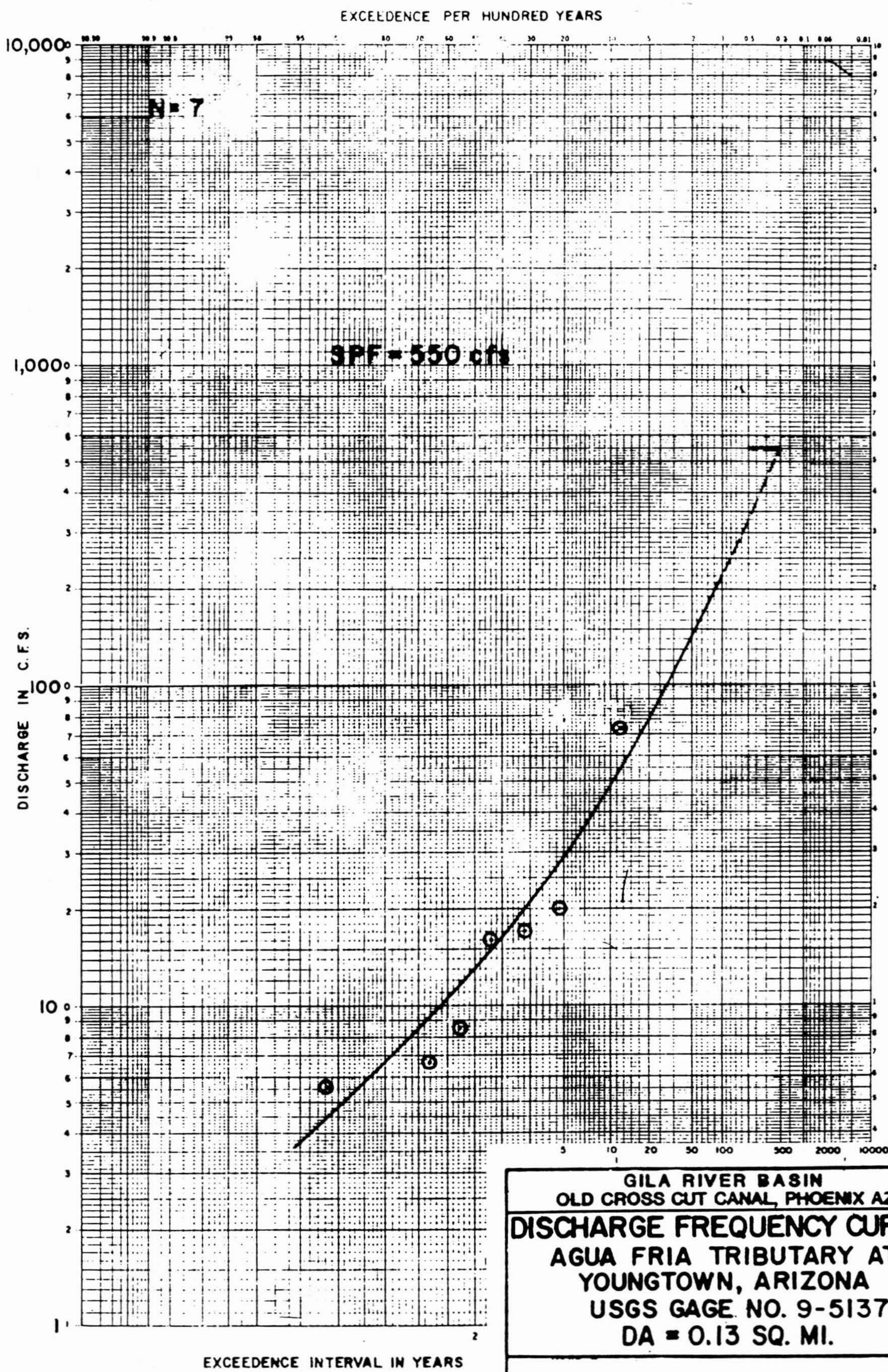
● STANDARD PROJECT PEAK DISCHARGE AT DAM OR DAMSITE.

▲ PROBABLE MAXIMUM PEAK DISCHARGE AT DAM OR DAMSITE.

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

ENVELOPING CURVE OF PEAK DISCHARGES

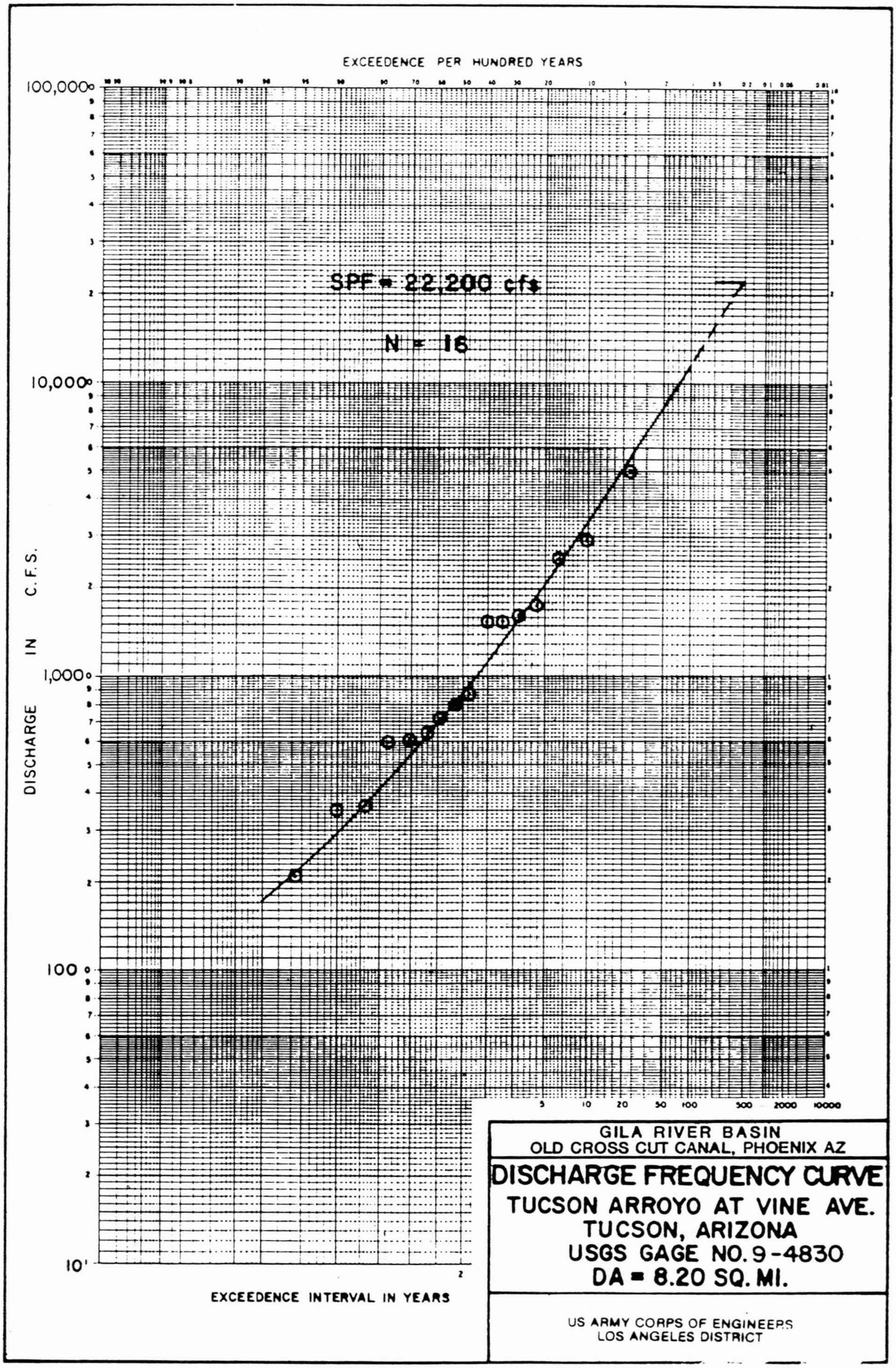
US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

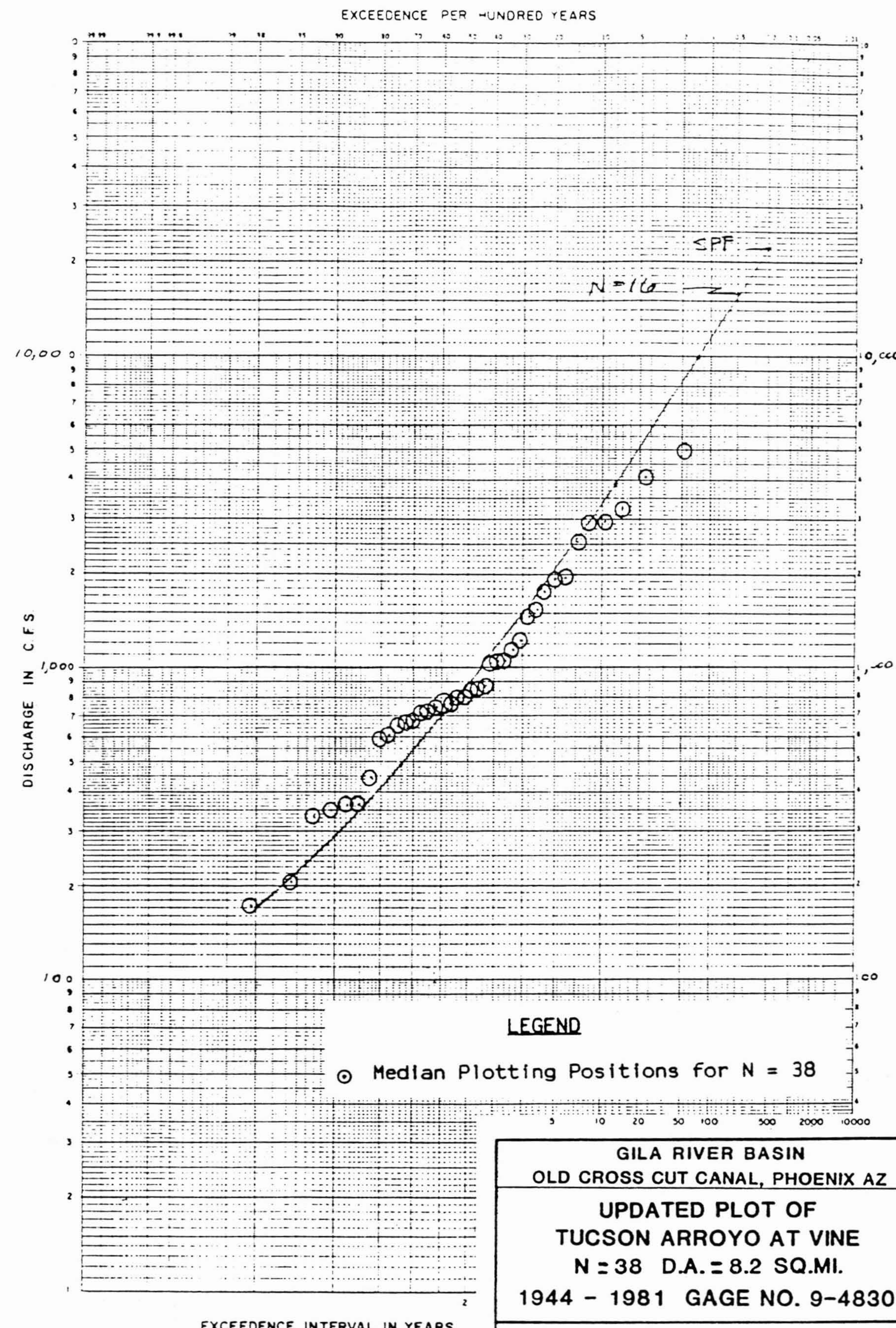


GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

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

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

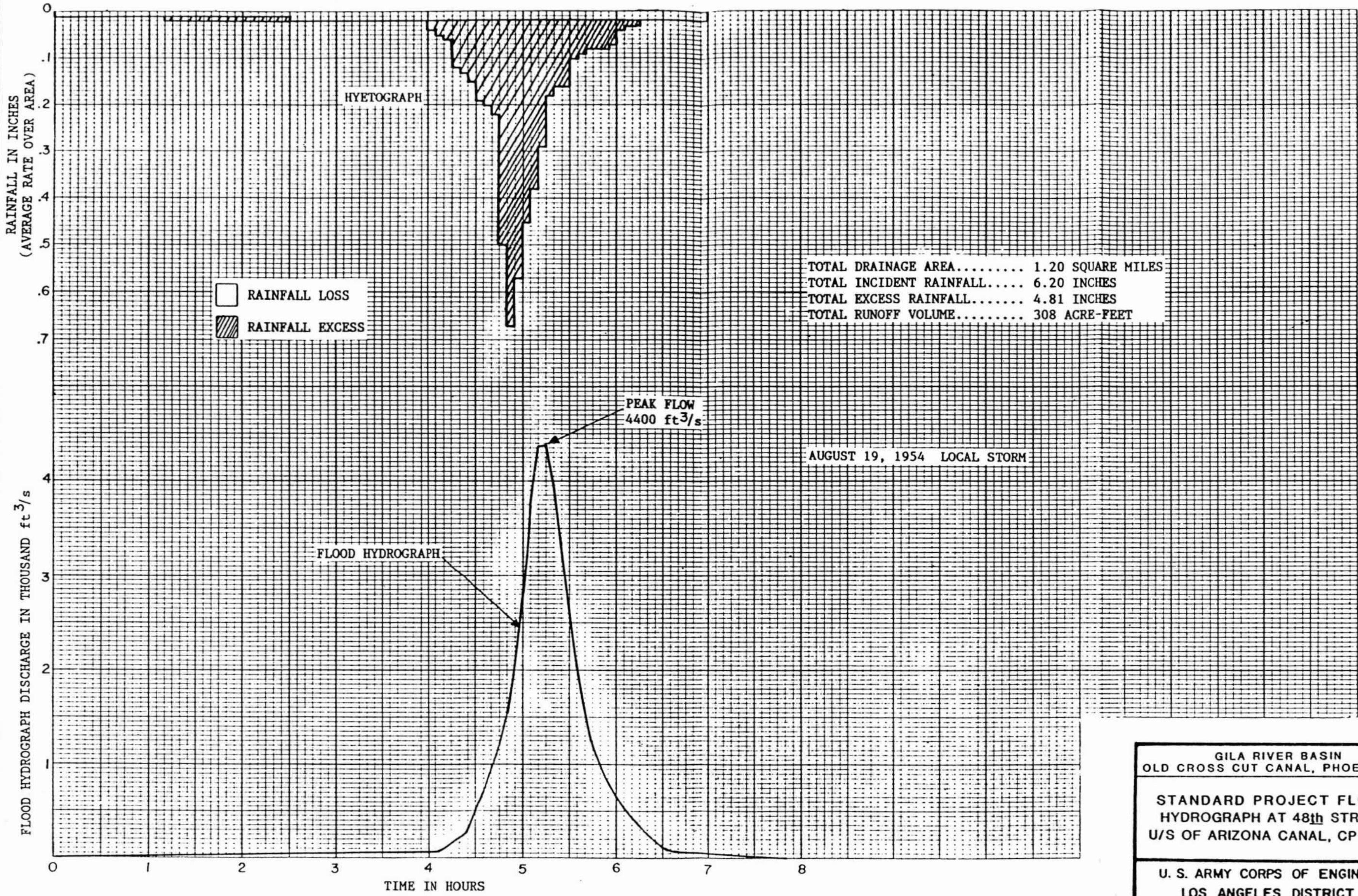

EXCEEDENCE PER HUNDRED YEARS
100,000
10,000
1,000
100
10
DISCHARGE IN C.F.S.
SPF = 22,200 cfs
N = 16
EXCEEDENCE INTERVAL IN YEARS
GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ
DISCHARGE FREQUENCY CURVE
TUCSON ARROYO AT VINE AVE.
TUCSON, ARIZONA
USGS GAGE NO. 9-4830
DA = 8.20 SQ. MI.
US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

UPDATED PLOT OF
TUCSON ARROYO AT VINE
N = 38 D.A. = 8.2 SQ.MI.
1944 - 1981 GAGE NO. 9-4830

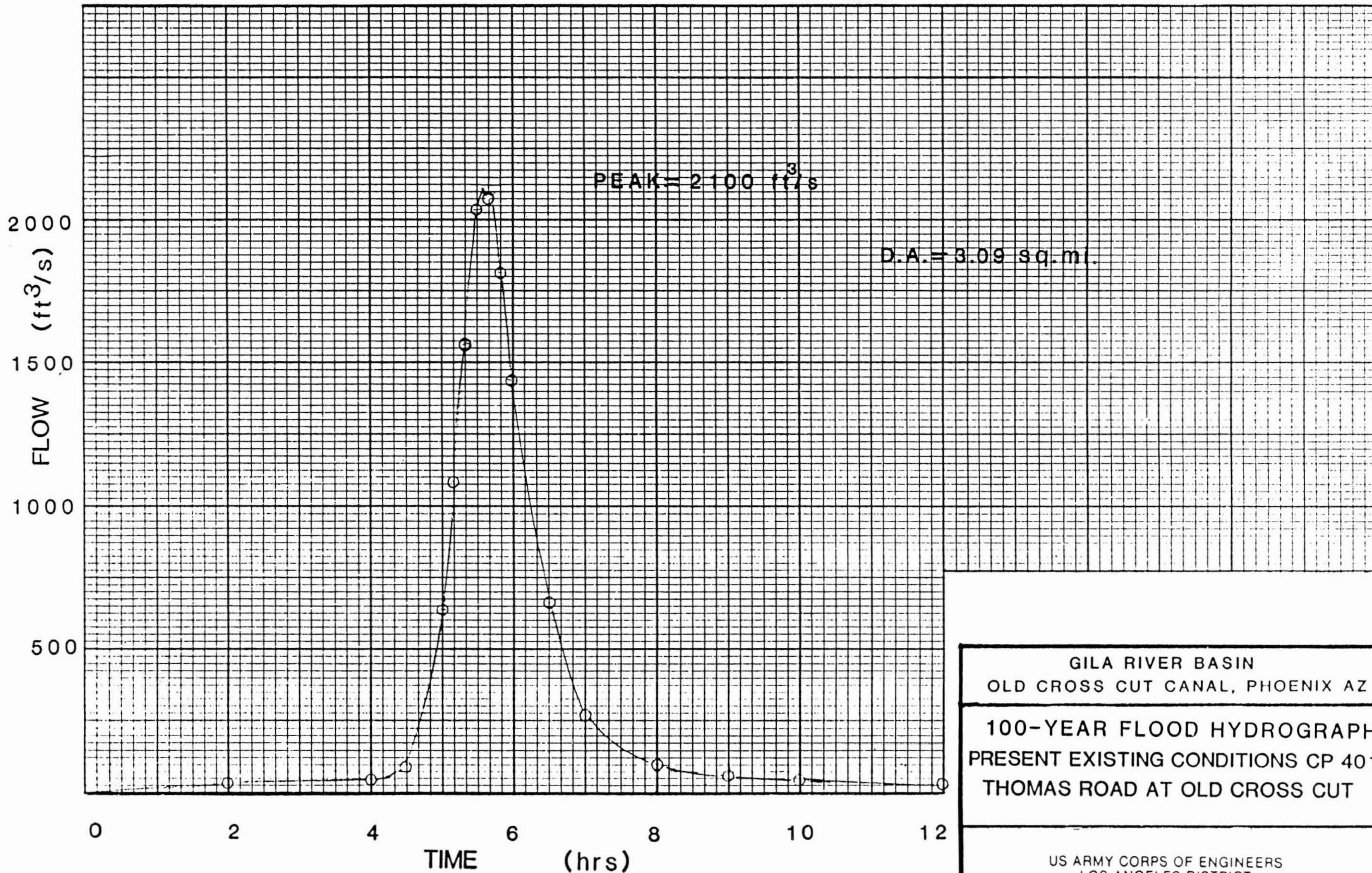
US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

STANDARD PROJECT FLOOD
HYDROGRAPH AT 48th STREET
U/S OF ARIZONA CANAL, CP 503U

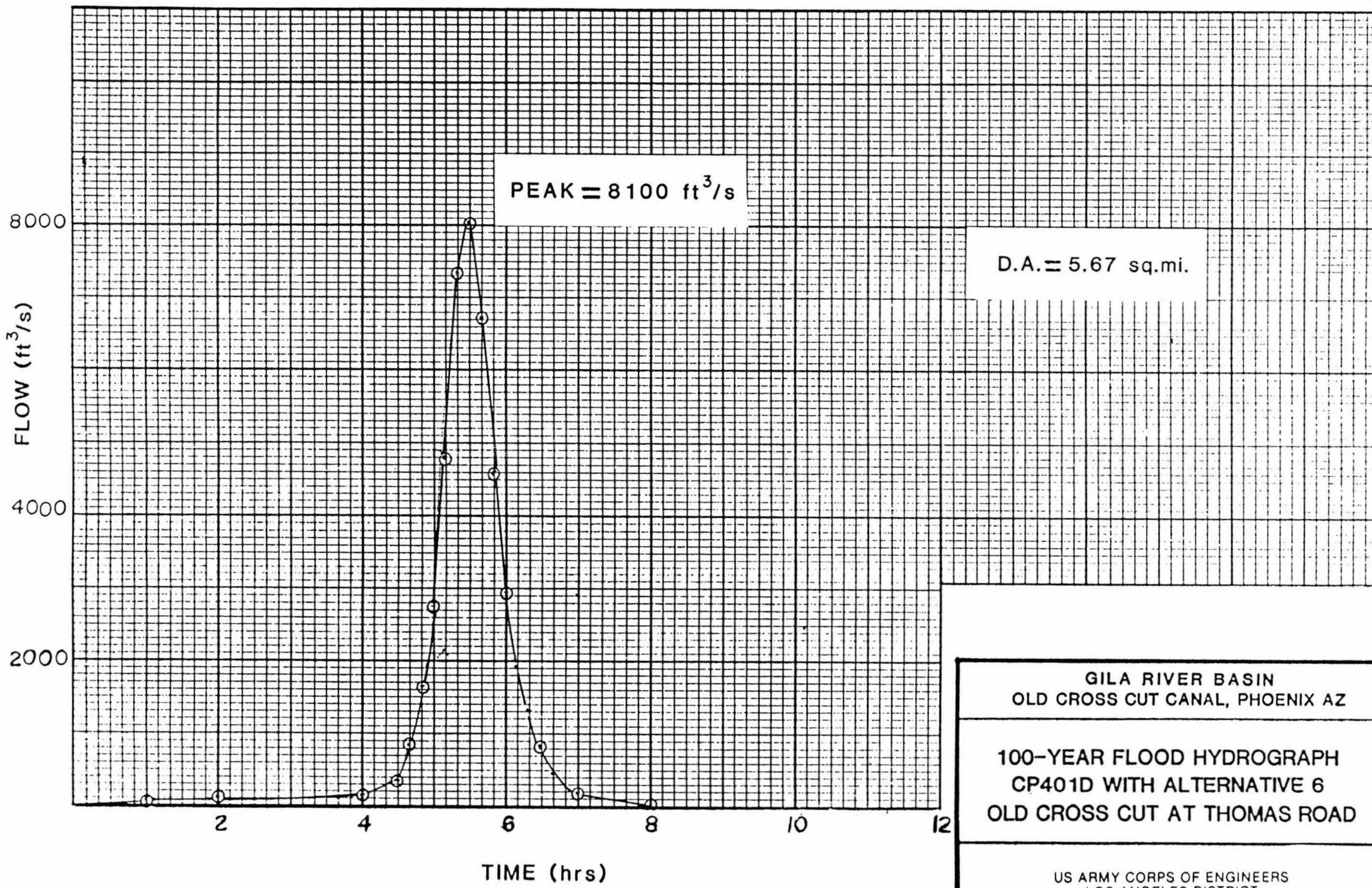
U. S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

100-YEAR FLOOD HYDROGRAPH
PRESENT EXISTING CONDITIONS CP 401
THOMAS ROAD AT OLD CROSS CUT

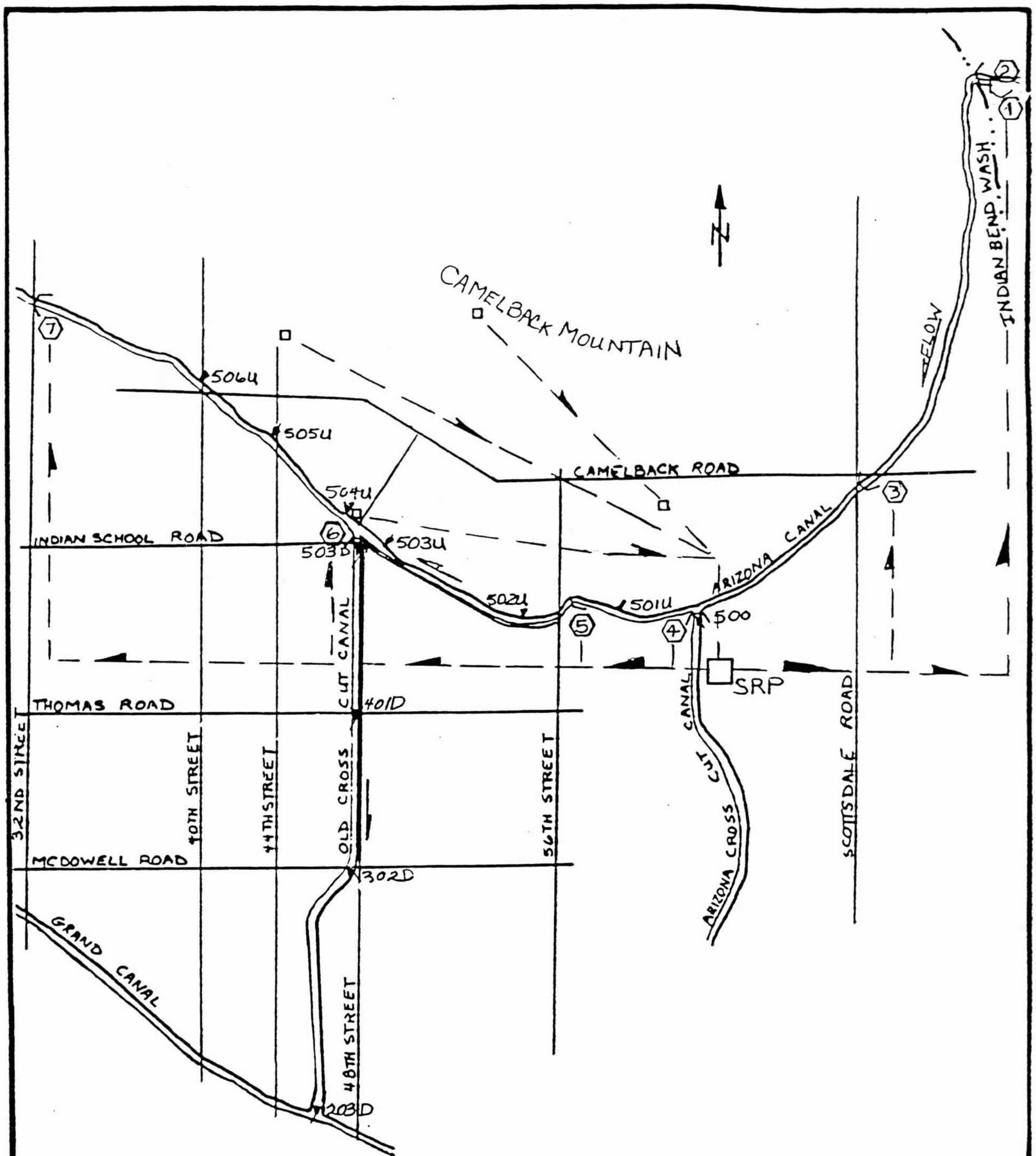
US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

100-YEAR FLOOD HYDROGRAPH
CP401D WITH ALTERNATIVE 6
OLD CROSS CUT AT THOMAS ROAD

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



LEGEND

506U	CONCENTRATION POINT
5	CANAL GATE
	PROPOSED RAIN GAGE
	SRP HEADQUARTERS
	RADIO LINKS

NOT TO SCALE

GILA RIVER BASIN
OLD CROSS CUT CANAL, PHOENIX AZ

SCHEMATIC OF
PROPOSED FLOOD
FORECASTING SYSTEM

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT