Standard Project Flood Agua Fria River

GILA RIVER BASIN, ARIZONA

STANDARD PROJECT FLOOD

AGUA FRIA RIVER

BETWEEN THE NEW RIVER CONFLUENCE AND THE GILA RIVER WITH NEW WADDELL DAM IN PLACE

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

JANUARY, 1989

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STANDARD PROJECT FLOOD

AGUA FRIA RIVER BETWEEN THE NEW RIVER CONFLUENCE

AND THE GILA RIVER WITH NEW WADDELL DAM IN PLACE

1. PURPOSE

This report is intended to serve as documentation for the determination of the Standard Project Flood (SPF) on the Agua Fria River between the New River confluence and the Gila River, for future conditions with New Waddell Dam and Phoenix Projects in place. New Waddell Dam, which will replace the existing Waddell Dam, has a much larger storage capacity than the old dam reducing the impact of storms occurring upstream of the dam. SPF values were based on local runoff due to thunderstorms rather than dam outflow based on a general storm, since local runoff resulted in larger downstream flows. Levees have been constructed on the lower Agua Fria River (below New River) designed for SPF flows previously determined for the condition with existing Waddell Dam (ref. 1). The levees extend from approximately Indian School Road to below Lower Buckeye Road on the west side of the Agua Fria River and from approximately Indian School Road to Buckeye Road on the east side of the river (see pl. 1). The Phoenix Project required the SPF with both the existing levees and New Waddell Dam in place.

2. SCOPE

For the determination of the SPF the following items were completed:

a. Various storms were evaluated for the Agua Fria River drainage area to determine which one(s) was most appropriate for SPF generation.

b. A rainfall-runoff model was developed for the Agua Fria River basin. (See pl. 2 for map of contributing drainage areas.) The selected Standard Project Storm (SPS) was centered over different portions of the watershed and resultant runoff evaluated.

c. The discharges resulting from the critical SPS centering were selected as the SPF in the Agua Fria River.

3. RESULTS

The SPF peak discharges are presented in table 1 for the following Agua Fria River concentration points (CP's):

a. Below the Agua Fria River and New River confluence (CP1039D). 102,000 cfs

b. At I-10 Freeway (CP1040). 99,000 cfs

c. At Avondale (CP1042). 97,000 cfs

d. Above the Gila River (CP1043). 92,000 cfs

See plates 2 and 3 for the location of the concentration points.

4. METHODOLOGY

4.1 General.

The 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. Two large storm events used in previous studies for this basin were considered as possible standard project storms (ref. 1): (1) the general summer storm of 26-29 August 1951 which occurred in southern Yavapai and northeastern Maricopa counties in Arizona and (2) the local summer thunderstorm of 19 August 1954 which occurred over the Queen Creek drainage east of Phoenix. These events were determined to be the storms with the most severe flood-peak producing rainfall that could be reasonably expected to occur over the Agua Fria River basin.

4.2 Standard Project Storm Selection.

The condition of a general summer storm (26-29 August 1951 storm) occurring upstream of New Waddell Dam was analyzed since this was the SPS for Waddell Dam prior to the construction of New Waddell Dam. The hydrograph resulting from this storm was routed through New Waddell, and the maximum outflow to the Agua Fria River below the dam was 53,000 cfs. The reservoir operation was based on a full water conservation pool at the start of the August 1951 storm, i.e. a starting storage in the reservoir of 856,400 ac-ft corresponding to a water surface elevation of 1,702 feet. This assumption gave the largest outflow possible but was used only for comparison purposes.

Based on the worst case outflow from the dam of 53,000 cfs routed downstream and combined with local contributing runoff, the resulting discharge above the New River confluence was less than the discharge resulting from a SPS thunderstorm type event on the New River above the Agua Fria confluence (ref. 1). In addition, the general summer storm does not produce significant local runoff or tributary flow downstream from New Waddell Dam. The local summer thunderstorm (Queen Creek) centered over the New River tributary (subareas 1 through 29, see pl. 2 for locations) resulted in a discharge above the confluence with the Agua Fria of approximately 69,000 cfs, including contributions from Dreamy Draw, Cave Buttes, Adobe, and New River Dams from residual precipitation, as well as coincident runoff from subarea 38. This SPS centering also resulted in residual precipitation over the Agua Fria River (below New Waddell Dam) which was comparable to 50- or 100-year rainfall. Therefore, the SPF for the Agua Fria River below the confluence of the New River with New Waddell Dam in place would result from the local summer thunderstorm rather than a general storm such as August 1951.

4.3 Standard Project Storm Precipitation.

Total precipitation for the SPS was originally obtained from the isohyets of the 19 August 1954 Queen Creek local thunderstorm (shown on pl. 4), transposed and critically centered over the subareas within the Agua Fria River basin. The heaviest precipitation of this storm (7.5 inches maximum) occurred in mountain and foothill areas where orographic influences were significant; therefore, the total storm depth was adjusted as it was transposed to the study area by means of 10-year, 6-hour precipitation values (pl. 5) published by the National Weather Service in NOAA Atlas 2 (ref. 4), which reflect orographic influences. For the New River and Agua Fria River drainage basins the 10-year, 6-hour precipitation value (pl. 5) was 2.0 inches. For the Queen Creek drainage basin the 10-year, 6-hour precipitation value was 2.36 inches. Therefore, to determine the maximum precipitation for the study area the following ratio was used:

$$\frac{7.5\text{ in.}}{2.36\text{ in.}} = \frac{\text{maximum precip.}}{2.0\text{ in.}}\text{, therefore, maximum precip.} = 6.36\text{ in.}$$

The average total-storm precipitation over each subarea was determined by reducing the transposed maximum point precipitation (6.36 in.) by means of depth-area reduction curves (pl. 6). These were constructed from the original depth-area curve developed from isohyets of the original 1954 Queen Creek storm and adjusted for orographic influences. The depth-area reduction curves are labeled according to the 10-year, 6-hour precipitation values. The precipitation data is shown in table 2.

The residual rainfall was also calculated for those sub-basins in the study area outside the selected storm centering. The residual areal precipitation was calculated using the following relationship based on mass balance:

$$T_t A_t = T_1 A_1 + T_2 A_2 \qquad \text{Eqn. 1}$$

where:

T = Areal precipitation (inches)
A = Drainage area of rainfall (square miles)

Subscripts:

t = Combined area values
1 = Values from storm center
2 = Residual values from area outside of storm center

As an example, for the case of the storm centered over New River subareas 1 through 29 with a drainage area of 125 sq. mi. (A_1) and a precipitation depth of 4.93 inches (T_1), the residual runoff for subarea 38 (drainage area of 36.1 sq. mi.) was computed. The total rainfall area is therefore 161 sq. mi., and applying the new depth-area reduction coefficient to the point precipitation of 6.36 inches, an areal precipitation of 4.77 inches is obtained. The residual areal precipitation for subarea 38 of 4.22 inches was determined from the equation above as follows:

$$4.77 \text{ in. } (161 \text{ sq. mi.}) = 4.93 \text{ in. } (125 \text{ sq. mi.}) + T_{38} (36 \text{ sq. mi.}), \text{ therefore}$$

$$T_{38} = 4.22 \text{ in.}$$

4.4 Standard Project Storm Pattern.

A number representing the storm pattern, which defines the time-distribution of precipitation for the local standard project storm, was determined for each subarea in the study using plate 7. Plate 7 uses both drainage area and 10-year, 6-hour precipitation to define the pattern number - the greater the pattern number the lower the intensity of rainfall (see pl. 8). The relationships presented in plates 7 and 8 were developed for a previous study (ref. 1). The storm pattern number curves from plate 7 were used directly for the areas over which the storm was centered. For the residual areas a weighted pattern number was determined: the percent of point rainfall was computed from a ratio of the residual to total areal point precipitation. Using the example from paragraph 4.3, the residual rainfall for subarea 38 is 4.22 in. Therefore, the percent of point rainfall = $\frac{4.22 \text{ in.}}{6.36 \text{ in.}}$ = 66 percent. An "equivalent drainage area" of 320 sq. mi. was obtained from plate 6 using the 10-year, 6-hour precipitation curve of 2.0 inches and 66 percent of point rainfall. The 320 sq. mi. "equivalent drainage area" along with the 10-year, 6-hour precipitation, produces a pattern number of 4.65 from plate 7 for the residual portion of the drainage area.

4.5 Rainfall Runoff Model.

4.5.1 General. A data model for the Los Angeles District Flood Hydrograph Package (LADFHP) computer program was developed for the New River and Agua Fria River drainages. Storm centerings were evaluated using the transposed Queen Creek local summer thunderstorm of 19 August 1954. The model was based on future conditions with New Waddell Dam, New River Dam, and Adobe Dam in place, and with a 100-year design for the Arizona Canal Diversion Channel (ACDC).

The impervious cover percentages were based on future development as well. Subarea characteristics and routing data are listed in tables 3 and 4. The storm centering which produced the largest peak flow in the Agua Fria below New River is discussed in paragraphs 4.5.2.

4.5.2 Storm Centered over the New River Basin. The Queen Creek storm was critically centered over subareas 1 through 29 of the New River drainage basin (125 sq. mi.), which covered the drainage areas for New River, Skunk Creek, Cave Creek and ACDC. In a previous study a peak SPF value of 69,000 cfs had been determined at the New River above the confluence with the Agua Fria River (ref. 1, table 1). The Agua Fria SPF model in this report incorporated the same subarea data and storm centering used in the previous study in order to match this discharge. The model was expanded to include the residual runoff from the local contributing drainage areas for New River subarea 38 and Agua Fria River subareas 36, 37, 39, 40, 41, and 42 (approximately 419 additional sq. mi.) as well as expected outflow from Dreamy Draw, Cave Buttes, Adobe, and New River Dams. Flow in excess of the ACDC channel capacity (maximum of 29,000 cfs) was determined to flow overland to the Salt or Agua Fria Rivers and would not significantly affect the peak flow in the Agua Fria River during the SPF.

The drainage areas located adjacent to the lower Agua Fria River (subareas 40, 41, & 42) were analyzed in more detail than in previous reports because of the impact of large upstream impoundment (New Waddell Dam) on downstream flow. Recently constructed levees extend from approximately Indian School Road to below Lower Buckeye Road on the west side of the Agua Fria River and from approximately Indian School Road to Buckeye Road on the east side of the river. See plate 1. The Interstate 10 (I-10) collector channel which runs east to west along the freeway perpendicular to the Agua Fria River and borders the south end of subarea 40, was designed for a maximum peak flow of about 9300 ft^3/s. The freeway is depressed below the surrounding ground level along the border of subarea 40 from approximately 93rd Avenue to Black Canyon Fwy. and would act as a catch basin if flows were to exceed the capacity of the collector channel. The freeway is raised above ground level from approximately 93rd Avenue to the Agua Fria River. Flow exceeding the collector channel in this area could travel south through the street underpasses and join with the Agua Fria River flow downstream below the end of the levee.

The maximum runoff from subarea 40, based on the SPS storm centering discussed in paragraph 4.5.2 above, was 13,500 cfs. Other storm centerings were analyzed, but the critical centering described therein resulted in the maximum flows in the Agua Fria River. All runoff from subarea 40 was assumed to reach the Agua Fria River via the I-10 collector channel, or by moving along the backside of the levees and entering through an opening in the levee system just upstream of the confluence of the I-10 collector channel with the Agua Fria River. Runoff from subarea 42, which is separated from subarea 40 by the I-10 collector channel, was not affected by the construction of the levees and would enter the river at approximately Buckeye Road. Not matching Simons/Li and JJA study

Runoff from subarea 41 was judged noncontributing to flow in the Agua Fria River based on an on-site field inspection as well as the existing topography. Flow from subarea 41 would continue south and enter the Gila River directly (ref. 5). In a sensitivity analysis, the runoff from subarea 41 was assumed to enter the Agua Fria River. This increased the discharges at concentration points 1042 and 1043 by approximately 3000 cfs. Therefore, the impact of runoff from subarea 41 on the flows in the Agua Fria River was small. For the purpose of this study, runoff from subarea 41 was excluded.

4.5.3 Routing and Combining Flow.

Channel routing was simulated using the Muskingum method. The parameters are shown in table 4.

4.5.4 Standard Project Flood Results. The discharges resulting from the storm centering described in paragraph 4.5.2 above were selected for each of the four concentration points as the final SPF value. The values are given in table 1, and the hydrographs at each of the concentration points are shown in figures 1 through 4. based on the routing of flows
four centers for four [ ] pts? no only one center

REFERENCES

1. Gila River Basin, Phoenix, Arizona and Vicinity (including New River) Hydrology Part 2, Design Memorandum No. 2, dated 1982.

2. Gila River Basin, New River and Phoenix City Streams, Arizona, Design Memorandum No. 2, Hydrology Part 1, dated October 1974.

3. Hydrology for Evaluation of New Waddell Dam, Agua Fria River Below New Waddell Dam to the New River Confluence, dated September 1988.

4. NOAA Atlas, 2, Precipitation - Frequency Atlas of the Western United States, Volume VIII - Arizona, 1973.

5. McMicken Dam Break Study, Breach at Trilby Wash, Case 2, dated April 1987, Flood Control District of Maricopa County.

TABLE 1: Standard Project Flood Discharges
Future Conditions with Project
Agua Fria River: New River Confluence to Gila River

Concentration Point	Drainage Area(1) (sq.mi)	SPF Peak Discharges(2) (cfs)
1039D	392	102,000
1040	474	99,000
1042	485	97,000
1043	485	92,000

D = Downstream of the confluence

(1) Equivalent to the uncontrolled contributing drainage area consisting of the following subareas:
CP1039D - Subareas 36,37,39,38,1-29
CP1040 - Subareas 36,37,39,40,38,1-29
CP1042 - Subareas 36,37,39,40,42,38,1-29
CP1043 - Same as CP1042

(2) Queen Creek storm centered over New River subareas 1-29. Residual rainfall over: New River drainage subarea 38 and Agua Fria River drainage subareas 36,37,39,40,41 and 42 runoff from subarea 41 will not contribute to flow in the Agua Fria River. Runoff from subareas 30,31,32,33,34, and 35 was included in the peak discharges above by estimating outflow from the controlling reservoirs during the SPF.

TABLE 2: Standard Project Storm Subarea Rainfall

Subarea	Point Precip. 6-hr (in)	Incre. Drainage Area of Rainfall (sq.mi)	Total Accum. Drainage Area (sq.mi)	Depth-Area Reduc. Coef. for Total Area	Average Precip. for Total Area (in)	Incre. Areal Precip. (in)	Incre. Pattern No.
		1. Storm centered over New River subareas 1-29					
1-29	6.36	125	125	0.775	4.93	4.93	4.10
38	6.36	36.1	161.1	0.75	4.77	4.22	4.65
36,37, 39	6.36	231	392	0.62	3.94	3.36	5.00
40,41*, 42	6.36	153	544	0.535	3.40	2.00	5.40

*Runoff from subarea 41 will not contribute to the Agua Fria River flow

TABLE 3

SUBAREA CHARACTERISTICS(1)

SUBAREA NO.(2)	DRAINAGE AREA (mi)	L (mi)	L (mi)	SLOPE (ft/mi)	BASIN n-VALUE	IMPERVIOUS COVER(3) (%)	S-GRAPH
				ACDC ABOVE CAVE CREEK			
1	4.91	3.95	1.87	222	0.025	40	Phoenix Valley
2	1.38	1.94	0.97	227	0.028	40	Phoenix Valley
3	1.37	1.19	0.45	298	0.025	45	Phoenix Valley
4	1.10	2.20	1.14	390	0.030	35	Phoenix Valley
5	1.18	2.24	0.91	390	0.030	35	Phoenix Valley
7	1.82	2.35	1.06	110	0.030	40	Phoenix Valley
8	2.74	3.28	1.49	80	0.025	50	Phoenix Valley
9	5.22	2.24	0.75	163	0.025	50	Phoenix Valley
				CAVE CREEK BELOW BUTTES DAM			
10	4.51	4.00	1.60	48	0.029	5	Phoenix Valley
11	0.49	1.44	0.80	69	0.030	5	Phoenix Valley
12	12.98	7.40	3.60	34	0.022	45	Phoenix Valley
13	3.09	5.00	2.64	34	0.030	40	Phoenix Valley
14	6.78	4.20	2.40	74	0.028	40	Phoenix Valley
15	1.40	2.20	1.40	29	0.030	40	Phoenix Valley
16	1.10	2.60	1.60	29	0.025	40	Phoenix Valley
				ACDC BELOW CAVE CREEK			
17	11.04	-	-	-	-	45	-
18	9.21	-	-	-	-	45	-
19	15.03	-	-	-	-	45	-
				SKUNK CREEK BELOW ABODE DAM			
20	6.30	3.60	1.70	33	0.020	40	Phoenix Valley
21	3.10	3.40	1.80	29	0.020	40	Phoenix Valley
22	3.47	5.00	2.80	86	0.020	40	Phoenix Valley
22,27	6.80	5.00	2.80	59	0.020	40	Phoenix Valley
23	2.34	2.80	1.30	57	0.025	35	Phoenix Valley
24	1.40	3.50	1.80	89	0.035	10	Phoenix Mountain

TABLE 3 (Continued)

SUBAREA CHARACTERISTICS(1)

SUBAREA NO.(2)	DRAINAGE AREA (mi)	L	L (mi)	SLOPE (ft/mi)	BASIN n-VALUE	IMPERVIOUS COVER(3) (%)	S-GRAPH
			NEW RIVER BELOW NEW BELOW RIVER DAM				
25	10.30	5.90	3.40	44	0.023	45	Phoenix Valley
26,27,29	11.68	6.00	3.60	40	0.020	45	Phoenix Valley
38	36.10	14.20	7.10	25	0.020	40	Phoenix Valley
			AGUA FRIA RIVER				
36	69.0	17.0	8.0	94	0.035	10	Indian Bend Wash
37	102.3	26.0	10.3	70	0.035	15	Indian Bend Wash
39	59.3	13.8	7.9	22	0.025	30	Indian Bend Wash
40	81.6	18.5	10.5	15	0.020	40	Indian Bend Wash
41(4)	59.6	16.6	8.7	18	0.025	30	Indian Bend Wash
42	11.3	6.9	3.2	12	0.025	35	Indian Bend Wash

Note: (1) Source of data. Ref. 1, Table 2.
(2) See plates 2 and 3 for subarea location.
(3) Future conditions impervious cover percentage (Ref. 1, Table 2)
(4) Noncontributing to Agua Fria River flow.

TABLE 4

PERTINENT ROUTING DATA (1)

CHANNEL REACH(2)	REACH LENGTH (ft)	AVERAGE VELOCITY (ft/sec)	MUSKINGUM COEFFCIENTS k (hrs)	x	NRCHS
		ACDC (TO CAVE CREEK)			
101 R 102(3)	4500	11	0.083	0.40	1
102 R 103	5000	11	0.083	0.40	2
103 R 104	6800	11	0.083	0.40	2
104 R 105	3700	11	0.083	0.40	1
105 R 107D	3600	11	0.083	0.40	1
107D R 108	4100	10	0.083	0.40	1
108 R 109	10,100	10	0.083	0.35	3
109 R 1016	5800	10	0.083	0.35	2
		CAVE CREEK			
1011 R 1013	25,600	9	0.25	0.25	3
1013 R 1015	11,400	8	0.25	0.30	2
1015 R 1016	12,000	6	0.25	0.30	2
		ACDC (TO SKUNK CREEK)			
1016 R 1018	14,700	12	0.25	0.30	1
1017 R 1018	24,000	3	0.25	0	9
1018 R 1019	17,500	9	0.25	0.25	1
1019 R 1022	11,300	9	0.25	0.25	1
		SKUNK CREEK			
1020 R 1021	16,400	10	0.25	0.30	2
1021 R 1024	8000	10	0.25	0.30	1
1023 R 1024	19,500	6	0.25	0.30	4

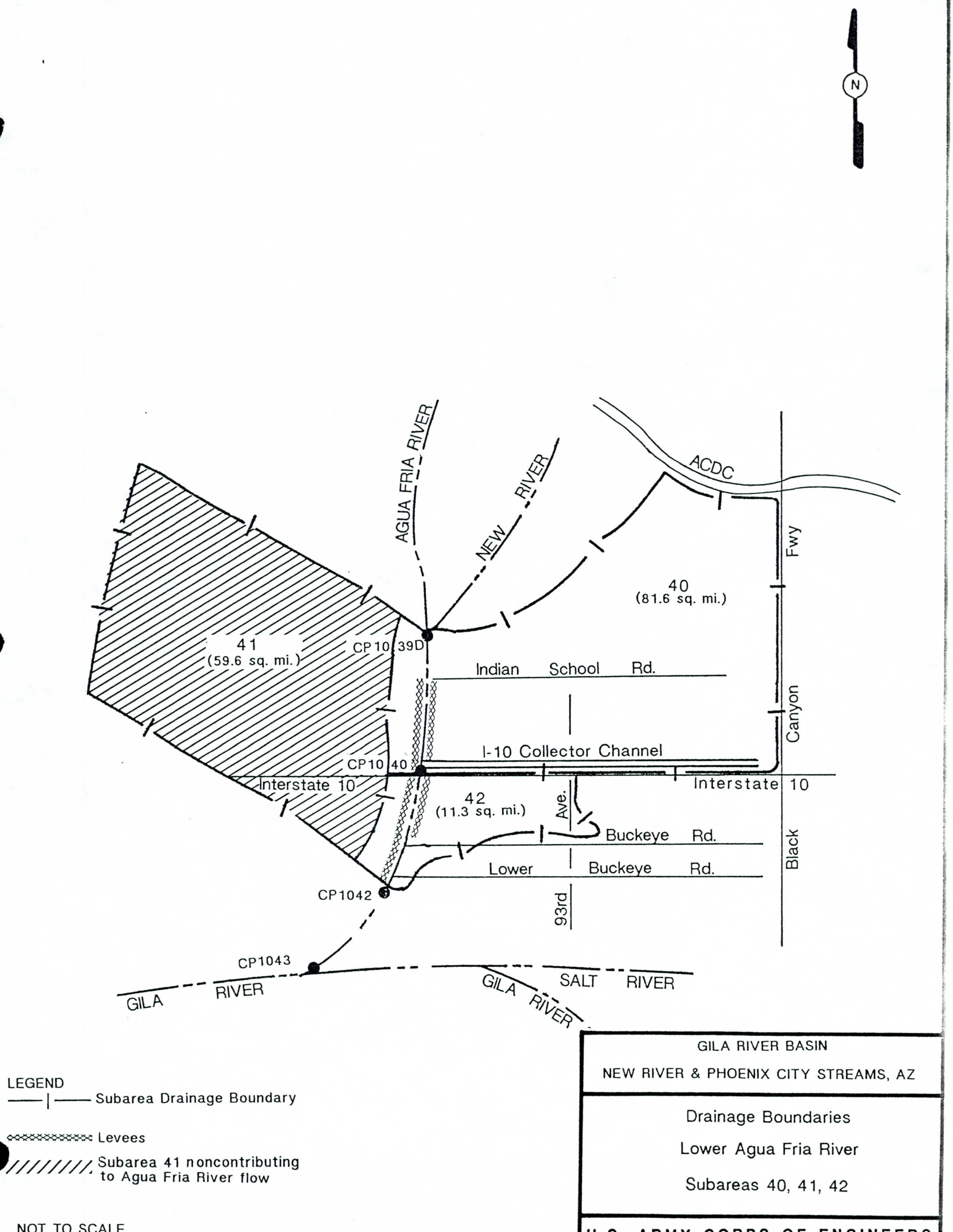
TABLE 4 (con't)

PERTINENT ROUTING DATA(1)

CHANNEL REACH(2)	REACH LENGTH (ft)	AVERAGE VELOCITY (ft/sec)	MUSKINGUM COEFFICIENT K (hrs)	X	NRCHS
SKUNK CREEK (con't)					
1024 R 1029(3)	18,900	10	0.25	0.30	2
1022 R 1029	9500	10	0.25	0.30	1
NEW RIVER					
1025 R 1029	21,600	10	0.25	0.30	2
1029 R 1039	43,800	7	0.25	0.20	7
MCMICKEN DAM AND OUTLET CHANNEL					
1036 R 1037	29,200	8	1.0	0.20	1
AGUA FRIA RIVER					
1037 R 1038	12,100	7&9	0.5	0.20	1
1038 R 1039	35,400	5	1.8	0.20	1
1039 R 1040	25,000	6	1.2	0.20	1
1040 R 1042	9000	5	0.6	0.20	1
1042 R 1043	21,200	6	1.1	0.20	1

X value too low for channelized area, velocities much higher

Footnote:
(1) Source of data, Ref. 1, Table 12.
(2) Refer to plates 2 and 3 for identification of reaches.
(3) This symbolizes the reach from subarea or concentration point "A" routed to subarea or concentration point "B" ("A" R "B").



GILA RIVER BASIN

NEW RIVER & PHOENIX CITY STREAMS, AZ

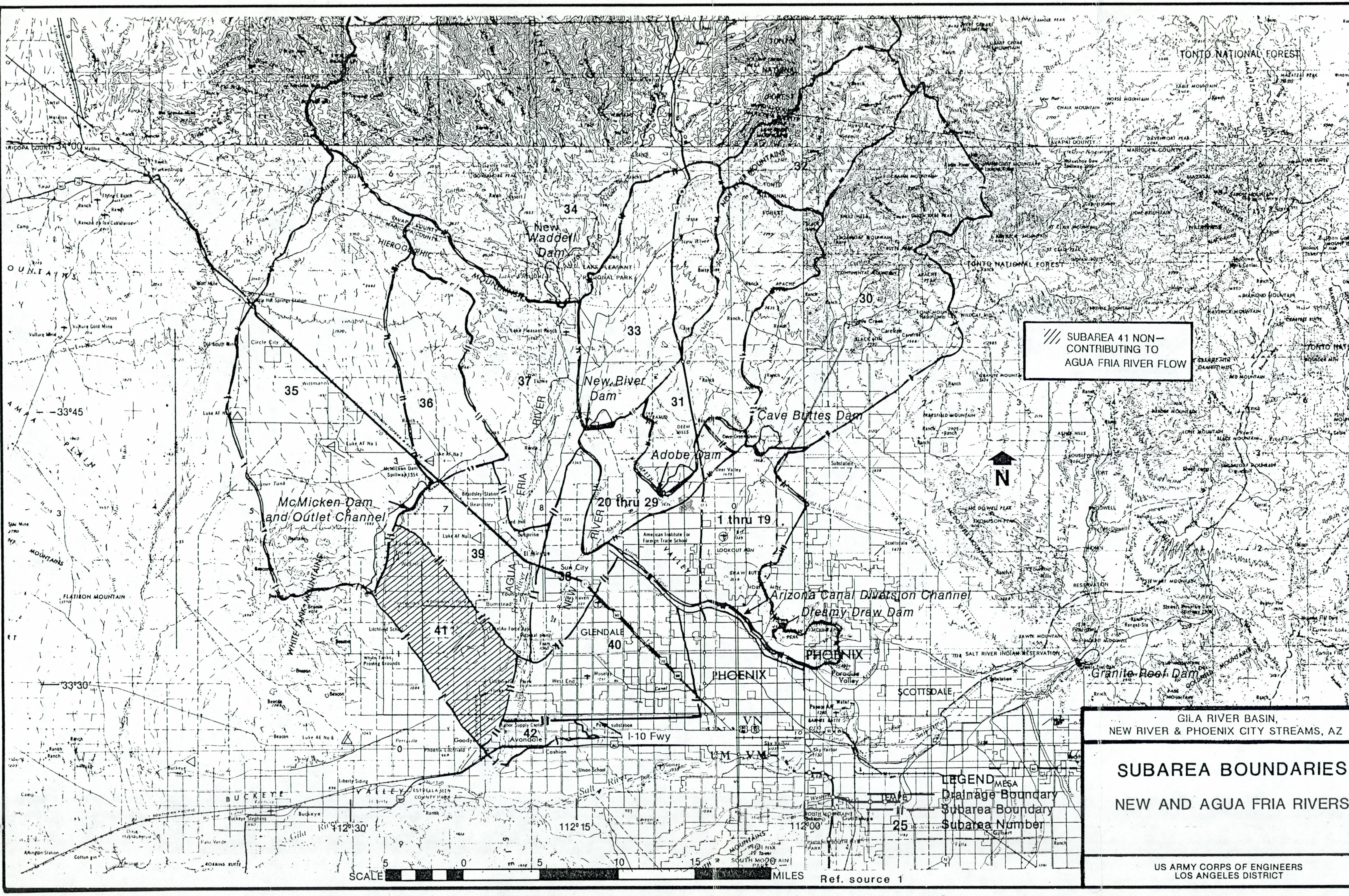
Drainage Boundaries

Lower Agua Fria River

Subareas 40, 41, 42

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 1



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

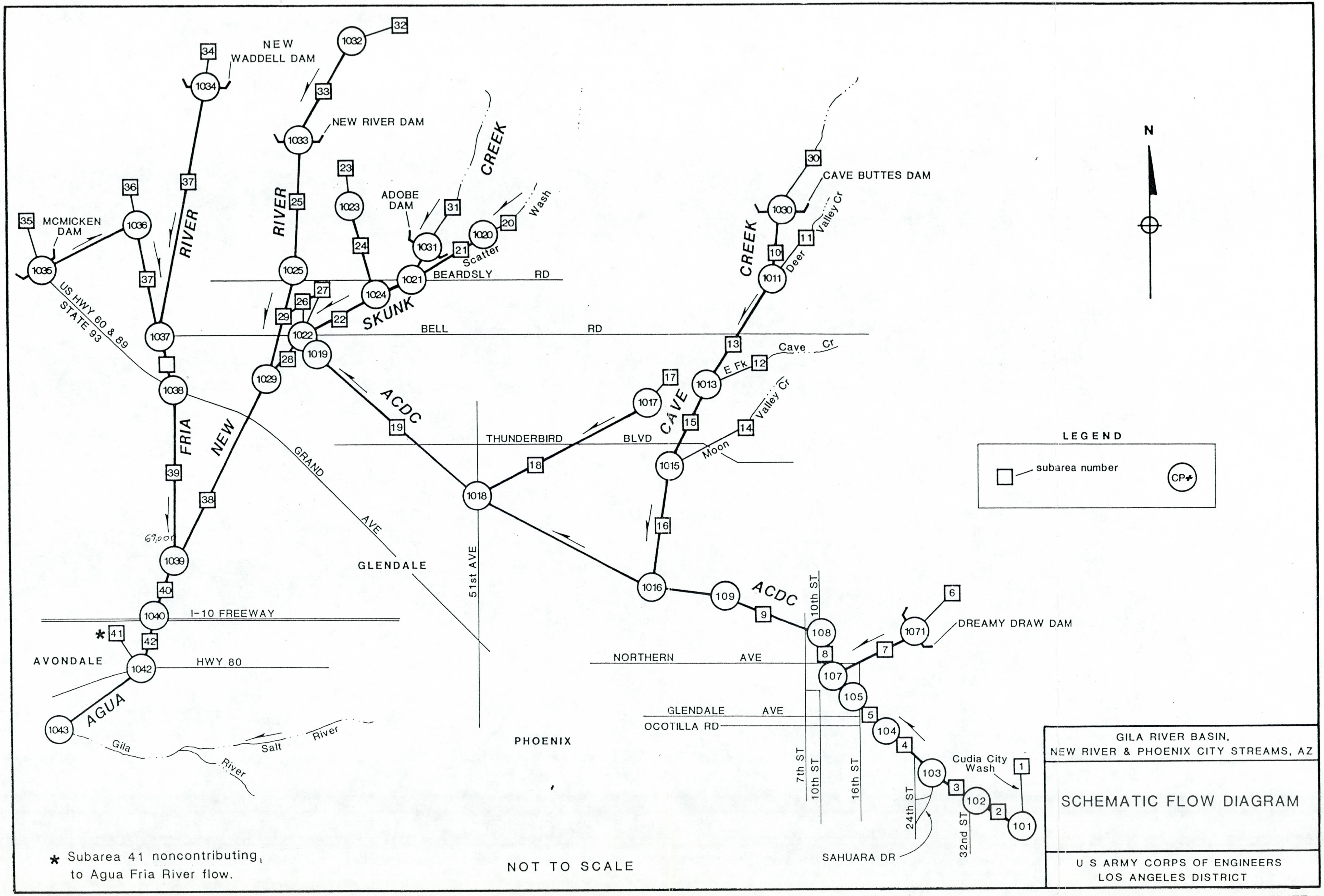
SUBAREA BOUNDARIES

NEW AND AGUA FRIA RIVERS

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 2

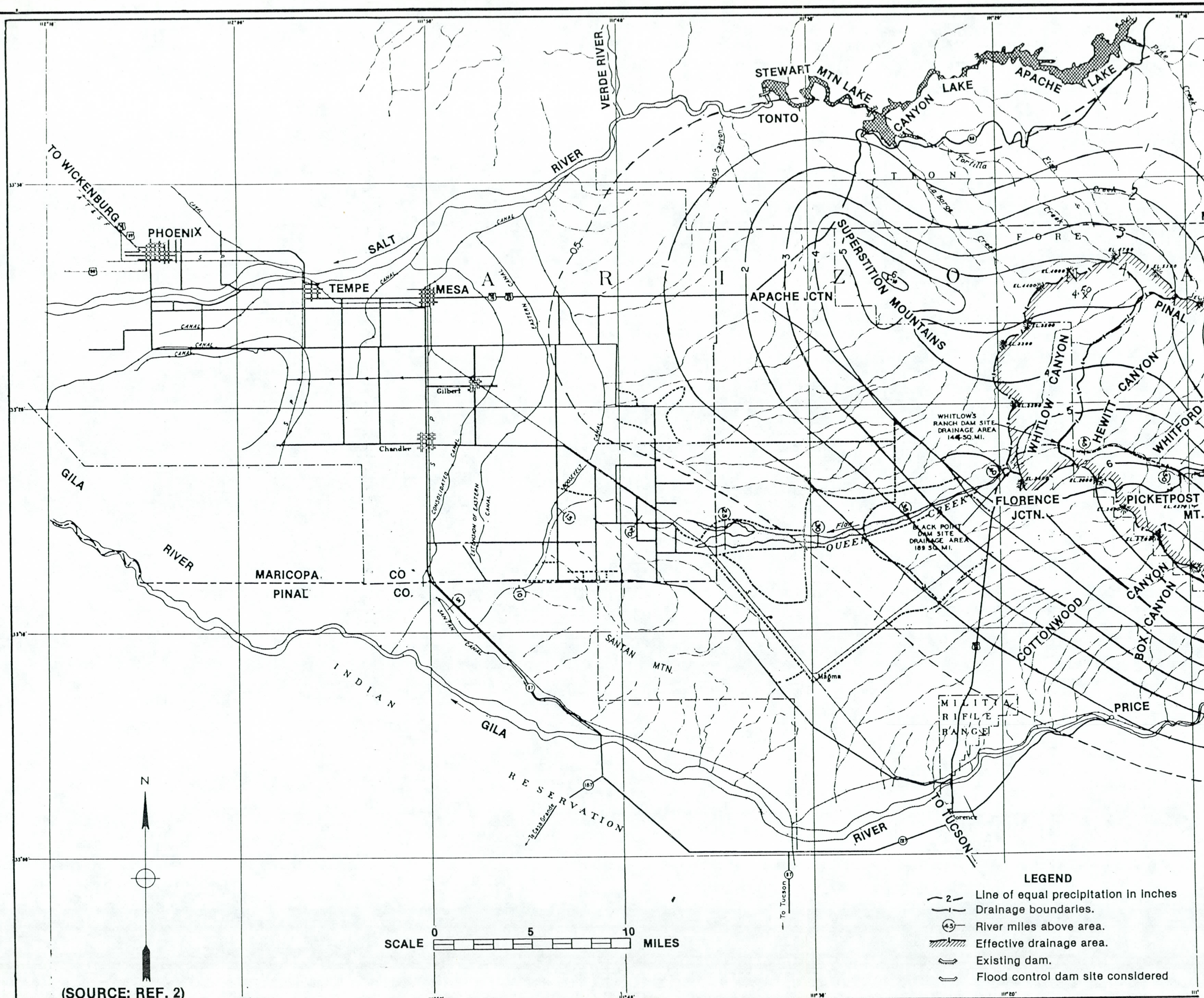
Revised May '85



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

SCHEMATIC FLOW DIAGRAM

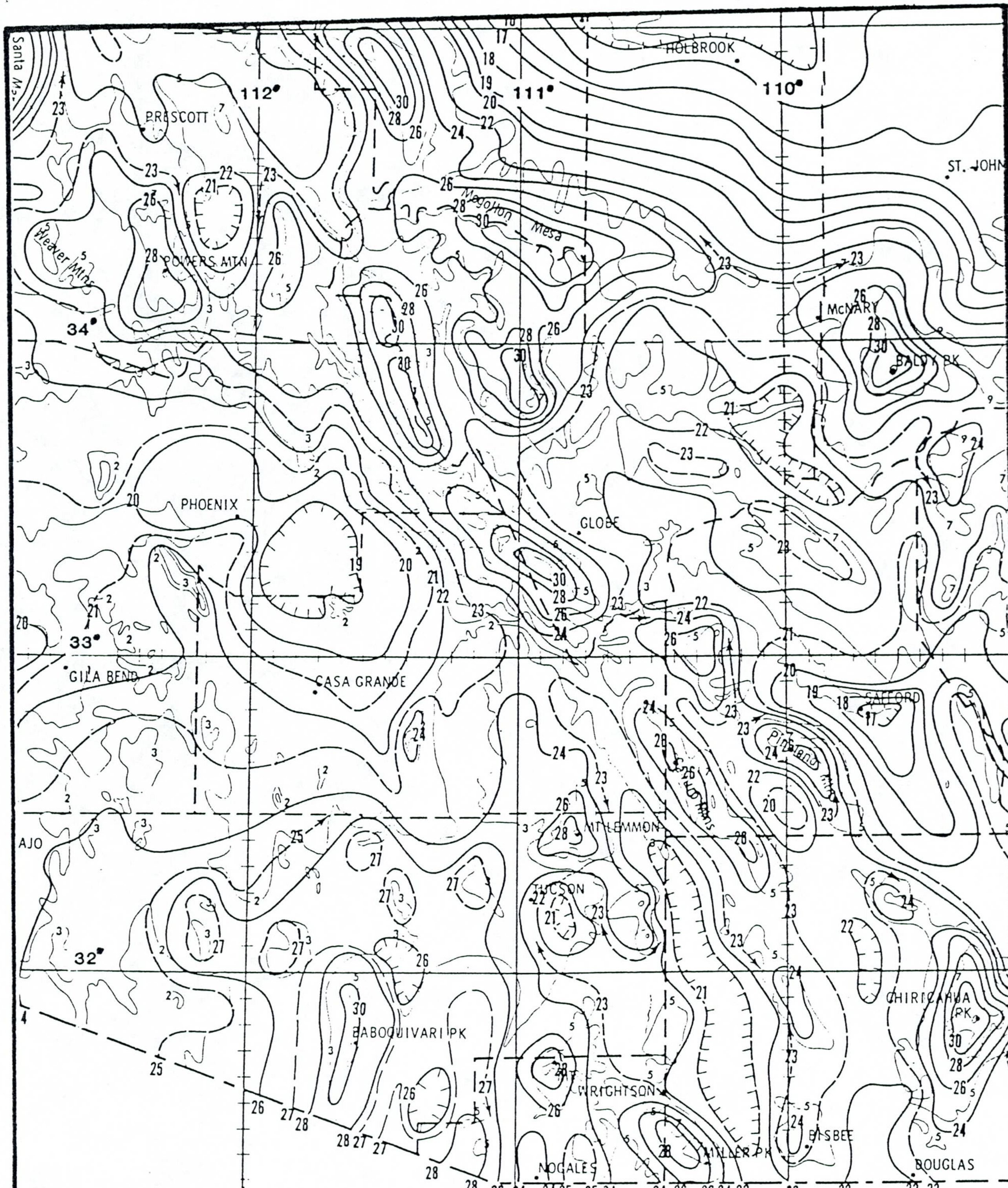
U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAS, AZ

ISOHYETAL MAP
STORM OF AUGUST 19, 1954

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



LEGEND

—28— ISOPLUVIALS IN TENTHS OF AN INCH

— 2 — ELEVATION IN THOUSANDS OF FEET

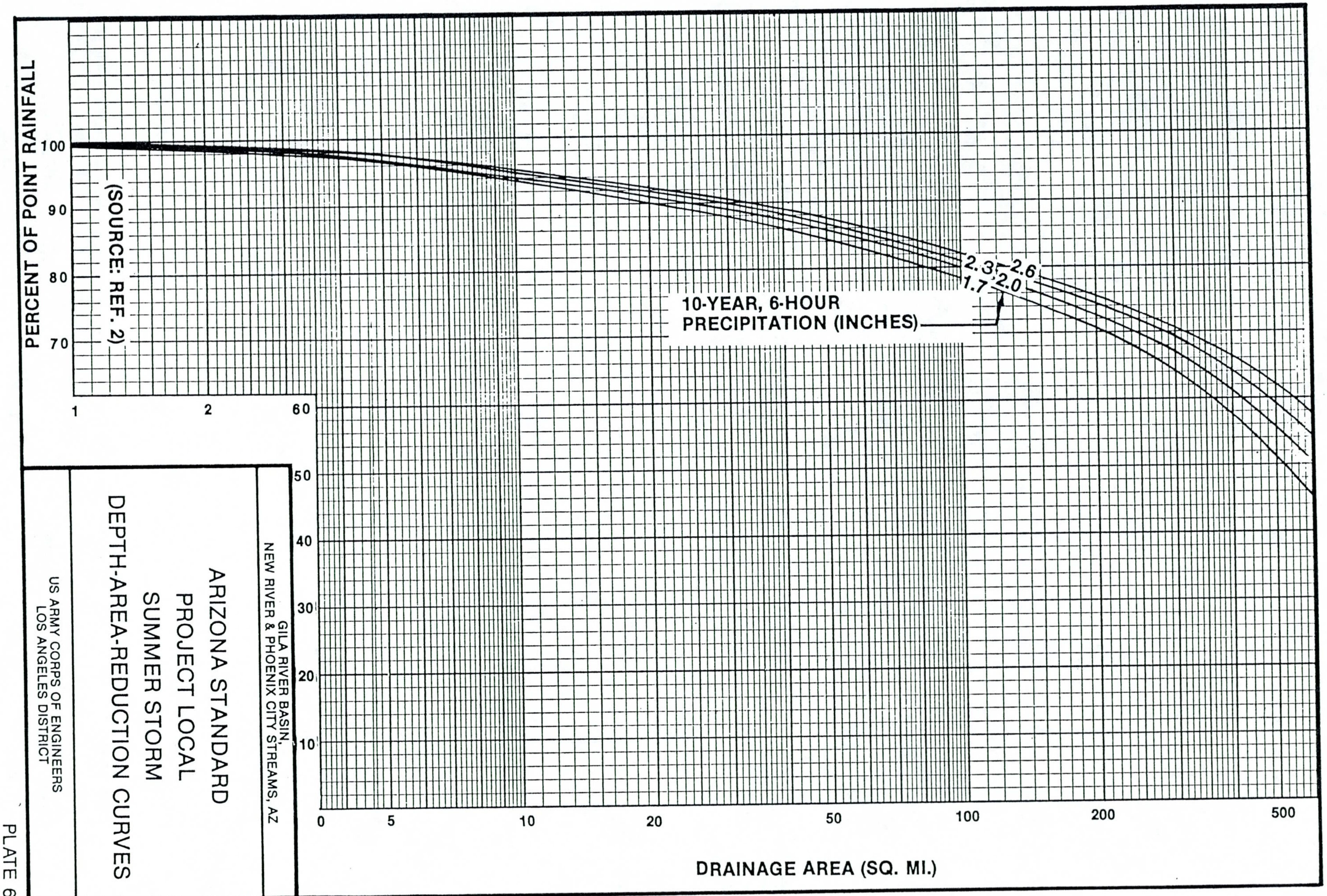
SCALE IN MILES

0 10 20 30

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

10 YEAR 6 HOUR PRECIPITATION MAP

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



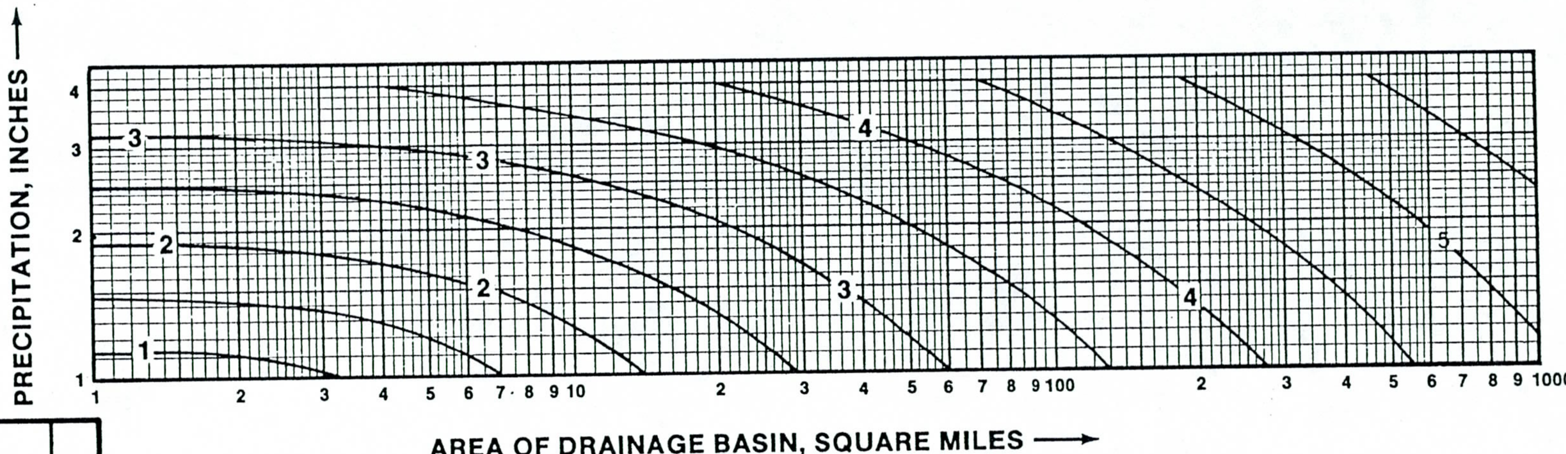
GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, AZ

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

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 6

(SOURCE: REF. 2)



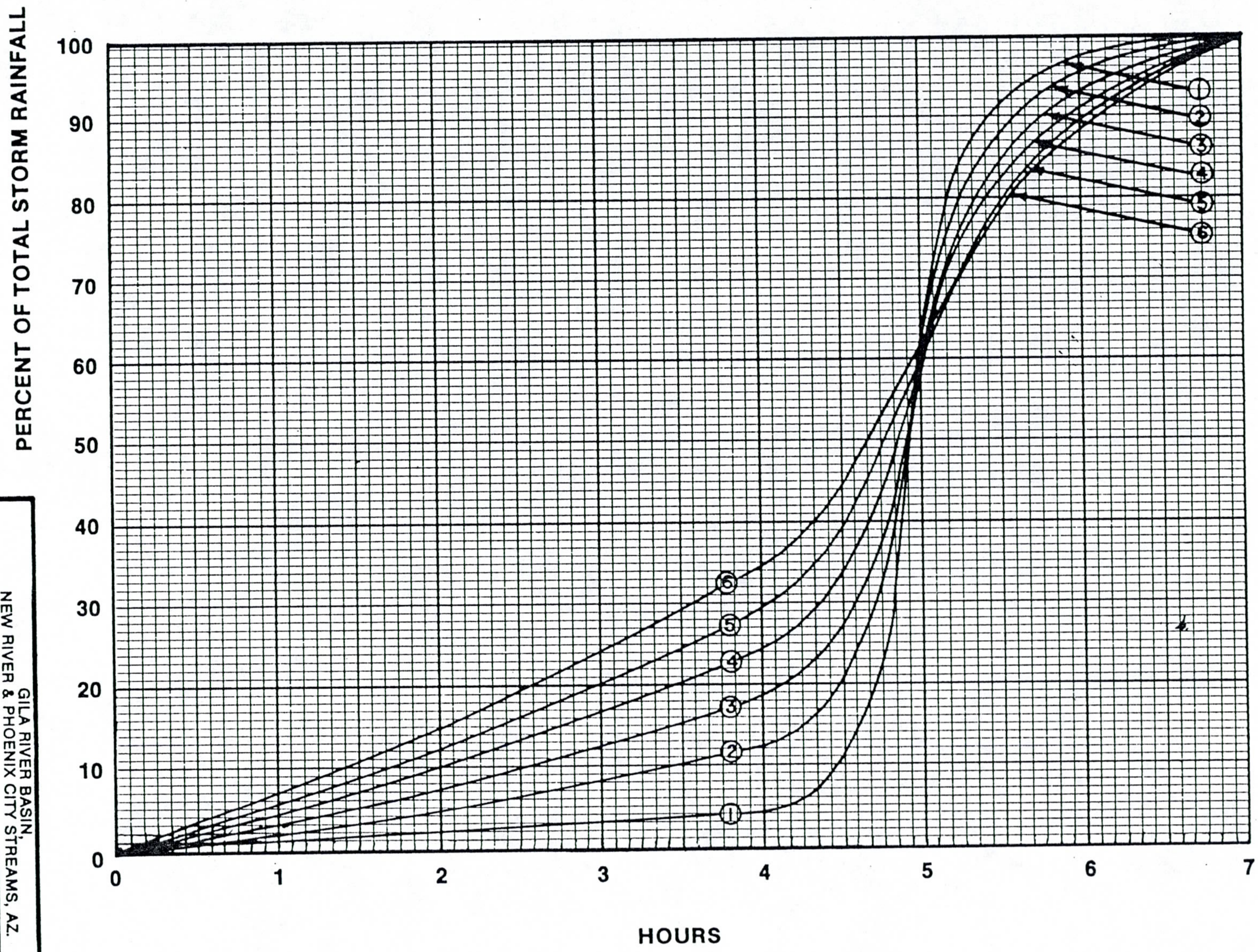
—2— PATTERN NUMBER
REFER TO PLATE 8
FOR ACTUAL PATTERN

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, AZ.

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

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 7



—2— PATTERN NUMBER
MAKE PATTERN NUMBER
SELECTION ON PLATE 7

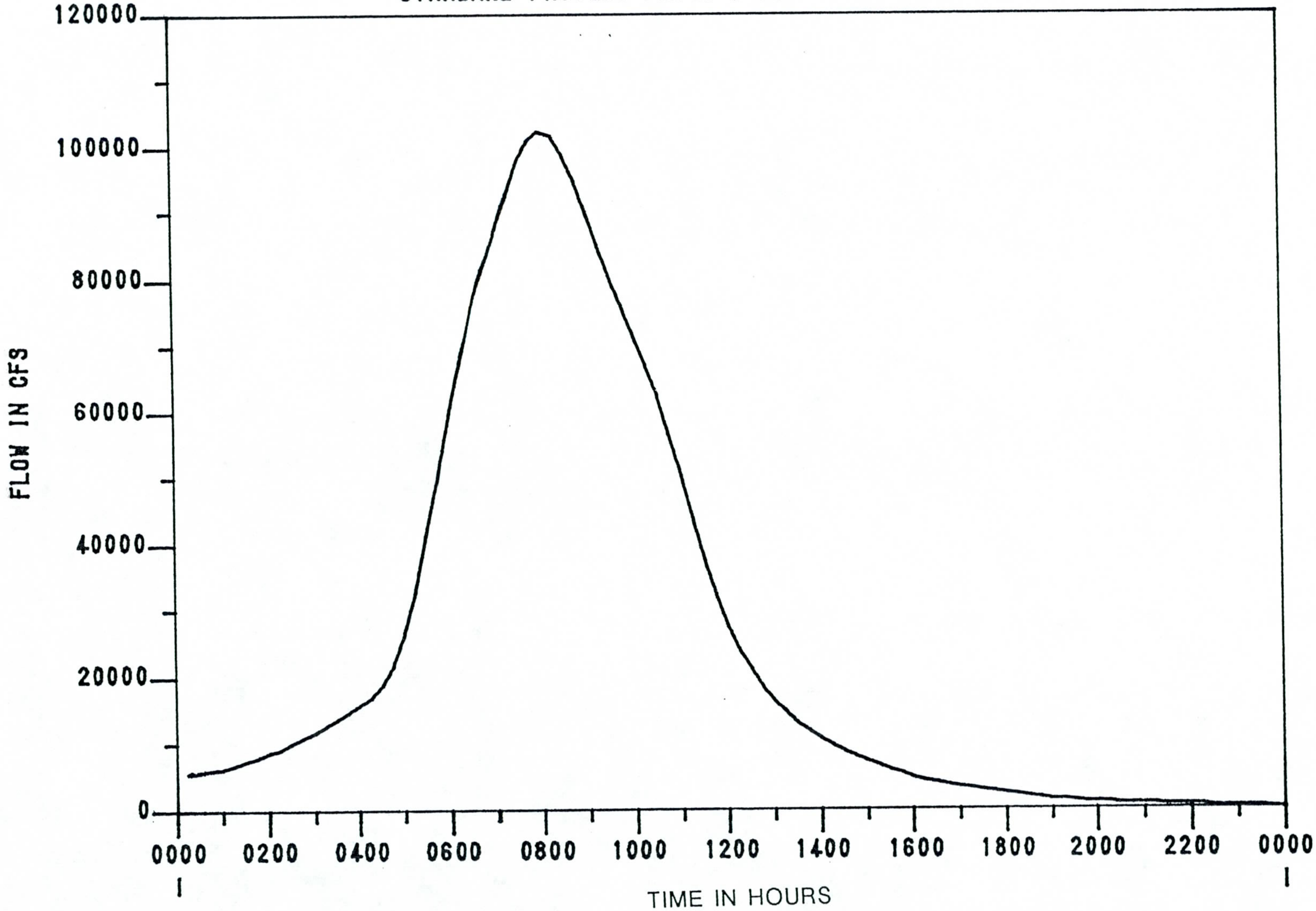
(SOURCE: REF. 2)

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, AZ.

ARIZONA
STANDARD PROJECT LOCAL
SUMMER STORM
PRECIPITATION PATTERNS

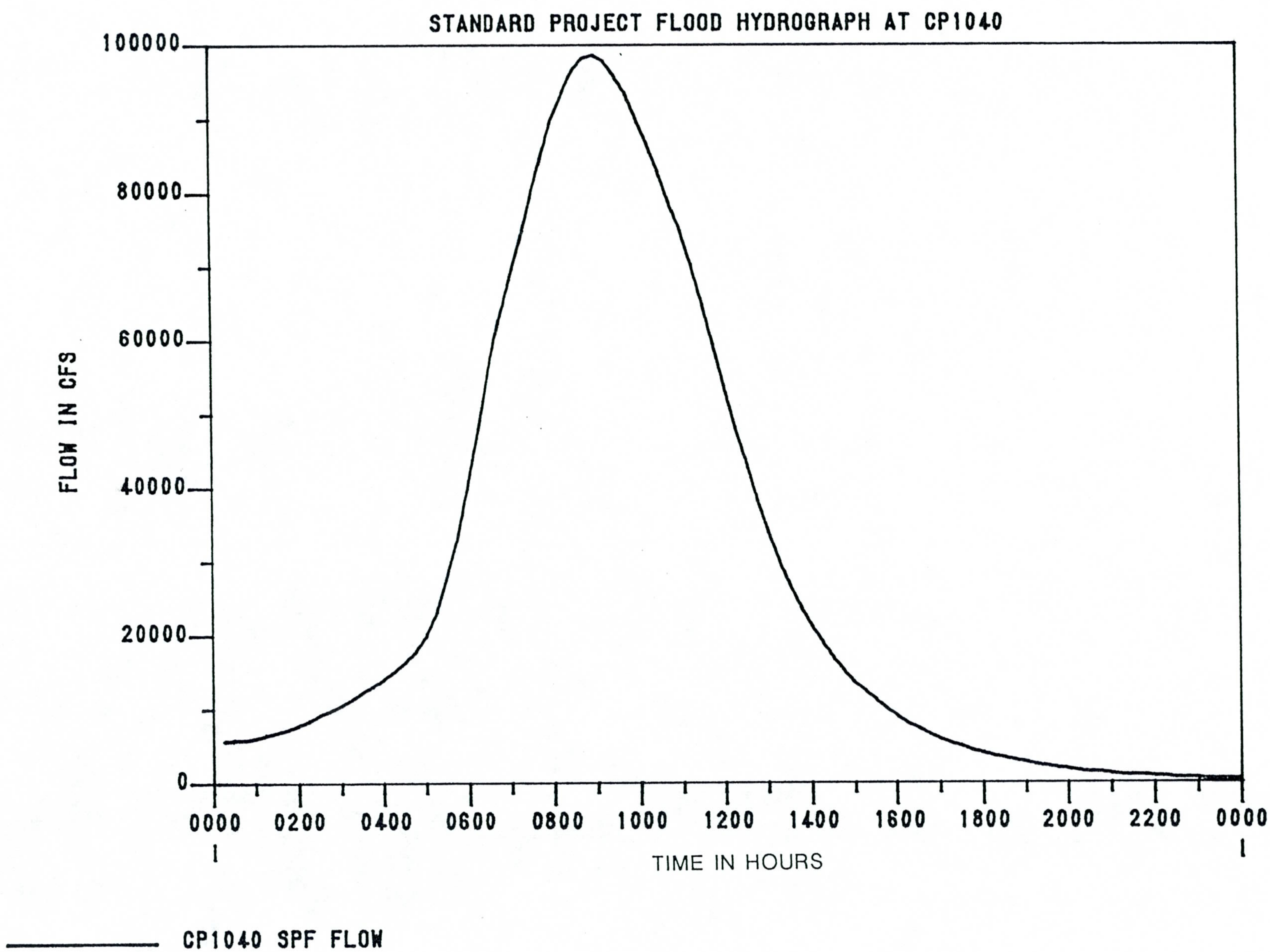
US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

STANDARD PROJECT FLOOD HYDROGRAPH AT CP1039D



CP1039D SPF FLOW

FIGURE 1



STANDARD PROJECT FLOOD HYDROGRAPH AT CP1040
FLOW IN CFS
100000
80000
60000
40000
20000
0
0000
0200
0400
0600
0800
1000
1200
1400
1600
1800
2000
2200
0000
TIME IN HOURS
CP1040 SPF FLOW

FIGURE 2

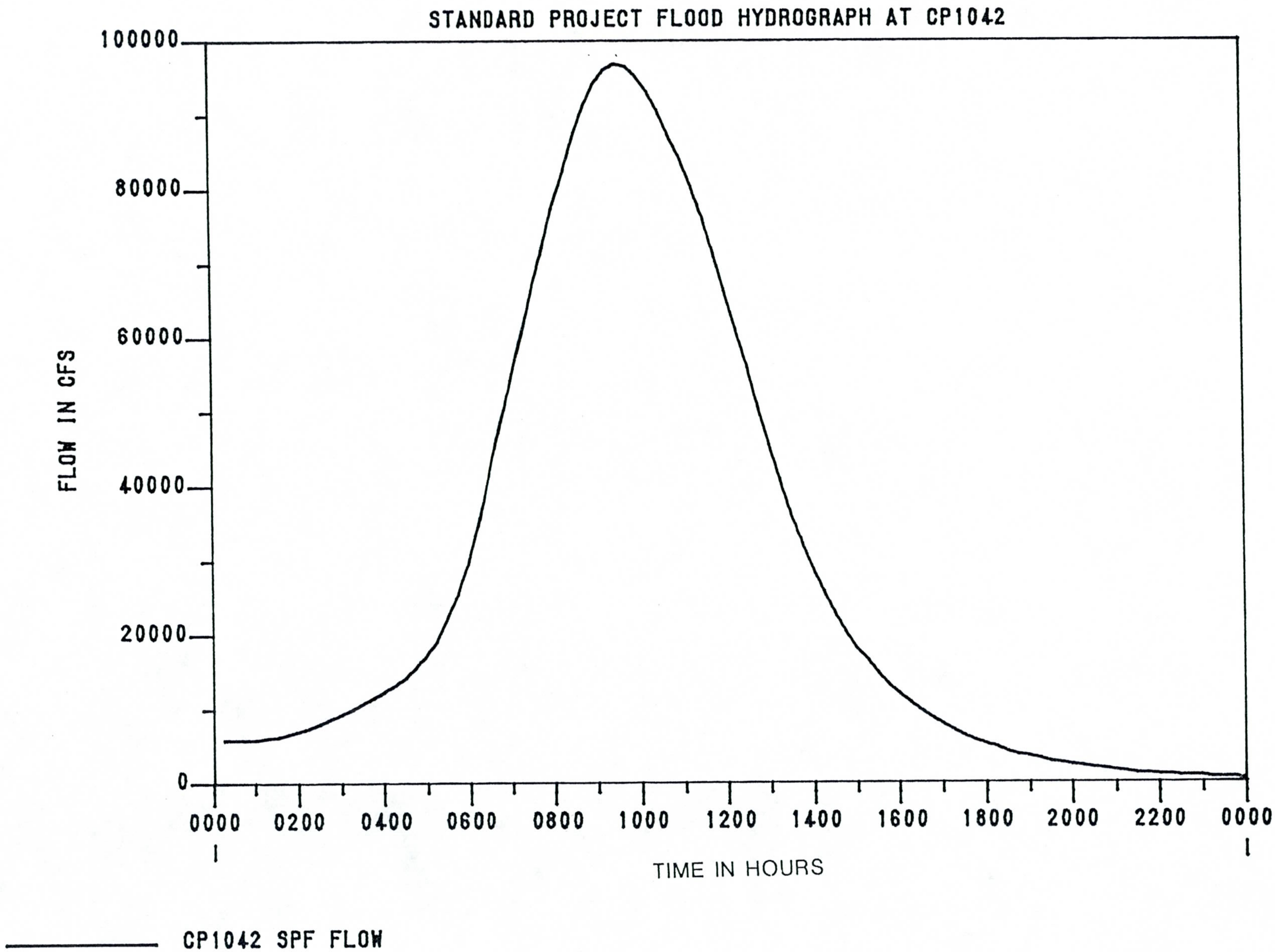

STANDARD PROJECT FLOOD HYDROGRAPH AT CP1042
FLOW IN CFS
100000
80000
60000
40000
20000
0
0000
0200
0400
0600
0800
1000
1200
1400
1600
1800
2000
2200
0000
TIME IN HOURS
CP1042 SPF FLOW

FIGURE 3

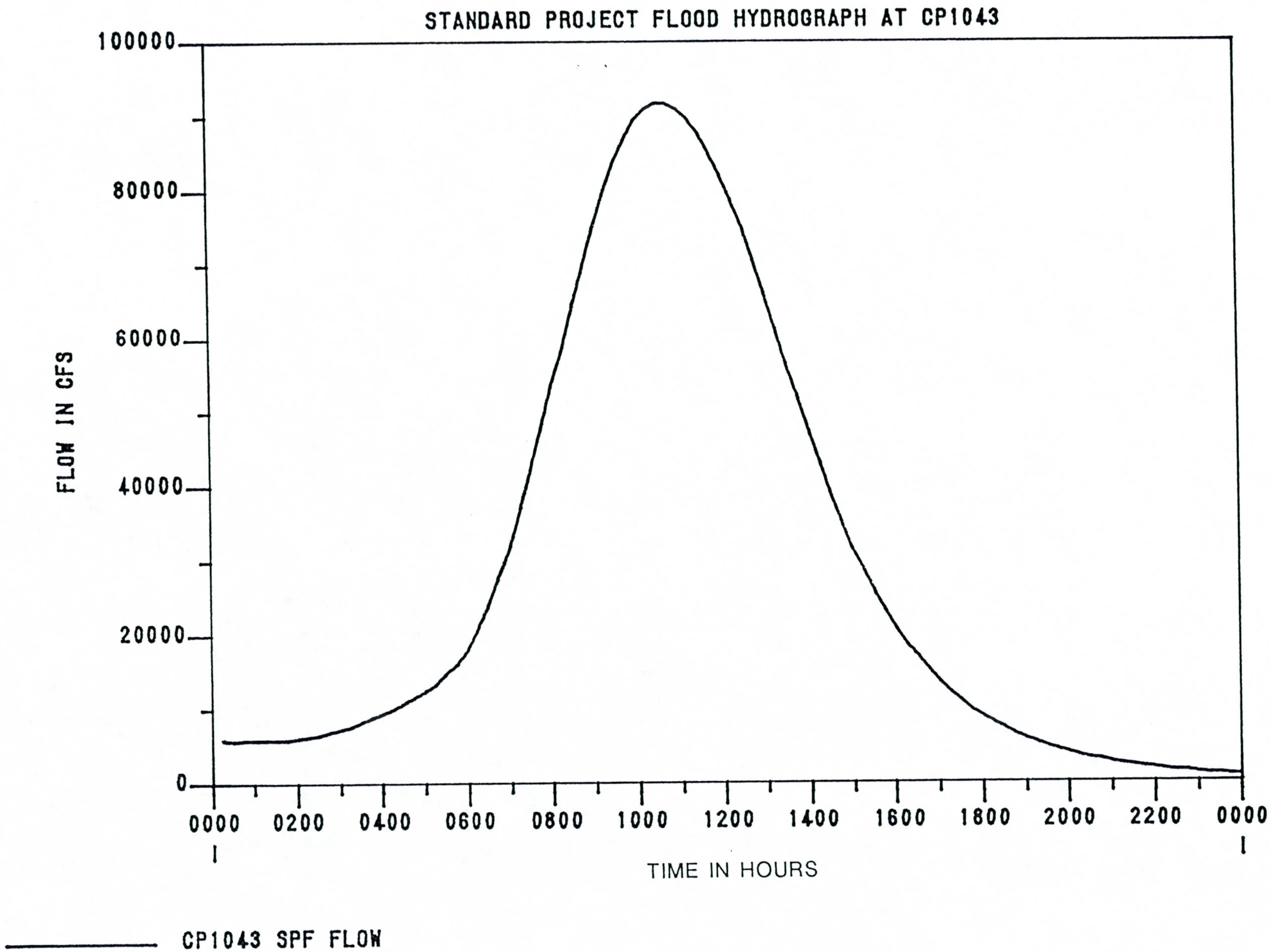

STANDARD PROJECT FLOOD HYDROGRAPH AT CP1043
FLOW IN CFS
100000
80000
60000
40000
20000
0
0000
0200
0400
0600
0800
1000
1200
1400
1600
1800
2000
2200
0000
TIME IN HOURS
CP1043 SPF FLOW

FIGURE 4