US Army Corps
of Engineers
Los Angeles District

Gila River Basin

Phoenix, Arizona, and Vicinity
(Including New River)

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Arizona Canal Diversion Channel
Dreamy Draw to Cudia City Wash
Economic Analysis

FINAL

March 1987

A118.937

Gila River Basin

Phoenix, Arizona and Vicinity

(Including New River)

Arizona Canal Diversion Channel

Dreamy Draw to Cudia City Wash

Economic Analysis

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EXECUTIVE SUMMARY

The Phoenix, Arizona, and Vicinity (including New River) flood control project was authorized by Congress in the Flood Control Act of 1965. One of the major features of the project was the Arizona Canal Diversion Channel (ACDC), extending from Skunk Creek to Dreamy Draw, a distance of about 13 miles. After severe flooding in June of 1972, the local sponsor, the Flood Control District of Maricopa County, at the request of the City of Phoenix, asked the Corps of Engineers to extend the ACDC about 4.2 miles to control Cudia City Wash and a number of smaller washes and drainage areas between Cudia City Wash and Dreamy Draw. After performing the appropriate analyses, including economic studies to establish the economic feasibility of the extension, the Corps of Engineers extended the ACDC under the Chief of Engineers' authority to make limited modifications to Congressionally authorized projects.

In their mark-up of the Energy and Water Development Appropriations Act for fiscal year 1986 (Public Law 99-141, dated November 1, 1985), the House Appropriations Subcommittee for Energy and Water Development directed the Corps of Engineers to conduct a cost-benefit analysis of the segment of the ACDC from Dreamy Draw to Cudia City Wash, now commonly known locally as Reach 4. This report presents the findings of this analysis.

One criterion for Federal investment in flood control projects is that the benefits derived from the project must exceed the costs of the project. For the purpose of this economic analysis, new hydrologic, hydraulic and economic studies were conducted. Such studies call for considerable engineering judgment to estimate flood characteristics for rare (large) events. The Los Angeles District has attempted to ensure reasonable judgments and results by comparison to historical floods. Little or no flooding damage should occur until floods are large enough to break across the Arizona Canal, and damage estimates from large floods should bear reasonable relationships to the 1972 flood, the largest flood for which adequate records exist. The benefit-to-cost

ratio for Reach 4, as determined by the studies conducted for this report, is 1.08 based on the discount rate (3-1/4 percent) established by law for evaluation of the Phoenix and Vicinity project. Thus, Reach 4 meets this particular criterion for Federal investment. Based on the fiscal year 1987 discount rate (8-7/8 percent) specified by the Water Resource Council for the economic evaluation of new water resource projects, the benefit-to-cost ratio is 0.4. Reach 4 does not meet the benefit-to-cost ratio criterion for Federal investment under the interest rate currently specified for new projects.

The remaining benefit-remaining cost ratio (RB/RC) displays the proportion of remaining annual benefits to remaining annual costs. These costs for Reach 4 do not include design costs already incurred to date and the costs of rights-of-way already acquired. At the 3-1/4 percent interest rate, the RB/RC for Reach 4 is 1.3. At 8-7/8 percent, the RB/RC is 0.5.

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1. Hydrologic Analysis

2. Hydraulic Analysis

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AUTHORITY

The basic authority for the Phoenix, Arizona and Vicinity (including New River) flood control project is provided by the Flood Control Act of 1965 (Public Law 89-298), which states, in part, under Section 204 that

* * * * * * *

> "The project for flood protection at Phoenix, Arizona and vicinity is hereby authorized substantially in accordance with the recommendations of the Chief of Engineers in House Document Numbered 216, Eighty-Ninth Congress...."

* * * * * * *

The Arizona Canal Diversion Channel extending from Skunk Creek to Dreamy Draw (pl. 1), a distance of about 13 miles, was one of a number of project flood control features included in the plan presented in House Document 216. After severe flooding in June 1972, the local sponsor, the Flood Control District of Maricopa County, at the request of the City of Phoenix, asked the Corps to extend the ACDC to control Cudia City Wash and a number of smaller washes and drainage areas. After performing the appropriate analyses, the Los Angeles District of the Corps of Engineers prepared a Post-Authorization Change report recommending that the proposed extension of about 4.2 miles be added to the project. In June of 1977, the Office of the Chief of Engineers approved the report, and the extension was added to the ACDC under the Chief's authority to make limited modifications to Congressionally authorized flood control projects. (See pl. 2.)

Further authority for this report is contained in the House mark-up of the Energy and Water Development Appropriations Act for fiscal year 1986 (Public Law 99-141, dated November 1, 1985) in which the House Appropriations Subcommittee for Energy and Water Development directed

"that the U.S. Army Corps of Engineers conduct a cost-benefit analysis, using current Federal guidelines and policy directives, of that portion of the Phoenix, Arizona, and Vicinity (including New River) Flood Control Project known as Reach 4 from Dreamy Draw to Cudia City Wash, and provide this analysis to the Committee for its review."

PURPOSE AND SCOPE

The purpose of this report is to present the results of the cost-benefit analysis of the Dreamy Draw to Cudia City Wash segment (Reach 4) of the Arizona Canal Diversion Channel (ACDC) as directed by the House Appropriations Subcommittee for Energy and Water Development.

The scope of the studies conducted to prepare this report was limited to that necessary to perform the required economic analysis for flood control only. Recreation was not included. Specific new studies conducted for the analysis included hydrologic studies to determine the magnitude of floodflows for various sizes of floods, hydraulic studies to determine areas and depths of flooding for these same sizes of flood, and economic studies to determine projected damages with and without the proposed project. The design and cost estimates that were available for the proposed project were revised to reflect the most recent design modifications and October 1986 prices. A new design and cost estimates were prepared for Reach 3 without any flow contribution from Reach 4 so that the incremental cost for Reach 4 could be determined. All of these studies were limited to the areas directly affecting or affected by Reach 4. Part of the cost of Reach 4 is an increase in the size and, therefore, the cost of the ACDC downstream from Reach 4 between Dreamy Draw and Cave Creek. This cost increase was determined and included in the Reach 4 costs.

DESCRIPTION OF STUDY AREA

LOCATION AND EXTENT

The Arizona Canal is a major irrigation canal running southeast to northwest across the City of Phoenix and finally terminating near Skunk Creek. Historically, floodflows eminating from washes and drainage areas north of the canal have ponded against and broken over the canal to cause flooding both upstream and downstream from the canal. The ACDC will be constructed adjacent and nearly parallel to the north side of the Arizona Canal. The purpose of the ACDC is to intercept the floodflows from the north and to transport them to Skunk Creek. Reach 4 will perform this function between Cudia City Wash, just west of 40th Street, and Dreamy Draw, just west of 12th Street (See pl. 2.)

Hydrology studies included drainage areas both above and below the Arizona Canal. The drainage areas above the canal extend to the ridges or divides in the mountains and hills north of the canal. The drainage areas are outlined on plate 1-2 of Appendix 1, Hydrologic Analysis. Hydrologic studies south of the Arizona Canal were necessary to differentiate between flooding resulting from flows originating north of the canal and flows originating south of the canal. The areas addressed below the canal in the hydrologic studies coincide with the areas determined to be flooded in the hydraulic analysis.

Hydraulic studies for determining flood damages were limited to those areas determined to be subject to flooding from flows originating from drainage areas located north of the Arizona Canal and east of Dreamy Draw to and including Cudia City Wash. Dreamy Draw was not included. The areas subject to flooding (see pls. 2-1 and 2-2 of the Hydraulics Appendix) extend in a southwesterly direction from the Arizona Canal to the Papago Freeway. The east and west extremes of the flood delineations are 40th Street and 13th Avenue, respectively.

A hydraulic design study was required to determine the size of the ACDC if Reach 4 is not built as a part of the Federal project. This study was necessary to determine the cost of Reach 3 of the ACDC (from Cave Creek to Dreamy Draw) attributable to Reach 4.

Economic studies to determine property values and flood damages were limited to the area subject to flooding from the streams and drainage areas tributary to Reach 4.

TOPOGRAPHY AND STREAM CHARACTERISTICS

The upper end of the study area is bounded by hills and low but steep mountains. The most prominent natural feature in the drainage area is Squaw Peak. The gross drainage area above the Arizona Canal is comprised of a number of roughly parallel and largely independent drainage areas ranging in size from 1.18 square miles to 4.91 square miles. The average slopes of the basins are fairly steep, ranging from 222 to 390 feet per mile. The largest of the streams tributary to Reach 4 is Cudia City Wash with a 100-year (1 percent chance of occurrence per year) peak discharge of 6200 cubic feet per second (ft^3/s). Other major concentrated inflows (defined as 1000 ft^3/s or greater for the 1 percent chance flood) will occur between Cudia City Wash and 32nd Street (2000 ft^3/s) and downstream from Ocotillo Road (1600 ft^3/s). The peak flows listed in this paragraph are smaller than peak flows listed in previous design documents because they are the result of a single storm centering above the Arizona Canal rather than the multiple-storm centerings that were used to determine peak design flows from the tributaries for the ACDC. Storm centerings are discussed in more detail on page 1-8 of Appendix 1.

No identifiable streams exist below the Arizona Canal in the Reach 4 study area. Runoff below the canal is carried in the streets. When the flow carrying capacity of the streets is exceeded, storm runoff flows over and through urban development. Slopes downstream from the Arizona Canal average about 21 feet per mile.

Additional peak flow and other hydrologic data both above and below the Arizona Canal are presented in tables 1-1 and 1-2 of Appendix 1, Hydrology.

ECONOMIC DEVELOPMENT

An inventory was conducted of existing development in the area determined to be subject to flooding in the hydraulic analysis. Under present conditions, about 21,000 single-family homes and about 3800 multi-family residences, many businesses, and extensive public facilities lie within the limits of the Standard Project Flood (SPF), the largest flood analyzed. The SPF is defined as the flood resulting from the most severe meteorologic and hydrologic conditions considered reasonably likely to occur in the drainage area. For the area addressed in this report, the SPF would have an estimated frequency of exceedence of about once in 500 years. The area was judged to be virtually 100 percent developed, and, therefore, no future development was assumed for the purpose of determining future flood damages. The total value of the property subject to flooding by the SPF was estimated at $2.4 billion.

FLOODING

FLOOD CHARACTERISTICS

The drainage areas tributary to the study area are small and steep, and peak flows from these drainage areas occur quickly, leaving little time for warning or preparation for flood-fighting measures. Flood volumes are small and durations short. Flooding patterns below the Arizona Canal are only partially predictable. The locations of inflow concentrations from the north and of spillways in the north and south banks of the canal provide some indications of where flooding will most likely occur.

The locations of breaks in the canal banks and the effects of existing development south of the canal are much less predictable. The locations of breaks in the canal banks are dependent upon physical conditions of the banks as much as the location of inflows, and therefore the locations of these breaks can vary from one flood event to another.

Theoretical determination of flooding depths and velocities in highly developed urban areas are normally determined using idealized assumptions of average depths and velocities. In real flooding events, depths and velocities tend to vary widely in developed areas. This situation is particularly true in the study area. Generally speaking, flooding depths and velocities will be the greatest immediately downstream from the Arizona Canal. Maximum depth under present conditions during the SPF will be a little under 4 feet. Depths will decrease as the flood moves southward to a maximum depth of about 1.5 feet deep at the Grand Canal. Depths and velocities will not be constant across the flood plain, but will vary widely. Detailed average depths for various floods throughout the flood plain are provided in Appendix 2, Hydraulic Analysis.

FLOOD HISTORY

Accounts of flooding in the study area are contained in three sources, in files of the Salt River Valley Water Users Association, or Salt River Project (SRP), as it is commonly known, in the Corps of Engineers' flood damage report on the flood of June 22, 1972, and in the Flood Control District of Maricopa County (FCDMC) report on the flood of August 1986.

The SRP records do not provide flood-damage data, but they do provide a good source of information on when flooding occurred. During flooding situations, SRP personnel patrol the Arizona Canal. After flooding events that damage the canal, SRP forces must effect repairs. These activities have resulted in a legacy of diaries and reports containing information on canal breaks, overflows, and spillway flows that indicate when flooding below the canal is likely to have occurred. Based on the SRP record of canal breaks and other information, flooding below Reach 4 of the ACDC occurred in 1939, 1943, 1972 and 1986. Near misses occurred in 1959 and 1978. In 1959, the canal broke at 7th Street, west of the study area. Floodwaters entering the Arizona Canal in Reach 4 contributed to the resultant flooding. In 1978, the Arizona Canal broke at Cave Creek. The canal did not break in Reach 4, but the canal was running full, and residents below the canal near the Biltmore area in Reach 4 were warned that they might have to evacuate if conditions worsened.

The June 1972 flood is the largest flood for which a detailed record of flood damages is available. This record is contained in the Corps of Engineers "Report on Flood of 22 June 1972, Phoenix Metropolitan Area, Arizona." The report is dated October 1972. Total damages in the Phoenix Metropolitan area (including Scottsdale) were $10,558,000. The report lists flood damages of $3,856,000 (1972 dollars) between the Arizona Canal and the Grand Canal as a result of breaks in the Arizona Canal between 16th Street and Cudia City Wash. Reach 4 would have

prevented these damages. Flood damages occurred north of the Arizona Canal due to ponding behind the canal in the 1972 flood. The flood-damage report estimates these damages at $608,000 (1972 dollars) in the City of Phoenix between 64th Street and Cave Creek. No breakdown was provided to show how much of the damage north of the canal occurred in the Reach 4 area, which comprises about one-third of the total distance. Reach 4 may have prevented approximately one-third of these damages. Damages also occurred south of the Grand Canal. Floodwaters originating north of the Grand Canal ponded along and caused breaks in and overflows of the canal resulting in damages estimated at $2,568,000 (1972 dollars). Some of these damages may have resulted from flows originating locally below the Arizona Canal or from Dreamy Draw, but a substantial portion of these damages are attributable to flows originating above the Arizona Canal between 16th Street and Cudia City Wash. These latter flows would have been intercepted by Reach 4 and their damages would have been prevented. The United States Geological Survey (USGS), using slope-area measurements, estimated the peak flow for this flood on Cudia City Wash upstream from MacDonald Drive at 3000 cubic feet per second (ft^3/s). This estimate is published in the section "Discharge Measurements of Miscellaneous Sites" of their "Water Resources Data for Arizona, Part 1. Surface Water Records, 1972." The 1972 flood peak discharges on Cudia City Wash were approximately 50-year floods (2 percent chance of being equalled or exceeded in any one year).

The most recent event that produced flood damages occurred August 28 and 29, 1986. Damages of approximately $110,000 occurred. About 30 homes were flooded according to the FCDMC report. This flood is estimated to be about a 10 year event (10 percent chance of being equalled or exceeded in any one year.

APPROVED PROJECT

The approved Federal flood control project for Phoenix, Arizona and Vicinity (including New River) is shown on plate 1. The plan includes 4 dams, the Arizona Canal diversion channel (ACDC), and flowage easements and limited structural measures along Skunk Creek, the New River and the Agua Fria River.

Construction has been completed on the 4 dams, Dreamy Draw Dam, Cave Buttes Dam on Cave Creek, Adobe Dam on Skunk Creek, and the New River Dam. The dams are all earth-fill, flood control dams, and no permanent pools will be maintained behind them. Construction has also been completed on the first reach of the ACDC, the trapezoidal earth-bottom channel from Skunk Creek to 51st Avenue and Cactus Road.

When completed, the Phoenix and Vicinity project will provide a high degree of flood protection to the Phoenix metropolitan area. The dams will control the Standard Project Floods on their respective streams, and the ACDC will provide protection to property between the channel and the Salt River from flood flows generated in the drainage areas between the dams and the ACDC.

Recreation facilities have been planned for three of the four dams and along the ACDC and along Skunk Creek, the New River and the Agua Fria River. Recreation construction has been slow to develop because of limited financial capability of local recreation sponsors. A first phase of construction has been completed and a second phase has been started to provide trailhead facilities at Dreamy Draw Reservoir. The Dreamy Draw facilities, which include an underpass under Northern Avenue, picnic ramadas, a restroom and other facilities, have been designed to compliment the extensive horse trail system through the adjacent Phoenix Mountain Preserve.

Family camping and day use recreation facilities are planned for the reservoirs at Adobe and Cave Buttes Dams. Day use facilities are also planned along the ACDC in the cities of Glendale and Peoria and at the Cave Creek sediment basin. Recreation development along the ACDC, Skunk Creek, the New River and the Agua Fria River will be limited to trail use amenities.

The ACDC is designed to control the 100-year flood (an event with a one percent chance of occurring in any given year) and will be constructed adjacent and nearly parallel to the north side of the Arizona Canal. The upstream end of the channel will be at Cudia City Wash, near 40th Street, and the mouth is at Skunk Creek, about 17 miles to the west. Channel design capacities range from 6,700 cubic feet per second at Cudia City Wash, to 29,000 cubic feet per second at the mouth. The upstream 12 miles of the channel will be rectangular concrete in section ranging from 36 feet wide at the upstream end to 60 feet wide upstream of Cave Creek. Downstream of Cave Creek the rectangular channel will be 110 feet wide. A trapezoidal concrete channel with base widths of 80 feet to 100 feet and 2 horizontal to 1 vertical side slopes will connect the rectangular concrete channel with the downstream 4 miles of earth channel. The bottom width of the earth channel will vary somewhat, but will be 220 feet in most places. The side slopes will be mostly 6 horizontal to 1 vertical with some variation to steeper slopes. Depths in the ACDC will range from about 20 to 25 feet. The top of the channel will be at or below ground level throughout most of its length to facilitate side inflows.

A sediment transport analysis showed that two streams tributary to the ACDC, Cudia City Wash and Cave Creek, have a potential for causing sediment inflows that could significantly affect channel carrying capacity. To resolve this problem, sediment basins were added on the two streams.

For construction contracting purposes, the ACDC has been divided into 4 segments, or reaches, of roughly equal length. The reaches are numbered from downstream to upstream in the order of scheduled construction. The fourth reach, or Reach 4 (see plate 2) the subject of this report, extends from Dreamy Draw to Cudia City Wash and will be 36 feet wide at Cudia City Wash and will increase to 40 feet wide upstream from Dreamy Draw. Depths will range from 21.5 feet to 24.5 feet. Two short covered sections have been included in reach 4. About 1300 feet of channel under Stanford Drive and about 4625 feet of channel through the Arizona Biltmore Estates will be covered because covering proved to be the least cost construction alternative through these reaches. Detailed design for Reach 4 is presented in "Design Memorandum No. 12, Feature Design for Arizona Canal Diversion Channel - 40th Street to Cactus Road," dated April 1986.

SUMMARY OF THE ECONOMIC ANALYSIS

"Appendix 3, Economic Analysis" contains a detailed description of the methodology used to test the economic feasibility of Reach 4. Detailed data derived for the economic analysis are also contained in Appendix 3. Economic feasibility of a Federal flood control project is demonstrated by a benefit-to-cost (B/C) ratio of one or greater.

The B/C ratio is determined by dividing the equivalent annual benefits produced by the project by the average annual costs of constructing and operating and maintaining the project. The B/C ratio for Reach 4 at a discount rate of 3-1/4 percent was determined to be 1.08. The 3-1/4 percent interest rate is the rate established by statute (Section 80(b), Public Law 93-251, Water Resources Development Act of 1974) for application to the Phoenix, Arizona, and Vicinity (including New River) project. At the fiscal year 1987 discount rate of 8-7/8 percent specified by the Water Resources Council for analyzing new Federal water resource projects, the B/C ratio for Reach 4 was determined to be 0.4.

The remaining benefit-remaining cost ratio (RB/RC), which displays the proportion of remaining annual benefits to remaining annual costs, was also determined. These costs for Reach 4 do not include design costs already incurred to date and the costs of rights-of-way already acquired. At the 3-1/4 percent interest rate, the RB/RC for Reach 4 is 1.3. At 8-7/8 percent, the RB/RC is 0.5.

Equivalent annual benefits to be derived from Reach 4 were determined by subtracting estimated average annual flood damages that would occur with the project from the estimated average annual damages that would occur without the project.

Average annual costs for the project were determined by amortizing estimated first costs over a 100-year life and adding estimated annual operation and maintenance costs. The first costs for construction of Reach 4, itself, were based on "Design Memorandum No. 12, Feature Design Memorandum (FDM) for Arizona Canal Diversion Channel - 40th Street to Cactus Road", dated April 1986. The FDM cost of $68,590,000 was updated for minor design revisions and price level changes (from October 1985 to October 1986) resulting in a Reach 4 cost of $72,760,000. Costs of design, construction, rights-of-way aquisition and relocations are included. In addition to the costs within reach 4, the total cost of Reach 4 includes the cost of increasing the size of the ACDC downstream from Reach 4 to accommodate Reach 4 flows. Determining the latter cost required determining the ACDC channel size without Reach 4, estimating the cost of that size of channel, and then subtracting the cost of the reduced sized channel from the cost of the channel designed to carry Reach 4 flows. This procedure provides an estimate of the incremental cost ($7,090,000) of the downstream channel to be attributed to Reach 4. With no inflow from Reach 4, design flows would reduce dramatically at the upper end of the remaining ACDC channel at Dreamy Draw, however, the effect of eliminating Reach 4 reduces with distance downstream. The following tabulation lists ACDC design flows with and without Reach 4:

Location	With Reach 4 (cfs)	Without Reach 4 (cfs)
Downstream Dreamy Draw	10,000	1,400
Downstream 10th Street Drain	13,000	6,400
Upstream Cave Creek	14,000	10,000
Downstream Cave Creek	25,000	21,000
Upstream Skunk Creek	29,000	25,000

For purposes of this report, no downsizing of the ACDC west of Cave Creek for the without Reach 4 condition was assumed for the following reasons:

1. The first 4-mile reach of the channel is complete and construction has started on the next mile.

2. The difference in the respective design flows is small, and a large part of any cost savings from downsizing could be largely absorbed by the cost of redesigning and by delays while awaiting a final decision on Reach 4.

3. The rights-of-way for the channel west of Cave Creek have almost all been acquired based on the current design (with Reach 4). Also, alternative studies have shown that decreases in design depth, rather than decreases in design width, would be more effective in reducing cost. These two reasons coupled with the small difference in design flow show that no reduction in rights-of-way would occur from down-sizing.

4. The current design (with Reach 4) maximizes the use of available rights-of-way, and thus, protection from Cave Creek, which is the greatest flood threat of all the streams tributary to the ACDC. Thus, the level of flood protection would be increased if Reach 4 were to be deleted from the project.

Pertinent cost and benefit data from the Economic Analysis is shown in the following table.

Arizona Canal Diversion Channel
(40th Street to Dreamy Draw)

Economic Analysis [a]

First Cost (October 1986 price levels)

Construction Reach 4	$46,800,000	
Reach 3[b]	7,090,000	
Lands and Damages	17,385,000	
Relocations	8,575,000	
Subtotal	$79,850,000	
Interest during construction	1,577,000	
Total	$81,427,000	
	(3-1/4%)	(8-7/8%)

Average Annual Cost

Interest and Amortization	$ 2,004,000	$3,151,000
Operation and Maintenance	88,000	88,000
Total	$ 2,092,000	$3,239,000

Equivalent Annual Benefits

Damages prevented	$ 2,254,000	$1,329,000
Excess of benefits over cost	$ 162,000	$-1,970,000
Benefit-to-cost ratio	1.08	0.4

a. First costs and benefits were discounted to 1981 and annualized over a 100-year project life. All numbers rounded to four significant figures.

b. Increased cost ($7,090,000) of required larger channel from Dreamy Draw to Cave Creek.

COMPARISON TO 1976 GENERAL DESIGN MEMORANDUM

Direct detailed comparison of damages and benefits determined in this analysis to the Phase I General Desgin Memorandum (GDM) published in March 1976 is not possible because of limitations on the data available from the earlier work.

Comparisons that can be made or inferred show that damages derived in this later analysis are substantially higher. The following limitations in the Phase I GDM work may have contributed to underestimation of overflow areas and hence flood damages:

a. No evidence is available that local runoff below the Arizona Canal was considered.

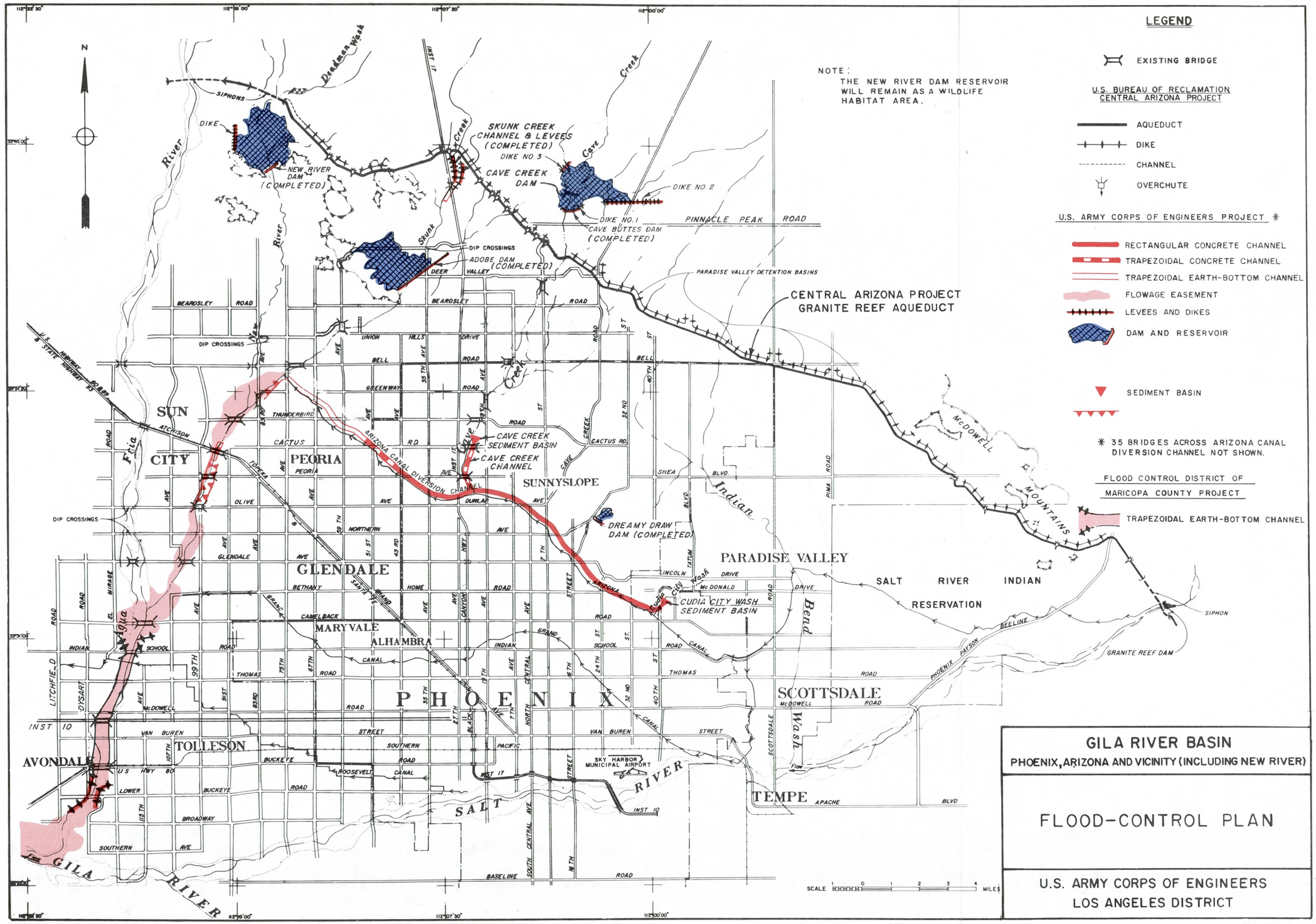
b. Damages below the Grand Canal were probably underestimated because of item a above and resultant flooding below the Grand Canal was very shallow.

c. The 1972 historical flood event was assumed to be a 50-year (2 percent chance of occurrence) event, and the damage data for that flood were price leveled and used directly in the damage-frequency analysis, rather than a hypothetical event which is normal practice. The 1972 event was determined to be about a 50-year event at one point on Cudia City Wash, but it was less than a 50-year event over the entire Reach 4 watershed and floodplain.

Of these three reasons considered, use of the 1972 historical data as representative of a 50-year event throughout the ACDC reaches had by far the greatest effect. This approach resulted in a very conservative estimate that understated potential damages. This report uses the 1972 flood to verify the without-project 50-year damages shown in table 3-5. However, use of a hypothetical flood which assumes a 50-year event over the entire floodplain results in a significantly larger overflow area, and hence, greater total damages.

CONCLUSIONS

This study has shown that, at the 3-1/4 percent interest rate established by law for the project (Section 80(b), Public Law 93-251, Water Resources Development Act of 1974), Federal investment in Reach 4 of the ACDC is justified, and the findings of the Phase I General Design Memorandum, completed in March 1976 are verified. At an interest rate of 8-7/8 percent, specified for Fiscal Year 1987 by the Water Resources Council for Federal investment decisions in new water resource projects, Reach 4 falls short of being economically justified with a B/C ratio of 0.4.

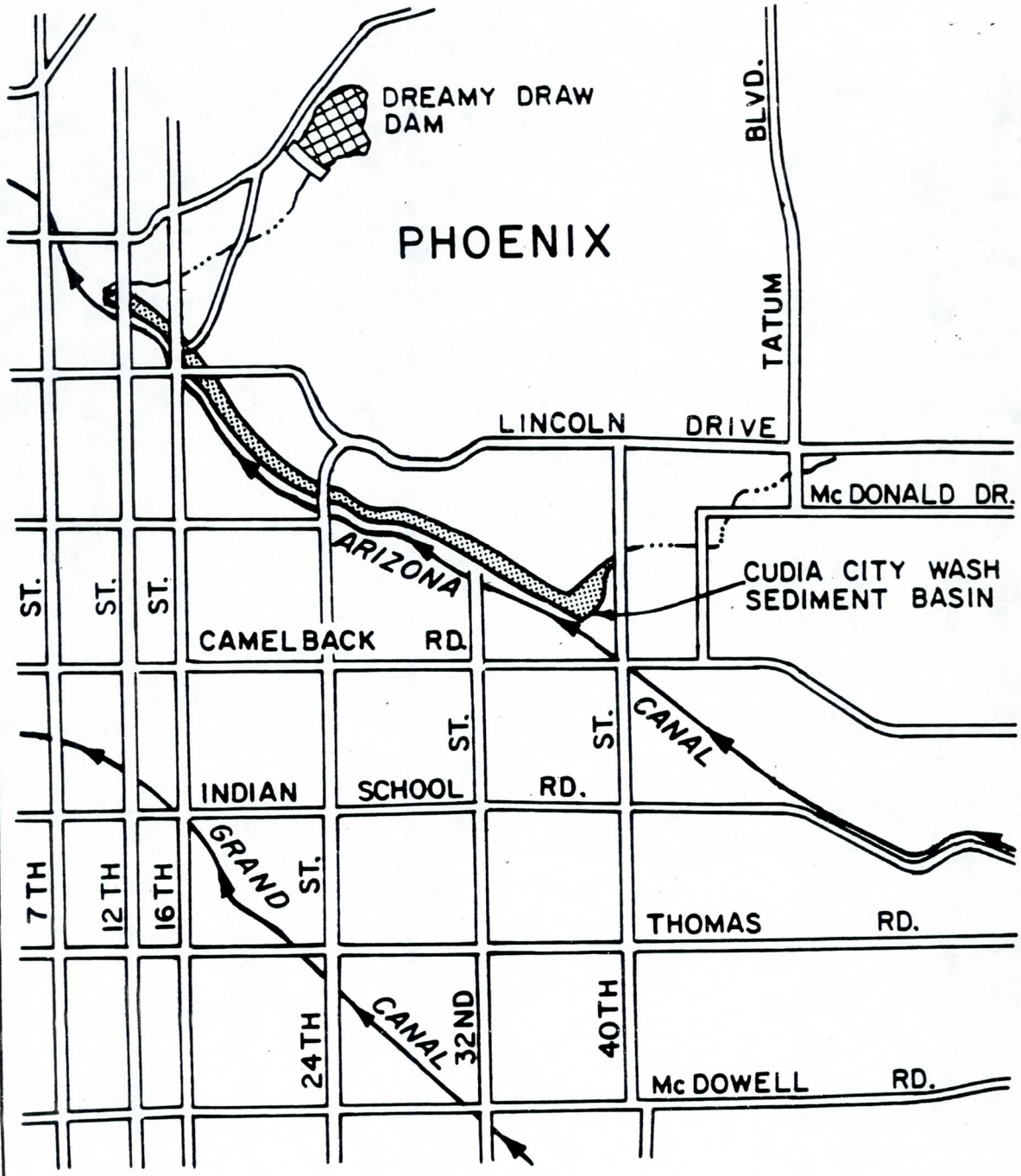


GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

FLOOD-CONTROL PLAN

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 1



NOT TO SCALE

REACH 4

GILA RIVER BASIN
PHOENIX, ARIZ. & VIC. (INC. NEW RIVER)

ARIZONA CANAL DIVERSION CHANNEL
DREAMY DRAW TO CUDIA CITY WASH
REACH 4

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

Appendix 1
Hydrologic Analysis

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I. INTRODUCTION

This appendix presents the development of discharge-frequency values for use in performing overflow studies on Reach 4 of the ACDC. Discharge-frequency values were developed for both with and without project (Reach 4 of the ACDC). Depth-discharge-frequency relationships and the overflow results are presented in Appendix 2, Hydraulic Analysis, of this report.

II. PLAN OF IMPROVEMENT

The project features of ACDC Reach 4 are discussed in the main report. Reach 4 of the ACDC will be just upstream from and parallel to the Arizona Canal. (See pls. 1-1 and 1-2.) It will extend about 4.2 miles from Cudia City Wash westward to Dreamy Draw near 12th Street. The design capacity of the ACDC (100-year future condition peak discharge) ranges from 6700 ft^3/s at Cudia City Wash to 9000 ft^3/s above the Dreamy Draw confluence. The ACDC will be entrenched for most of its length to allow side inflow over the channel walls. In local areas adjacent to the channel where ponding occurs, pipe inlets will be provided. Confluence structures will be required at major washes, such as Dreamy Draw, 32nd Street, 16th Street, Ocotillo Road, and Cudia City Wash. During Phase II GDM studies (Ref. 1), a sediment basin design was included at the mouth of Cudia City Wash. Another feature includes covering the ACDC through the Biltmore Estates from about 1800 feet west of 32nd Street to 24th Street (a distance of about 4600 feet) in order to allow for maximum use of prime property. The channel will also be covered for about 1300 feet from 32nd Street eastward.

III. STRUCTURES AFFECTING RUNOFF

As shown on plate 1-2, there are two existing westward flowing canals in the study area, the Arizona Canal and the Grand Canal. These canals have an approximate capacity of 700 ft^3/s each and a slope of about 0.0002 ft/ft. As runoff flows south from the Phoenix Mountains, it first ponds behind the Arizona Canal and then overflows at six locations where the south bank elevation dips. These flows continue southwestward to the Grand Canal where they overflow at approximately four locations. (See pl. 1-3.) Ponding behind the Grand Canal is minimal since the bank is generally not elevated above the ground. Study conditions include these two canals flowing full with irrigation water without extra capacity to carry floodflows. During larger events, ponding volumes behind both canals are small enough to be filled from antecedent runoff alone.

Reach 4 of the ACDC will capture all flow up to the 100-year frequency from the Phoenix Mountains and Cudia City Wash, and transport it to Skunk Creek. Flow exceeding the ACDC design capacity would cross both the ACDC and the Arizona Canal with essentially no attenuation.

The Arizona Department of Transportation (ADOT) has constructed two 21 foot diameter tunnels beginning north of the Casa Grande Freeway extension which is presently under construction. These tunnels, each with a capacity of about 3500 ft^3/s, will carry storm runoff to the Salt River. The City of Phoenix plans to build improvements to direct storm runoff into the ADOT tunnel system. The Casa Grande Freeway will initiate the impoundment and conveyance of local floodwaters. Thus, the southern boundary of subarea AD6 consists of the freeway.

IV. SUMMARY OF METHODOLOGY

The hydrologic methodology used to determine design discharges for the ACDC and related overflows is described in detail in Ref. 2 and briefly summarized below.

A. STANDARD PROJECT FLOOD

The Standard Project Flood (SPF) is 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, and changes in channel characteristics may be incorporated into the hydrologic description of the basin. The standard project storm is then centered over the watershed in the most critical flood-producing manner. Application of the rainfall-loss-rate function enables determination of the rainfall excess, which is then applied to the subbasin unit hydrographs to produce the subbasin flood hydrographs. The subbasin flood hydrographs are then combined and routed to the desired concentration point. The elements involved in the computation are described in the following paragraphs.

1. Drainage Area Boundaries. Drainage area boundaries above the Arizona Canal were determined during Phase I studies. Drainage area boundaries below the Arizona Canal were determined from the combination of canal breakout flows and downstream topography. These boundaries are shown on plate 1-2.

2. Standard Project Storm. The August 19, 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, transposed to the study area by means of 10-year, 6-hour precipitation values published by the National Weather Service in NOAA Atlas 2, was used to determine the standard project flood.

3. Storm Centerings. Storm centerings were not made for each individual subarea above the Arizona Canal, as in design studies, since flow in and downstream from the ACDC merge into one overflow area. The combined area above the Arizona Canal (9.9 mi^2) was the smallest storm centering used in determining discharges. For concentration points below the Arizona Canal, the storm was centered over all subareas contributing to the frontal flow line, i.e., A1,B1,C1,D1 plus the area upstream from the ACDC (see pl. 1-2). The flow at the ACDC and Arizona Canal was determined, routed to the downstream concentration point and then added to the local flow generated downstream from the ACDC. The various storm centerings used are listed in tables 1-1 and 1-2.

4. Rainfall-Runoff Relationships. Regional unit hydrograph and loss-rate studies for the general Phoenix region are described in detail in Ref. 2 and 3. Twenty-two observed floods were reconstituted during these studies to derive relationships between rainfall and runoff applicable to most basins within the region. The Los Angeles District's 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. The lag time of each subarea is a function of shape, slope, and a basin roughness factor, "n". Basin "n" values varied from 0.025 above the Arizona Canal to 0.15 in the urbanized subareas below the Grand Canal. The Phoenix Valley S-graph was used to determine the time distribution of runoff hydrographs. To express rainfall-runoff relationships, the Hydrologic Engineering Center's (HEC) exponential loss rate function was utilized by applying the following parameters:

> DLTKR - initial accumulated loss during which the loss rate coefficient is increased; STRKR - starting value of the loss coefficient on the exponential loss curve; RTIOL - ratio of the loss coefficient on the exponential loss curve to that corresponding to 10 inches more of the accumulated loss; ERAIN - exponent of precipitation in the loss rate equation. Values for these variables to be used with the local standard project storm were taken from the Part 2 Hydrology Report, (Ref. 2) and are as follows: DLTKR = 1.0; ERAIN = 0; STRKR = 0.38; and RTIOL = 2.0.

For computational purposes, percent impervious describes the watershed under future conditions. Impervious values ranged from 20 percent above the Arizona Canal to 50 percent below the Arizona Canal.

5. Generation of SPF (Without-Project) Flood Hydrographs. Standard project flood peak discharges were computed for both with- and without-project conditions. 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. The following describes the methodology used to determine the without-project total hydrograph.

For the purpose of this hydrologic study, six breakout locations were determined to exist along the canal. The occurrence of breakout was judged to not increase nor attenuate the flood hydrographs. Specifically, a break of the low canal bank would not significantly increase the peak discharge of the breakout flow. Also, the canal would be full at the time of the flood and would not provide sufficient storage to cause significant attenuation of the peak discharge. These breakout hydrographs were then routed (See Section 7) to each concentration point or frontal flow line (See table 1-2) and 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 using the unit hydrograph procedure for drainage areas between the Arizona Canal and the Grand Canal defined by four combined areas (A1+ A2+ A3+ A4), (B1+ B2+ B3+ B4), (C1+ C2+ C3+ C4), and (D1+ D2+ D3+ D4). Next, these hydrographs were ratioed proportionally by drainage area size to obtain hydrographs at each upstream frontal flow line. The time of hydrograph peak at each frontal flow line was judged to be a proportion 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 (A1, B1, C1, D1). This procedure was repeated for each successive reach to the Grand Canal (A2, B2, C2, D2, etc.).

Local discharges at the 5th frontal flow line, which is downstream of the Grand Canal, were also determined by the unit hydrograph procedure. All A, B and C subareas plus AC5 were combined to form one area. Subareas D1 through D5 were combined to form another subarea. See plate 1-2 and table 1-3 for subareas identified. These were then combined to yield a single hydrograph at CP AD5. The SPF peak at CP AD6 was judged to have the same flow per foot of width as CP AD5. Therefore, the hydrograph at CP AD5 reduced by the cross-section width ratio of AD6 to AD5 to obtain the hydrograph at AD6. Up to 7000 cfs of the flow not reaching AD6 could enter the storm drain tunnels north of the Casa Grande Freeway.

6. Generation of SPF (With-Project) Flood Hydrographs. The SPF with project dischrges were determined in a similar manner to the without-project SPF with the following difference. The ACDC diverts up to the 100-year capacity and spills the remaining flow into the Arizona Canal which will breakout at the same locations as in the without-project analysis. The breakout flows were therefore significantly reduced and then routed at lower velocities than the SPF without-project.

7. Flood Routing. There are seven existing detention basins and one proposed sediment basin in this study area, (plate 1-2). Flow through these basins was analyzed using the Modified-Puls routing procedure. For each frequency, the basins were considered to be full to spillway crest with sediment and antecedent flows. The existing detention basin elevation-storage and elevation-spillway discharge relationships are listed in table 1-4.

Channel and concentrated overland routing was accomplished by the Muskingum method. 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. Rating curves of average flow velocity versus discharge per unit cross section width were computed using Manning's equation. 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. 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, are based on judgment. For improved channels, X values of 0.3 to 0.4 are appropriate, depending on the type of improvement. Channel flow occurs in the ACDC and the area north of the Biltmore Estates. For breakout flow, below the Arizona Canal, a X value of 0.0 is appropriate. Table 1-5 shows pertinent Muskingum Routing Data for the ACDC breakout flow.

B. DISCHARGE-FREQUENCY ANALYSIS

Peak discharge-frequency values, with and without the ACDC Reach 4, for future conditions of development were determined in part from the discharge-frequency relationships in the Part 2 Hydrology Report, (Ref. 2). The relationships are summarized as follows.

n-Year Flood	Frequency Adjustment in percent of SPF
SPF	100
100	45
50	32
25	21
10	12

1. Without-Project Conditions. In determining without-project 100-year values, the corresponding frequency adjustment (.45) was applied to the SPF breakout hydrograph. Using the reduced hydrographs, 100-year breakout flow widths were estimated and average overland flow velocities were determined. See table 1-5 for the 100-year routing parameters. The local 100-year flow was determined using the same (.45) reduction factor in addition to a flow width reduction factor. The latter adjustment reflects the reduced overflow width of the 100-year breakout flow. The 100-year discharges listed in table 1-2, are again, the combined breakout and local discharges.

Flow widths for the 50- and 25-year frequencies were determined by a hydraulic overflow analysis in an iterative process with the hydrology determination. To obtain the discharges of these events, frequency and flow width reductions were made to the 100-year breakout and local flood hydrographs. Thus, no routing was performed for these lower magnitude events. The peak discharge values are listed in table 1-2.

2. With-Project Conditions. The 100-, 50-, and 25-year with-project condition does not include breakout flow across the Arizona Canal. Thus, the total n-year hydrograph at each frontal flow line simply consisted of the SPF without-project local flow hydrograph proportioned according to the aforementioned frequency adjustments. Therefore, the overflow boundary for these events is the same as the study area boundary which was used to determine the SPF without-project runoff hydrograph.

The preceding method of determining discharge-frequency values maintains the same discharge-frequency relationships that are presented in paragraph IV B above. These discharge-frequency relationships are consistent with all previous discharge-frequency determinations for Phoenix and vicinity studies.

C. COMPARISON WITH THE JUNE 1972 FLOOD

On June 22, 1972, a thunderstorm occurred over northeastern Phoenix that caused extensive flooding on Cudia City Wash south of the Arizona Canal when the Canal was overtopped at 32nd and 40th Streets. The Arizona Canal also broke at other locations between Cudia City Wash and Dreamy Draw, but the inundation was small relative to that caused by Cudia City Wash.

The U.S. Geological Survey 1972 Surface Water Records (Ref. 5) 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, table 1-1, 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 2.6 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.)

In this present analysis, the standard project storm was progressively centered over the total contributing drainage area of Reach 4 study area, from the Phoenix Mountains to the Casa Grande Freeway. The synthesized 50-year flood was produced by the entire area upstream of a contributing concentration point. Additional breaks over the Arizona Canal (pl.1-2) caused more widespread flooding below the Arizona Canal than in the 1972 event. The damages from the 1972 flood, adjusted to 1986 conditions, were used to calibrate those obtained in this study from areas A and B for the 50-year flood.

D. HISTORICAL FLOODS OVERTOPPING REACH 4

A search of historical flooding accounts indicates overtopping of the Arizona Canal in Reach 4 of the ACDC at least 4 times since 1939. These events were in September 1939, August 1943, June 1972 and most recently in August 1986. In 1959 and 1978 floodwater entered the canal but did not overlfow in reach 4.

As a result of the August 1986 flood, the Flood Control District of Maricopa County published a "Report of Flooding near 32nd Street and the Arizona Canal, August 28-29, 1986." It gave a detailed description of the event and reported flood damages of $109,858 downstream from the Arizona Canal. They also reported an estimated peak discharge of 1400 ft^3/s on Cudia City Wash at 40th Street and Stanford Drive based on high water marks. Discharge-frequency relationships, developed by the Corps, estimated that the August flood was approximately a 10-year flood.

Based on the above historical information the economic analysis presented in this report used the 10-year flood as the point of zero damages on the discharge-damage curve for without-project conditions.

V. ACDC DESIGN FLOWS

In the event that Reach 4 is not constructed, the design capacities of Reach 3 would change while the size of reaches one and two would not change since construction is eminent. Project design flows without Reach 4 were computed with the assumption that no flow east of Dreamy Draw would contribute to Reaches 1-3 of the ACDC.

Table 1-6 presents design flows for the ACDC should Reach 4 not be built.

VI. RESULTS

The SPF and n-year flows, with and without ACDC Reach 4 for future conditions, are presented in tables 1-1 and 1-2. Flows were not reduced above the storm drain tunnels at the Casa Grande Freeway, because a storm drain plan has not yet been developed by local interests.

Table 1-1. Peak Discharges for Economic Analsysis
Future Conditions With Project.

CP	Location	Drainage Area[1] (mi^2)	Storm Centering (mi^2)	Flow for Each Frequency in ft^3/s SPF	100-yr	50-yr	25-yr
Subarea A							
101	u/s from ACDC	4.9	9.9	14,000	6,200	4,400	2,900
101	d/s from ACDC	4.9	9.9	6,900	0	0	0
A1		5.9	12.4	4,300	400	280	190
A2		7.3	15.9	3,500	740	560	370
A3		8.3	18.9	2,800	1,000	730	480
A4	at Grand Canal	9.3	22.4	2,700	1,200	880	580
Subarea B							
102	u/s from ACDC	1.4	9.9	4,500	2,000	1,400	940
102	d/s from ACDC	1.4	9.9	3,400	0	0	0
103A	u/s from ACDC	0.3	9.9	800	350	250	170
103A	d/s from ACDC	0.3	9.9	300	0	0	0
B1		2.2	12.4	2,700	250	180	120
B2		2.8	15.9	2,300	480	340	230
B3		3.4	18.9	2,100	650	460	300
B4	at Grand Canal	4.0	22.4	1,900	780	560	360
Subarea C							
103B	u/s from ACDC	1.0	9.9	1,300	570	410	270
103B	d/s from ACDC	1.0	9.9	1,300	0	0	0
C1		1.4	12.4	1,300	170	120	80
C2		2.0	15.9	1,500	380	270	180
C3		2.5	18.9	1,600	550	390	260
C4		3.1	22.4	1,700	720	510	340
Subarea D							
104	u/s from ACDC	1.1	9.9	3,500	1,600	1,100	730
104	d/s from ACDC	1.1	9.9	2,900	0	0	0
105	u/s from ACDC	1.2	9.9	3,800	1,700	1,200	800
105	d/s from ACDC	1.2	9.9	3,400	0	0	0
D1		2.9	12.4	3,600	280	200	130
D2		3.8	15.9	3,200	610	430	280
D3		4.7	18.9	2,900	870	620	410
D4	at Grand Canal	6.0	22.4	2,900	1,200	870	570
AD5		27.3	----	8,300	3,700	2,700	1,700
AD6		(2)	----	1,400	580	420	270

Note:

See plate 1-2 for location of CPs.
1. Includes area above the Arizona Canal.
2. Discharges based on ratio of X-Section length at CP AD6 to AD5.

Table 1-2. Peak Discharges for Economic Analysis
Future Conditions Without Project.

CP	Location	Drainage Area[1] (mi^2)	Storm Centering (mi^2)	Flow for Each Frequency in ft^3/s SPF	100-yr	50-yr	25-yr
Subarea A							
101	u/s from Ariz. Canal	4.9	9.9	14,000	6,200	4,400	2,900
101	d/s from Ariz. Canal	4.9	9.9	14,000	6,200	4,400	2,900
A1		5.9	12.4	13,000	5,000	3,500	2,300
A2		7.3	15.9	11,000	4,300	3,000	2,000
A3		8.3	18.9	9,600	3,800	2,700	1,800
A4	at Grand Canal	9.3	22.4	8,600	3,400	2,400	1,500
Subarea B							
102	u/s from Ariz. Canal	1.4	9.9	4,500	2,000	1,400	940
102	d/s from Ariz. Canal	1.4	9.9	4,500	2,000	1,400	940
103A	u/s from Ariz. Canal	0.3	9.9	800	350	250	170
103A	d/s from Ariz. Canal	0.3	9.9	800	350	250	170
B1		2.2	12.4	4,400	1,900	1,300	870
B2		2.8	15.9	3,800	1,600	1,100	730
B3		3.4	18.9	3,400	1,600	1,100	730
B4	at Grand Canal	4.0	22.4	3,000	1,400	1,000	650
Subarea C							
103B	u/s from Ariz. Canal	1.0	9.9	1,300	570	410	270
103B	d/s from Ariz. Canal	1.0	9.9	1,300	570	410	270
C1		1.4	12.4	1,400	470	310	200
C2		2.0	15.9	1,700	600	420	270
C3		2.5	18.9	1,800	780	550	360
C4		3.1	22.4	1,800	900	640	420
Subarea D							
104	u/s from Ariz. Canal	1.1	9.9	3,500	1,600	1,100	730
104	d/s from Ariz. Canal	1.1	9.9	3,500	1,600	1,100	730
105	u/s from Ariz. Canal	1.2	9.9	3,800	1,700	1,200	800
105	d/s from Ariz. Canal	1.2	9.9	3,800	1,700	1,200	800
D1		2.9	12.6	6,100	2,600	1,800	1,200
D2		3.8	15.9	5,500	2,200	1,500	1,000
D3		4.7	18.9	5,000	1,900	1,400	870
D4	at Grand Canal	6.0	22.4	4,700	1,800	1,300	800
AD5		27.3	----	16,000	6,600	2,700	1,700
AD6		(2)	----	2,500	1,200	390	150

Note:

See plate 1-2 for location of CPs.
1. Includes area upstream from the Arizona Canal.
2. Discharges based on ratio of X-Section length at CP AD6 to AD5.

Table 1-3. Basin Characteristics Above The Arizona Canal.

Subarea	Drainage Area (mi^2)	L (mi)	Lca (mi)	Slope (ft/mi)	Impervious Cover (%)	Basin n-Value
1	4.91	3.95	1.87	222	40	0.025
2	1.38	1.94	0.97	227	40	0.028
3	Biltmore Estates Subareas					
4	1.10	2.20	1.14	390	35	0.030
5	1.18	2.24	0.91	390	35	0.030
			Biltmore Estates Subareas			
1	0.230	1.10	0.47	382	20	0.035
2	0.110	0.57	0.30	368	25	0.035
3	0.045	0.32	0.16	125	35	0.030
4	0.012	0.26	0.12	115	40	0.030
5	0.008	0.15	0.07	187	40	0.030
6	0.032	0.32	0.20	178	25	0.035
7	0.048	0.45	0.20	209	20	0.040
8	0.031	0.40	0.15	163	40	0.030
9,10	0.053	0.49	0.23	173	20	0.035
11	0.031	0.30	0.15	200	30	0.030
12	0.004	0.06	0.03	175	20	0.025
13	0.170	0.91	0.49	154	45	0.025
14	0.243	0.76	0.28	210	30	0.030
15	0.178	0.72	0.30	180	35	0.030
A	0.009	0.133	0.076	211	70	0.030
B	0.021	0.178	0.057	153	70	0.030
C	0.011	0.152	0.076	158	80	0.025
D	0.027	0.189	0.133	127	80	0.025
E	0.034	0.246	0.095	97	60	0.030
F	0.017	0.227	0.057	114	70	0.030
G	0.008	0.125	0.038	96	70	0.030
			Subareas Below ACDC			
A1-A4	4.30	4.11	2.06	28	50	0.15
B1-B4	2.31	3.56	1.78	33	30	0.15
C1-C4	2.13	3.28	1.64	35	50	0.15
AC5 +Above*	12.71	5.86	2.93	25	50	0.15
D1-D4	3.65	3.35	1.68	35	50	0.15
D1-D5	4.72	4.58	2.29	31	50	0.15

*Includes A1-A4, B1-B4, and C1-C4.

Table 1-4. Elevation-Storage-Outflow Data Biltmore Estates Detention Structures.

Structure No.	Elevation (ft)	Outflow (ft^3/s)	Storage (ac-ft)
1	1,292.5	0.0	0.00
	1,294	8.0	0.71
	1,296	21.0	1.59
	1,300	28.0	3.89
	1,304	34.0	7.12
	1,307	295.0	10.50
	1,308	1,250.0	11.90
2	1,286	0.0	0.00
	1,288	4.3	0.13
	1,290	8.3	0.96
	1,294	13.0	4.63
	1,296	14.8	7.13
	1,298	16.4	10.30
	1,300	501.0	14.58
D	1,256	0.0	0.00
	1,257	3.0	0.40
	1,258	9.0	0.90
	1,260	16.0	2.00
	1,260	35.0	2.24
	1,261	76.0	2.48
	1,261	150.0	2.80
8	1,246	0.0	0.00
	1,248	11.0	3.11
	1,250	74.0	8.76
	1,251	150.0	11.46
10	1,255	0.0	0.00
	1,256	3.7	0.62
	1,257	7.0	1.31
	1,258	8.0	2.08
	1,259	9.6	2.97
	1,260	20.0	3.87
	1,261	27.0	4.97
11	1,272	0.0	0.0
	1,273	0.01	1.03
	1,274	65.0	2.19
	1,275	338.0	3.54
	1,276	1,450.0	5.08

Note: See plate 1-2 for location. Structure 3 has no spillway, so no routing was performed.

Table 1-5. Pertinent Routing Data
(of Breakout Flow)

Reach[a]	Length (ft)	X[b]	SPF(W/P) K[b] (hr)	SPF(W/P) NRCHS #	SPF (W/O) K[b] (hr)	SPF (W/O) NRCHS #	100-Yr (W/O) K[b] (hr)	100-Yr (W/O) NRCHS #
SUBAREA A								
101 R A1	6440	0	0.222	2	0.219	2	0.177	2
A1 R A2	6440	0	0.235	2	0.225	2	0.174	2
A2 R A3	4200	0	0.335	1	0.309	1	0.244	1
A3 R A4	4650	0	0.205	2	0.363	1	0.279	1
A4 R AD5	9240	0	0.212	4	0.259	3	0.255	1
SUBAREA B								
102 R B1	3200	0	0.235	1	0.259	1	0.222	1
103A R B1	3200	0	0.221	2	0.214	2	0.371	1
B1 R B2	5500	0	0.219	2	0.248	2	0.367	1
B2 R B3	4500	0	0.211	2	0.239	2	0.353	1
B3 R B4	5600	0	0.233	2	0.261	2	0.389	1
B4 R AD5	9240	0	0.242	3	0.268	3	0.205	3
SUBAREA C								
103B R C1	2380	0	0.202	1	0.230	1	0.199	1
C1 R C2	4000	0	0.339	1	0.279	2	0.333	1
C2 R C3	4750	0	0.224	2	0.230	3	0.218	2
C3 R C4	6180	0	0.286	2	0.213	4	0.279	2
C4 R AD5	9240	0	0.232	3	0.217	4	0.224	3
SUBAREA D								
104 R D1	3800	0	0.255	1	0.293	1	0.229	1
105 R D1	3800	0	0.286	1	0.334	1	0.290	1
D1 R D2	4280	0	0.339	1	0.357	1	0.275	1
D2 R D3	4000	0	0.341	1	0.382	1	0.284	1
D3 R D4	5600	0	0.249	2	0.268	2	0.203	2
D4 R AD5	9240	0	0.246	3	0.267	3	0.208	3

Note: Refer to plate 1-3 for schematic of routing. See Ref. 4 for the routing parameters used in the SPF with project analysis above the Arizona Canal.

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

b. Muskingum coefficients.

Table 1-6. ACDC Flows

CP[a]	Location	Reach	100-Yr Future Condition Peak Flows: With Reach 4 mi^2	With Reach 4 ft^3/s	Without Reach 4 mi^2	Without Reach 4 ft^3/s[b]
101	Cudia City Wash	4	4.9	6700	--	0
102	East of 32nd Street	4	6.3	7900	--	0
103	At Sahuaro Drive	4	7.7	8300	--	0
104	At Ocotillo Road	4	8.8	8700	--	0
105	West of 16th Street	4	9.9	9000	--	0
1071	West of Dreamy Draw	3	----	----	3.68	1400
1072	At Northern Avenue	3	11.8	10,000	4.52	2600
108	West of 10th Street	3	14.5	13,000	7.26	6400
1016U	East of Cave Creek	3	19.9	14,000	12.5	10,000
1016D	West of Cave Creek	2	61.1	25,000	53.9	21,000
1018	At 51st Avenue	2	70.3	27,000	63.1	23,000
1019	East of Skunk Creek	1	85.4	29,000	78.2	25,000

a. See Ref. 4 plate 43 for location of CPs.

b. These flows were used in the hydraulic backwater analyses. The design capacities for Reaches 1 and 2 were not changed by this report regardless of the status of Reach 4.

VII. REFERENCES

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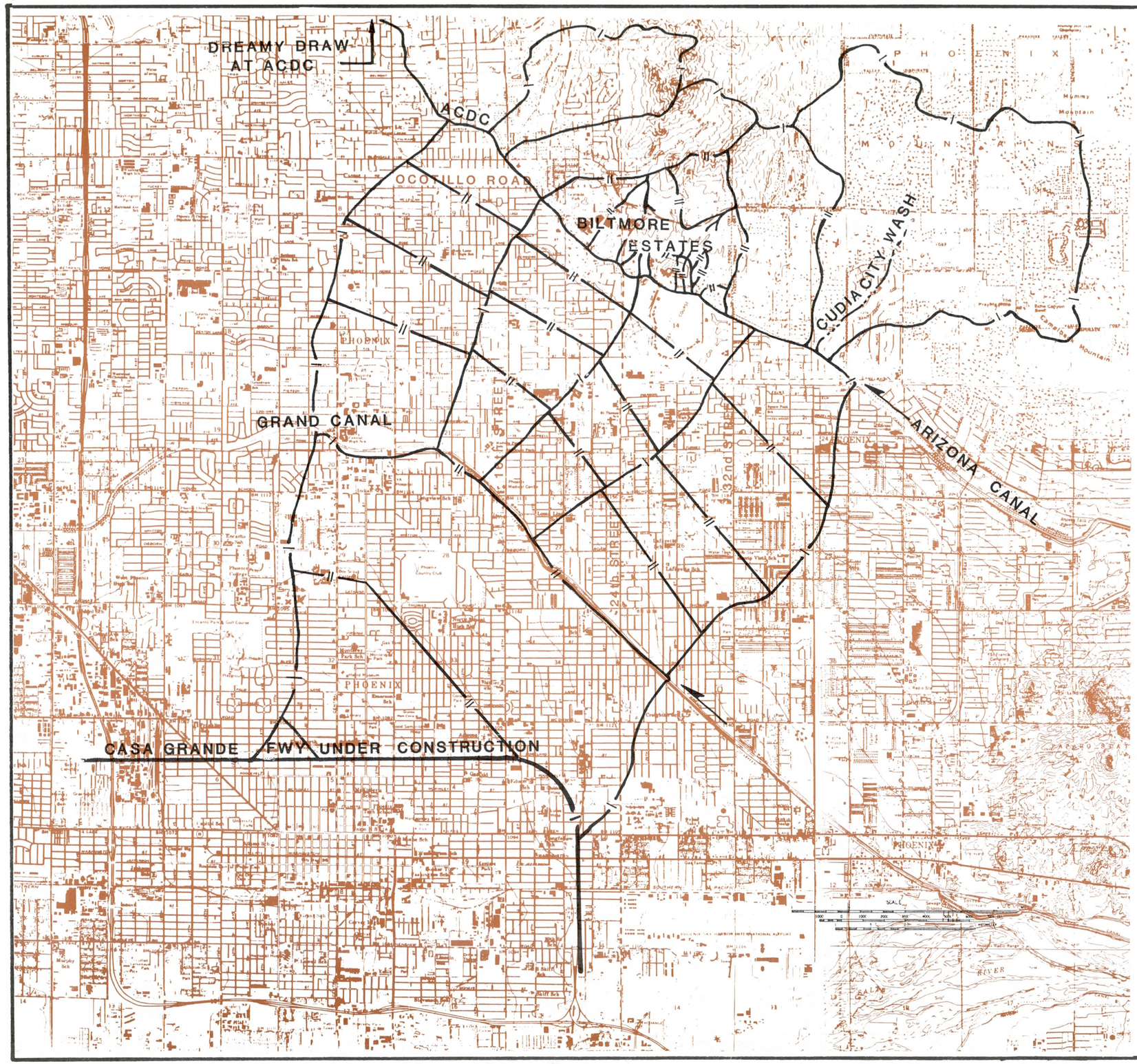
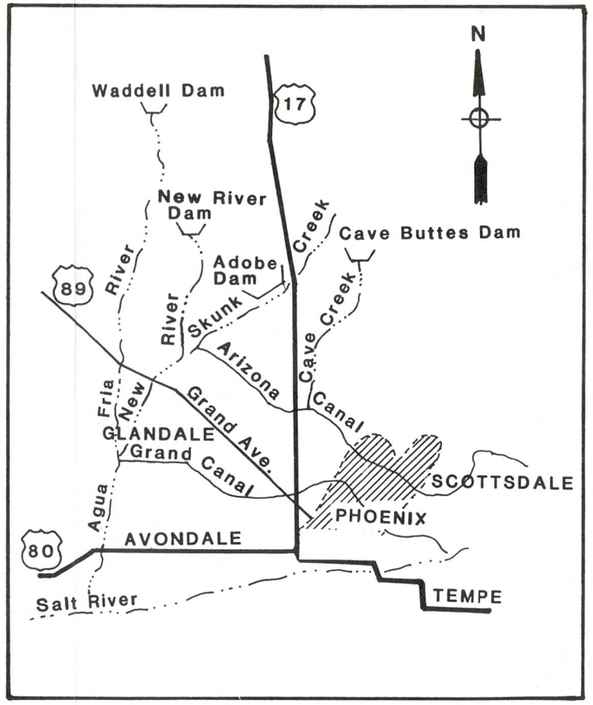
5. _____. United States Geologic Survey. Water Resources Data for Arizona, Part 1. Surface Water Records, 1972.





LEGEND

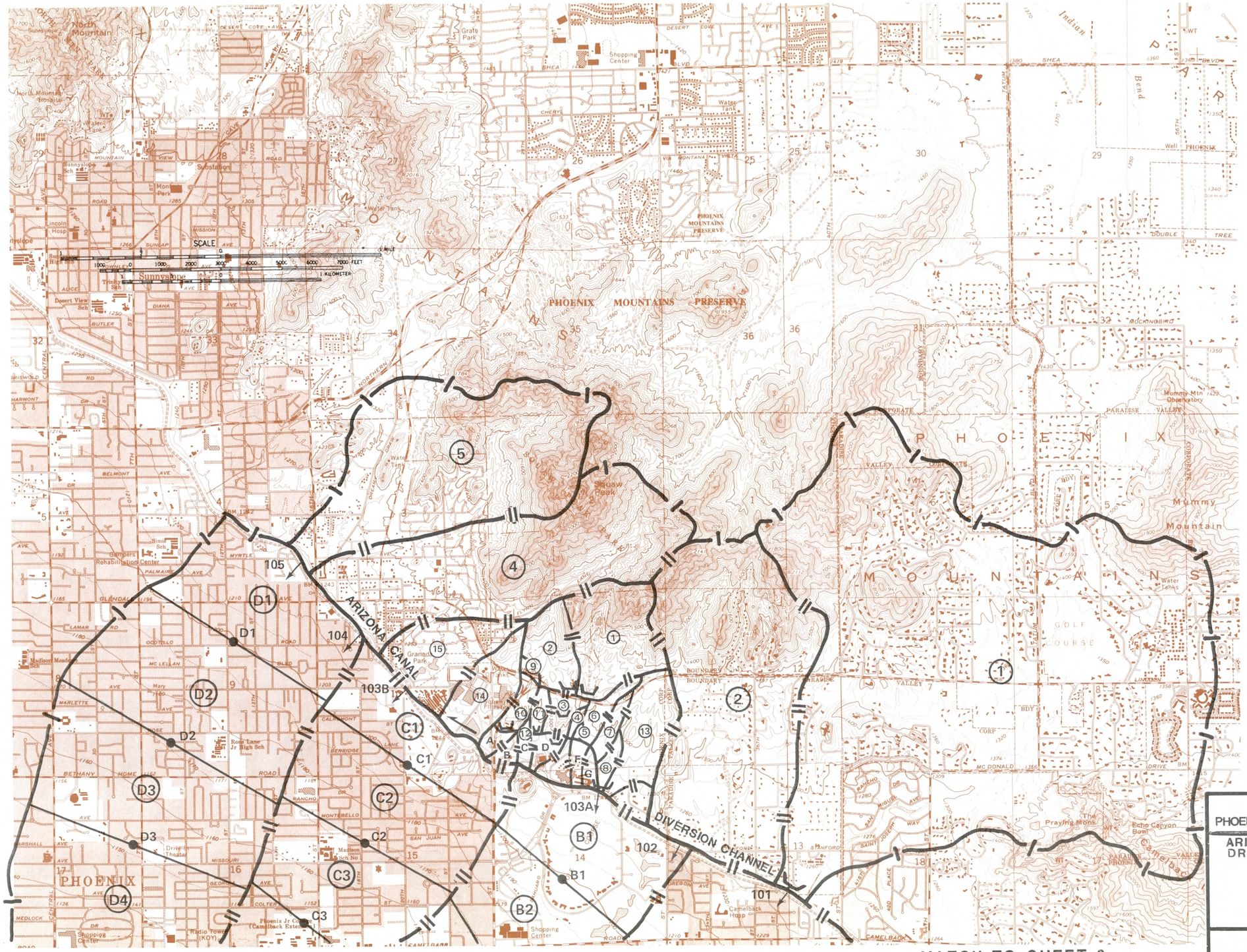
- DRAINAGE AREA BOUNDARY
- SUBAREA BOUNDARY
- RIVER OR CREEK
- CANAL
- INTERSTATE HIGHWAY
- STUDY AREA

GILA RIVER BASIN
PHOENIX, AZ AND VICINITY (INCLUDING NEW RIVER)

ARIZONA CANAL DIVERSION CHANNEL
DREAMY DRAW TO 40TH STREET

STUDY AREA LOCATION

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



N

LEGEND

- DRAINAGE AREA BOUNDARY
- SUBAREA BOUNDARY
- CP
- BREAKOUT LOCATION
- DEBRIS BASIN
- (1) SUBAREA NUMBER
- (1) BILTMORE ESTATES SUBAREA

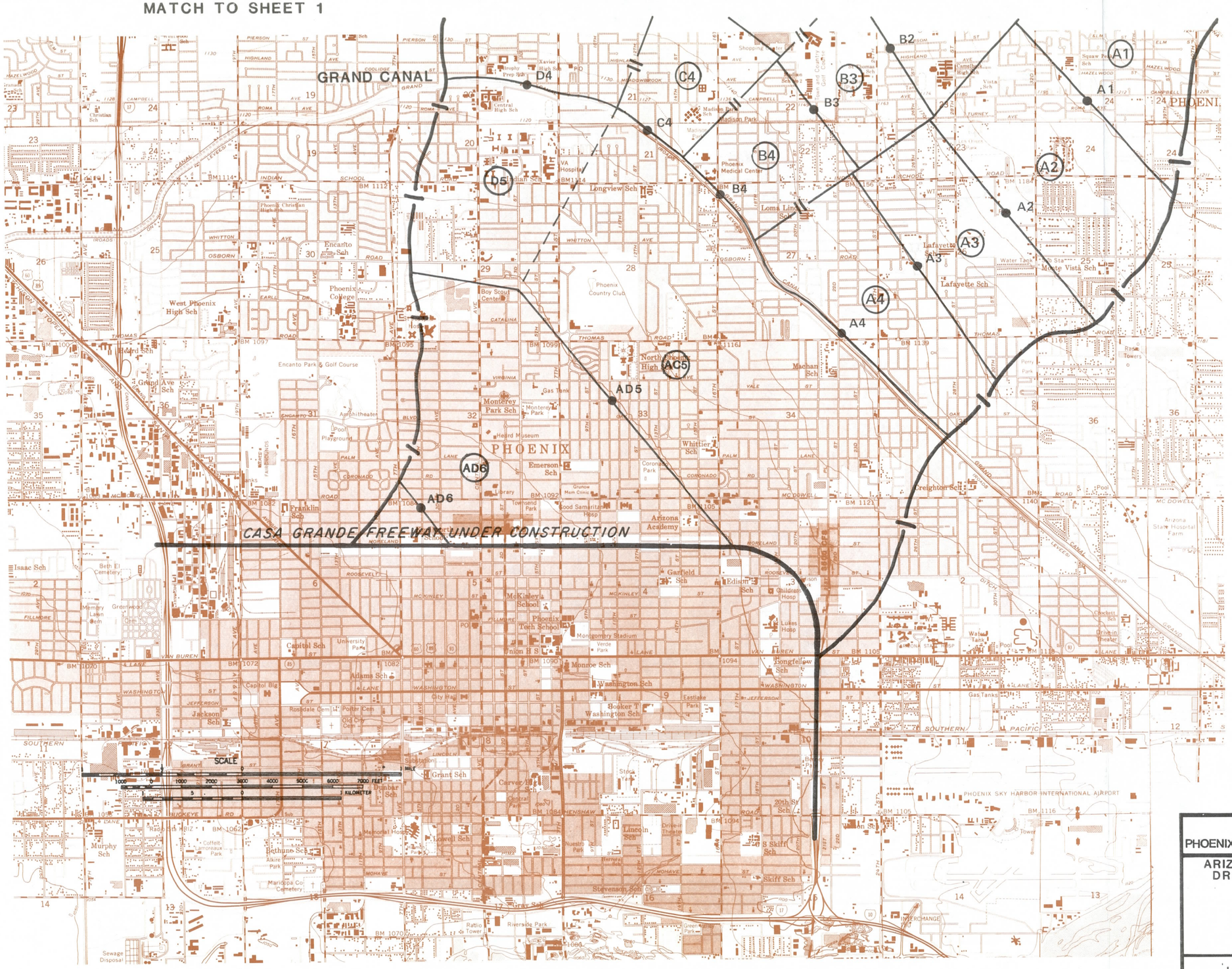
GILA RIVER BASIN
PHOENIX, AZ AND VICINITY(INCLUDING NEW RIVER)

ARIZONA CANAL DIVERSION CHANNEL
DREAMY DRAW TO 40TH STREET

DRAINAGE AREA BOUNDARIES

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

MATCH TO SHEET 1



N

LEGEND

DRAINAGE AREA BOUNDARY

SUBAREA BOUNDARY

CP

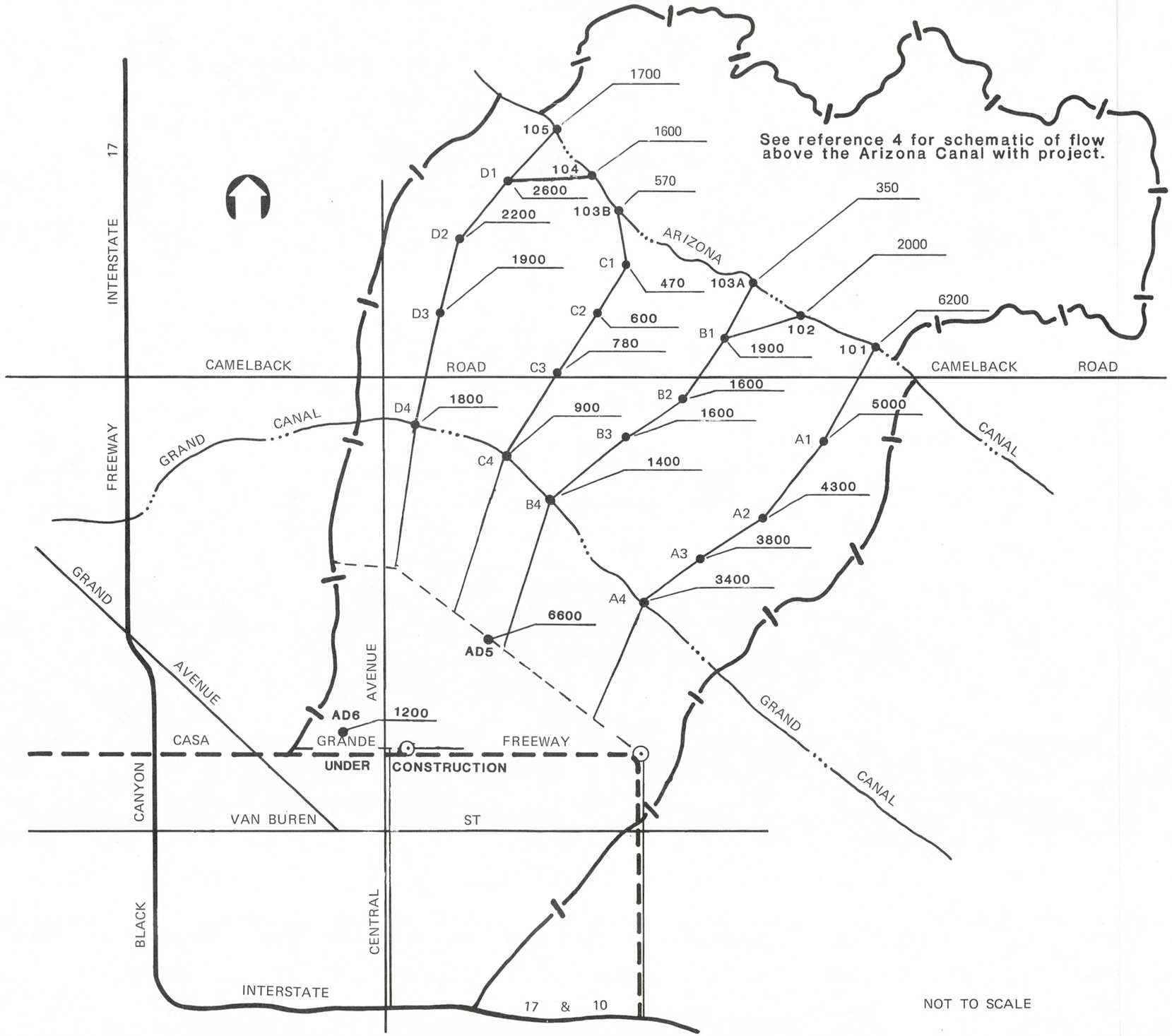
SUBAREA NUMBER

GILA RIVER BASIN
PHOENIX, AZ AND VICINITY (INCLUDING NEW RIVER)

ARIZONA CANAL DIVERSION CHANNEL
DREAMY DRAW TO 40TH STREET

DRAINAGE AREA BOUNDARIES

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



GILA RIVER BASIN
PHOENIX, AZ AND VICINITY (INCLUDING NEW RIVER)

ARIZONA CANAL DIVERSION CHANNEL
DREAMY DRAW TO 40TH STREET

OVERFLOW

SCHEMATIC DIAGRAM

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

Appendix 2
Hydraulic Analysis

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2-1 Dreamy Draw to Cudia City Wash, Areas Subject to Flooding, without the project.

2-2 Dreamy Draw to Cudia City Wash, Areas Subject to Flooding, with the project.

HYDRAULIC ANALYSIS

I. INTRODUCTION

The purpose of this section is to present the investigations, assumptions, methodology, and results used in the determination of overflow areas below the Arizona Canal between Dreamy Draw and 40th Street. Flood boundaries and average flood depths were estimated for the 25-year flood, 50-year flood, 100-year flood, and Standard Project Flood (SPF), for two conditions each: with and without the Reach 4 section of the Arizona Canal Diversion Channel (ACDC). The flood data in this study was generated to evaluate the economic viability of Reach 4 and also to indicate residual flooding that will remain after construction of the project.

II. GENERAL PROCEDURE

A simplified, trial-and-error procedure was utilized to determine overflow boundaries and average flood depths. Briefly, the procedure involved the following major steps: (1) determine initial flood boundaries, effective roughness coefficient, and depth-discharge-velocity relationships; (2) utilize the estimated discharge-velocity relationships in hydrologic flood routing computations to determine a trial set of discharges; (3) use the trial discharges and flood boundaries to adjust the effective roughness coefficient to generate depths that would reconstitute actual damages from the June 1972 flood; (4) refine the depth-discharge-velocity relations and determine a revised set of discharges with another flood routing iteration; (5) repeat the procedure until the difference between the previous and last effective roughness coefficients is insignificant.

III. STUDY CONDITIONS

The study was based on future hydrologic conditions, for which the water-shed was assumed fully developed for two cases: with and without Reach 4 of the ACDC. Full development includes the two new 21-foot diameter storm drain tunnels being constructed by the Arizona Department of Transportation (ADOT), plus a collector system for urban drainage to be constructed by the City of Phoenix. The tunnels will flow north to south and will extend from north of the Casa Grande Freeway to the Salt River. Each tunnel will have a capacity of 3500 ft^3/s. The study was terminated at the Casa Grande Freeway, since further downstream the difference between without-project and with project flows would be small.

IV. PAST STUDIES AND FIELD RECONNAISSANCE

The locations where overtopping of the Arizona Canal would occur were established from research of previous reports and from field reconnaissance. During the field work, spillways were located, heights

of canal banks were measured in important areas, locations of runoff concentration points were verified, and areas of exceptionally high and/or wider embankments were noted.

A field inspection of the Grand Canal was also made. Low spots, canal facilities, and obstructions were noted. The banks are uniformally graded along the top without any significant depression or low areas to concentrate flows overtopping the banks. In no case were the raised banks observed in the field to be high enough to cause significant ponding. Hence it was concluded that runoff from a major storm would travel directly across the Grand Canal without any major interference, ponding, or redirection.

Other agencies were also consulted during this field investigation. These agencies included the Flood Control District of Maricopa County, the Salt River Project in Phoenix, and ADOT.

V. ASSUMPTIONS AND PROCEDURES

It was judged that the Arizona Canal would overflow at the four spillways in the study reach and also at three other locations along the bank that were breached during the June 22, 1972 flood. In order to determine flood discharges below the canal, trial flood boundaries for the without-project SPF were drawn by judgment on United States Geological Survey (USGS) quadrangle maps at each end of the study reach, extending from the Arizona Canal downstream to the Casa Grande Freeway. The floodplain and drainage area thus defined was subdivided into zones corresponding to concentration points above the canal. A total of six cross sections was located across the entire floodplain. A set of trial relations of discharge versus depth and velocity were established for each zone separately and were used to route estimated flood hydrographs below the Arizona Canal and determine peak flood discharges. The hydrologic analysis considered runoff from both above and below the Arizona Canal.

The floodplain was further subdivided by drawing three additional sets of trial overflow boundaries, representing the 100-year, 50-year, and 25-year floods, for without-project conditions. Since the floodplain is flat and provides no significant confinement to the flow, the trial flood boundaries were set by judgment and later checked to insure a reasonable relationship between width and depth of the overflow. The location of these boundaries was based on: (1) assuming the SPF boundaries at the canal breaks would spread outwardly from the general direction of flow at an angle of about 45° on each side; (2) assuming progressively smaller angles of spread for the lesser floods, and (3) following the general ground slope of the floodplain. The influence of the topography is pronounced at the lower end of the study reach, where changes in direction of the general ground slope force the watershed boundaries and hence the flood boundaries to converge. The trial flood boundaries were set to insure a reasonable increase in both areal extent of inundation as well as average flood depth with an increase in the magnitude of the flood.

The without-project SPF flood boundary was also used for the with-project SPF, 100-year, 50-year, and 25-year floods as well. A single boundary was selected because it conforms with the sheetflow characteristic of the flat urban floodplain with respect to both the manner in which runoff is generated and also the manner in which the resulting overland flow behaves.

For both with-and without-project conditions, the flood boundaries delineate zones of inundation only within the limits of Reach 4. Residual runoff will occur concurrently from adjacent watersheds and will cause inundation in the floodplain beyond the flood boundaries resulting from this study. Further discussion is provided in Appendix 1.

Average flood depths were computed along each of the cross sections using Manning's equation simplified for wide, shallow flow. The average flood depths were computed relative to the top of curb.

VI. CALIBRATION

An important task in conducting an overflow study is to calibrate the procedures to reconstitute known data observed from an actual, historical flood. In this particular study, the results are extremely sensitive to analytical estimates of the effective roughness of the flood-plain. Such estimates involve considerable engineering judgment.

Unfortunately, sufficient specific information on discharges, depths, and areal extent of historical floods with which to conduct a hydraulic calibration is not available for the study area. However, sufficient data on flood damages is available to conduct an economic calibration of the model. Specifically, sufficient damage data is available for the June 22, 1972 flood, which was determined to represent approximately a 50-year event in the Cudia City Wash portion of the Reach 4 overflow area.

An additional piece of information with which to calibrate is the lack of significant flood damage to contents without a break in the Arizona Canal in the past 25 years or so since the floodplain has been fully urbanized. As a result, at least the with-project 25-year depths (no overflow from the Arizona Canal) should be essentially non-damaging to contents. For this study area, the average maximum non-damaging depth for contents has been estimated to be 0.5 feet.

To begin the calibration, initial overflow data was required. Trial flood boundaries were set as discussed above. An effective roughness coefficient was estimated to allow for computation of the average flood depth across the entire across section, including the portion obstructed by buildings. For the initial value of the effective roughness coefficient, conveyance in the floodplain beyond the street right-of-way line was ignored. Since the floodplain is fully urbanized with relatively little variation in the type and density of development, a

single value of roughness was used for the entire floodplain. An analytically estimated effective Manning's "n" value of 0.3 was used as a first trial value.

The initial set of flood discharges were determined and the initial depths were computed. The June 22, 1972 flood was determined to represent approximately a 50-year flood from Cudia City Wash on Cudia City Wash above the Arizona Canal. It was found that reducing the initial 50-year depths with- and without-project by 20 percent, but with no adjustment to the initial trial flood boundaries, would adequately reproduce the observed 1972 flood damages adjusted to present-day values. Further discussion of the economic calibration is provided in Appendix 3.

Consequently, the initial 50 year depths were reduced by 20 percent, and a calibrated "n" value of 0.25 was determined. Using this revised "n" value, a single curve of unit discharge versus average velocity representative of the entire floodplain was developed. This curve was used in the flood routing procedures to revise the computation of the discharges.

In the flood routing computations, it is important to use a value of average flow velocity that results in a reasonably accurate estimate of the travel time of the flood hydrograph through a given reach. In the subject floodplain most of the flow will be conveyed in the streets up to the right-of-way line, and the remainder in the open areas around buildings. Consequently a discharge-weighted average value of velocity was used. The value was computed from the equation:

$$V = \frac{Q_S V_S + Q_B V_B}{Q_S + Q_B}$$

where V is velocity

Q is discharge
S denotes flow within streets
B denotes flow beyond streets

Values of discharges and velocities within and beyond the streets were determined from analytical estimates of effective roughness coefficients using different percentages of obstruction on the floodplain beyond the street right-of-way.

The revised discharges were used with the initial boundaries and revised 50-year depths to recalibrate the model and check the effective "n" value. The resulting "n" value was 0.27, which was sufficiently close to the value of 0.25 found in the previous trial. Therefore, sufficient calibration of the model was achieved, and sufficiently close agreement was attained between the depths, discharges, and velocities resulting from the trial-and-error procedures involving the coordinated hydrologic-hydraulic analysis.

Final average flood depths were computed using the revised discharges and an "n" value of 0.25. The reasonableness of the depths was checked against criteria used in flood insurance studies. This

criteria indicates that for unconfined overflow on broad alluvial floodplains such as the area below Reach 4, the average flood depths should not exceed 3 feet. The computed average flood depths satisfy this criteria for all floods at all locations, except immediately below the Arizona Canal. At these locations, the SPF without-project depths are greater than 3 feet because the overflow is expanding from relatively confined paths controlled by the length of the canal spillways and the length of breaches in the canal bank. Since the average flood depths were found to be satisfactory, no adjustment of the flood boundaries was required.

The reasonableness of the computed overland flow depths was also checked against the depths observed in the June 22, 1972 flood. As indicated in the Corps of Engineers report entitled Report on Flood of 22 June 1972, outside water depths ranged from 0.4 to 4 feet in the zone of the Reach 4 floodplain inundated by that flood. In that same zone (basically subareas A and B), the 50-year without-project depths indicated on Plate 2-1 range from 0.6 to 2.6 feet above the top of curb. The referenced report states that ". . . through the Biltmore Shopping Center at 24th Street and Camelback Road . . . outside water depths in the shopping area ranged from 6 inches to 2 feet." The computed average depth for the comparable 50-year flood at this location is 0.6 feet or 7 inches. The referenced report also states that "The Camelback High School at 28th Street and Campbell Avenue was damaged when floodwaters 5 inches to 1-1/2 feet deep entered the buildings . . ." The computed average depth for the 50-year flood at this location is about 0.8 feet, or 10 inches. Thus, the computed depths are reasonable compared to the depths observed during an historical flood with a comparable frequency. Finally, the with-project 25 year depths are all less than 0.5 feet and thus satisfy the calibration criteria discussed above.

The discharge-velocity rating curve finally adopted reflects 87 percent of the flood discharge conveyed within the street right-of-way and 13 percent beyond it in the open areas around the structures on the floodplain.

VII. RESULTS

The resulting average flood depths and flood boundaries are shown on Plates 2-1 and 2-2. On Plate 2-1, the without project floodplain between the SPF boundary and the 25-year, 50-year, and 100-year boundaries each will be affected by local runoff generated below the Arizona Canal. Depths for the respective floods in these areas are the same as indicated for with-project conditions shown on Plate 2-2.

As discussed above, the results are very sensitive to the procedures and judgments used. Analytical estimates of effective roughness coefficients are especially sensitive to judgments made on the degree of obstruction on the floodplain. It is possible that a study using different procedures and judgments would yield significantly different

flood boundaries and flood depths. However, the results displayed are considered to be the most reasonable for the study area because they satisfactorily reconstitute data on historical flood damages.

The overflow maps were prepared for the sole purpose of estimating a single value of average annual flood damage for the entire floodplain affected by Reach 4 of the ACDC. The maps are an idealized representation of the inundation in the subject floodplain. Average flood depths were computed over very wide cross sections using a simplified hydraulic analysis. Actual flood depths at any particular location will vary somewhat from the average depths indicated on the maps because of local variations in topography. However, any error in the estimate of the flood damages from depths being greater or less than actual depths at any given location will be essentially cancelled out by the process of using average depths over large areas. Also, the with-project SPF flood boundaries reflect inundation only for the floodplain below Reach 4 of the ACDC. Inundation of the floodplain to the east and west will still occur from other sources of runoff but was not studied for this report.

FLOODING WITHOUT PROJECT

LEGEND

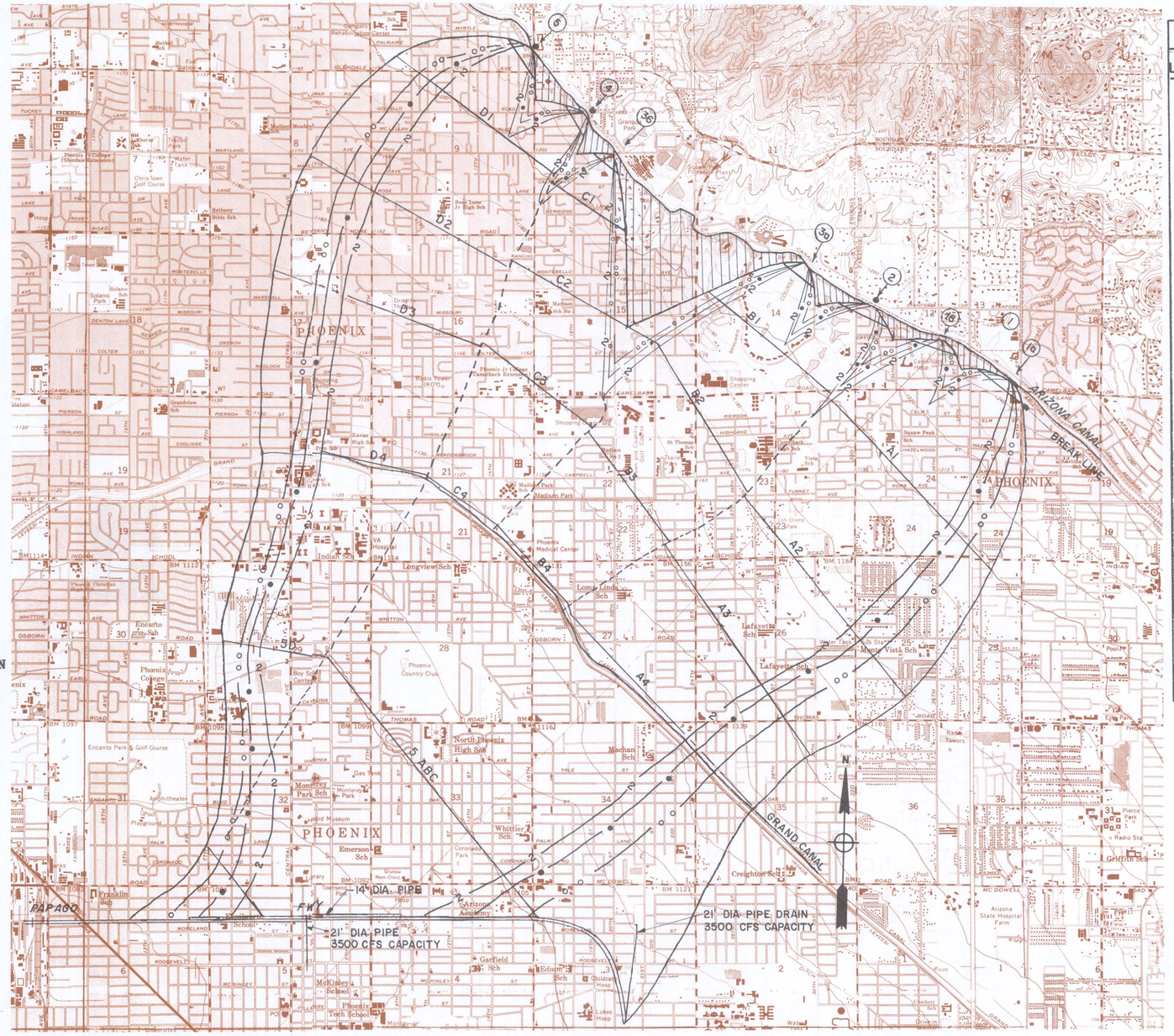
- SPF WITHOUT PROJECT
- 100-YEAR WITHOUT PROJECT
- 50-YEAR WITHOUT PROJECT
- 25-YEAR WITHOUT PROJECT

FLOODED AREAS AND FLOOD DEPTHS SHOWN ARE FOR ECONOMIC ANALYSIS ONLY. NO INFERENCE SHOULD BE DRAWN REGARDING WHICH PORTIONS OF THE AREA SHOWN MAY ACTUALLY FLOOD AT ANY GIVEN TIME. THIS MAP SHOULD NOT BE USED FOR ANY FLOODPLAIN REGULATION.

DEPTHS SHOWN ON THE TABLE AT THE FAR RIGHT ARE AVERAGE DEPTHS ACROSS THE ENTIRE CROSS SECTION SHOWN ON THE MAP. ACTUAL DEPTHS CAN VARY CONSIDERABLY ACROSS ANY CROSS SECTION.

THE CROSS-HATCHED AREAS AND AREAS OUTSIDE THE SPF LINES WERE NOT INCLUDED IN THE ANALYSIS BECAUSE THEY ARE NOT SUBJECT TO FLOODING FROM FLOWS CROSSING THE ARIZONA CANAL IN REACH 4. WITH PROJECT DEPTHS WILL APPLY IN THESE AREAS.

FOR ANY FREQUENCY FLOOD SMALLER THAN THE SPF, AVERAGE FLOODING DEPTHS BETWEEN THAT FLOOD'S LIMITS AND THE SPF LIMITS WOULD BE THE SAME AS WITH PROJECT FLOOD DEPTHS (SEE PLATE 2-2).



AVERAGE FLOOD DEPTH*				
LOCATION	SPF	100 YR	50 YR	25 YR
1a	3.4	2.5	2.3	2.2
1b	3.8	2.8	2.6	2.5
2	2.5	1.9	1.8	1.7
3a	.3	.2	.2	.2
3b	1.9	1.5	1.3	1.2
4	3.2	2.2	2.2	2.2
5	2.4	1.9	1.8	1.7
A1	1.9	1.3	1.1	.9
B1	.4	.2	.2	.2
C1	1.0	.5	.5	.4
D1	1.4	.9	.8	.7
A2	1.8	1.1	1.0	.8
B2	1.1	.7	.6	.4
C2	.9	.5	.4	.3
D2	1.2	.8	.7	.6
A3	1.6	1.0	.9	.8
B3	1.3	.8	.6	.5
C3	.9	.5	.4	.3
D3	1.2	.8	.7	.5
A4	1.5	1.0	.8	.7
B4	1.4	.9	.7	.6
C4	1.2	.8	.6	.5
D4	1.4	.9	.8	.6
5ABC } 5D	1.6	1.0	.7	.6
6	1.6	1.2	.7	.6

* Above top of curve

GILA RIVER, BASIN
PHOENIX, ARIZ. & VIC. (INC. NEW RIVER)

ARIZONA CANAL DIVERSION CHANNEL

DREAMY DRAW TO CUDIA CITY WASH
AREAS SUBJECT TO FLOODING
WITHOUT PROJECT

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FLOODING WITH PROJECT

LEGEND

FLOODED AREA WITH PROJECT

FLOODED AREAS AND FLOOD DEPTHS SHOWN ARE FOR ECONOMIC ANALYSIS ONLY. NO INFERENCE SHOULD BE DRAWN REGARDING WHICH PORTIONS OF THE AREA SHOWN MAY ACTUALLY FLOOD AT ANY GIVEN TIME. THIS MAP SHOULD NOT BE USED FOR ANY FLOODPLAIN REGULATION.

DEPTHS SHOWN ON THE TABLE AT THE FAR RIGHT ARE AVERAGE DEPTHS ACROSS THE ENTIRE CROSS SECTION SHOWN ON THE MAP. ACTUAL DEPTHS CAN VARY CONSIDERABLY ACROSS ANY CROSS SECTION.

THE CROSS-HATCHED AREAS AND AREAS OUTSIDE THE SPF LINES WERE NOT INCLUDED IN THE ANALYSIS BECAUSE THEY ARE NOT SUBJECT TO FLOWS CROSSING THE ARIZONA CANAL IN REACH 4.

AVERAGE FLOOD DEPTH*				
LOCATION	SPF	100 YR	50 YR	25 YR
1a	2.3	0	0	0
1b	2.5	0	0	0
2	2.1	0	0	0
3a	.2	0	0	0
3b	1.9	0	0	0
4	2.8	0	0	0
5	2.3	0	0	0
A1	1.1	.3	.2	.2
B1	.3	.1	.1	.0
C1	.9	.3	.2	.2
D1	1.0	.2	.2	.1
A2	.9	.4	.3	.2
B2	.8	.3	.2	.2
C2	.8	.3	.3	.2
D2	.9	.3	.3	.2
A3	.8	.4	.3	.3
B3	1.0	.4	.4	.3
C3	.8	.4	.3	.3
D3	.8	.4	.3	.3
A4	.7	.4	.4	.3
B4	1.0	.6	.5	.4
C4	1.1	.7	.5	.4
D4	1.0	.6	.5	.4
5ABC } 5D }	1.1	.7	.6	.4
6	1.1	.7	.5	.4

* Above top of curb

GILA RIVER, BASIN
PHOENIX, ARIZ. & VIC. (INC. NEW RIVER)

ARIZONA CANAL DIVERSION CHANNEL

DREAMY DRAW TO CUDIA CITY WASH
AREAS SUBJECT TO FLOODING
WITH PROJECT

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

Appendix 3
Economic Analysis

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I. PURPOSE

The purpose of this appendix is to present the economic analysis used to measure the beneficial contributions to National Economic Development (NED) associated with the flood hazard reduction features of Reach 4 of the Arizona Canal Diversion Channel (ACDC), a feature of the Phoenix, Arizona, and Vicinity (Including New River) Project. Two measures of economic efficiency were utilized in analyzing each of these alternatives: benefit-to-cost ratio and net benefits. The benefit-to-cost ratio indicates whether a given proposal analyzed at the rate of return established by the Water Resources Council (3-1/4 percent[1]) would return more in benefits than it would cost. The net benefits represent the difference between the benefits and the costs, analyzed at the Water Resources Council discount rate.

1. Projects authorized prior to December 1969, not under construction, with signed local assurances, use an applicable discount rate of 3-1/4 percent.

II. METHODOLOGY

Methodology used in the development of benefits in this report is in accordance with Economic and Environmental Principles and Guidelines for Water Related Land Resources Implementation Studies (March 10, 1983). Backup data is on file in the Los Angeles District.

Benefits herein presented for Reach 4 of the ACDC flood control improvements consist of inundation reduction benefits and savings in emergency costs.

Estimates of project costs and benefits were based on October 1986 price levels. The channel was assumed to be operative for 90 years. Sufficient allowance was made for annual operation and maintenance costs to ensure the long-range functioning of the project. Construction costs were converted into annual payments over the life of the project, using a discount rate of 3-1/4 percent. Annual operation and maintenance costs were then added to arrive at total annual charges.

The values derived for costs and benefits, which typically accrue at differing times, are made comparable by conversion to an equivalent time basis (using the appropriate discount rate and period of analysis). For this project, the applicable discount rate is 3-1/4 percent. The Phoenix and Vicinity comprehensive plan has a project life of 100 years for the entire system. The project base year was established as 1981 when the first completed feature, Cave Buttes Dam, became operational. The last year of project life is 2080. Reach 4 improvements are scheduled for completion in 1991. Reach 4 project life is, therefore, 90 years (1991 to 2080). To calculate benefit-cost ratios, all costs and benefits associated with Reach 4 are discounted from this period to 1981, then annualized over the 100-year economic life of the system.

III. COST AND BENEFIT SUMMARY

Detailed estimates of annual costs and benefits are presented in subsequent paragraphs. To facilitate the comparison of costs and benefits, however, they are summarized in the following subparagraphs.

A. The equivalent average annual inundation reduction benefits for Reach 4 of the ACDC are $2,129,000. These benefits result from damages prevented to structures and contents for facilities susceptible to flood losses. Annual benefits resulting from emergency cost savings total $125,000. Total annual project benefits are $2,254,000.

B. The total annual charges for construction of the project are $2,092,000. This results in a benefit to cost ratio of 1.08 to 1.

IV. PROJECT STUDY AREA

The project study area covers an area of about 20 square miles in the urban core of Phoenix. The Arizona Canal and the Casa Grande Freeway (currently under construction) define the north and south boundaries respectively, while the east and west limits are generally contained within the area between West 7th Street and 32nd Street. Field surveys were used to estimate present land use in the overflow areas studied in this report.

V. NO-ACTION ALTERNATIVE (BASE CASE)

The no action alternative represents the situation most likely to occur without a specific plan (without-project condition). This alternative was established as a base case for comparison with the authorized project plan.

The without-project condition presented in this report assumes no future public flood control facilities will be built by local interests in Reach 4.

VI. EVALUATION OF BENEFITS

INUNDATION REDUCTION

Primary benefits attributable to the project plan include inundation reduction or physical damages prevented. Inundation reduction benefits result from reducing flood losses that would occur in the flood plain in the absence of a flood control project. Inundation reduction benefits are measured as the reduction in the amount of flood damages or related costs.

To determine the present value of property subject to flooding, an inventory was conducted. Overflow boundaries for the Standard Project Flood (SPF) were delineated on aerial photographs of Metropolitan Phoenix. This area was then divided into 1/2-mile square sections using major surface streets as boundaries. Structure counts and measurements of structure square footage were made for each section. Identification of structure type was made with the use of zoning maps (if not readily apparent from the photos). Field investigations of the study area were conducted to determine estimates of structure values (using Marshall and Swift Valuation Appraisals). Structures were also sampled in the field for estimates of first-floor elevations.

Open space within the flood plain is virtually non-existent. Therefore, no projection was made to accommodate future development.

Under present conditions, the SPF (for this study, an event with a probability of occurring once every 500 years) would inundate approximately 21,300 single family homes and 3800 other multi-family residences, businesses, and public facilities. Total value of development in the overflow area subject to flooding is about 2.4 billion dollars. Table 3-1 displays land use for the SPF overflow area under existing conditions. The impact of growth in personal income (affluence) was accounted for by increasing the value of residential

contents over a 15-year period (1986 to 2001). No growth in the value of damageable residential structural values was assumed. Future per-capita consumption of damageable residential contents was limited to a maximum of 75 percent of structural value in accordance with Principles and Guidelines. Table 3-2 displays the value of property under present conditions.

Future flood damages were assessed using overflow drawings for major floods indicating with and without project flood characteristics. Overflow areas were cross-sectioned at various intervals, noting with and without project boundaries, average flood depths, and flows (pls. 2-1 through 2-4, Appendix 2). Table 3-3 displays flooding characteristics by overflow area for various flood events.

Depth-damage relationships were used to evaluate the impact of the anticipated flows on development in the flood plain. These relationships (shown in table 3-4) were developed for each land-use category based on previous Corps studies. Depth-damage curves were applied to damageable property values to develop unit flood damages.

By combining the damage-discharge and discharge-frequency curves (Plate 3-1), a damage-frequency relationship was derived. Average annual damages were then calculated by using standard damage-frequency integration techniques. Equivalent average annual damages were computed next by summing the present worth of average annual damages and applying the capital recovery factor (partial payment series) for 3-1/4 percent discount rate. Table 3-6 presents the average and equivalent average annual damages for the flood plain without the project.

Impact of new channel capacities was reflected in the with-project overflow delineations, which noted residual flood characteristics (flows and depths) by frequency of event. Flood damages prevented were calculated by comparing expected damage under with and without project conditions. The annual benefits under existing conditions in 1986 are $2,619,000 (Table 3-7).

Average annual damages remaining with the project in place were calculated by comparing the with project annual damage to the without project annual damage. With 100-year protection, there is an estimated $682,000 in residual damage (table 3-8) in 1986. Most of this residual damage occurs below the Grand Canal. Note the generally higher with-project depths shown in table 3-3 for the area below the Grand Canal.

EMERGENCY COST SAVINGS

Emergency costs include those expenses resulting from a flood that would not otherwise be incurred, such as the costs of evacuation and reoccupation, flood fighting, and disaster relief; increased costs of normal operations during the flood; and increased costs of police, fire, or military patrol.

Emergency costs encompass a wide variety of programs. Some, such as emergency shelter and food, are primarily a function of occupancy of the floodplain and not of the value of development in the floodplain. Emergency costs are not projected to increase as a direct function of physical losses but as a function of population.

For this analysis, savings in emergency costs were attributed to lowering or eliminating the cost of temporary housing. These savings were determined in the following manner: (1) The number of displaced housholds was calculated per flood event. It was assumed that only those structures which incurred interior flooding would result in evacuated occupants. (2) Total number of days necessary for evacuation was estimated based on depth of inundation. (3) A daily cost of $60 per day was applied per household for temporary housing, higher food costs incurred under evacuation, and other additional expenses. Table 3-9 summarizes this data. The total number of households displaced by the SPF flood (27,943) reflects the assumption that multi-family residential structures contain an average of four first floor households per structure. Thus, it is assumed that approximately 10 percent of the residences in the overflow have first floor elevations above this flood stage.

Equivalent average annual benefits resulting from savings in emergency costs for this project total $125,000.

VII. COMPARISON TO JUNE 1972 FLOOD

The storm of June 22, 1972 produced an estimated 50-year flood on Cudia City Wash. As a result, the Arizona Canal was overtopped at 32nd and 40th Streets. (Other areas in the Reach 4 study area incurred flooding, but of less than 50-year flood intensity. See further discussion in Hydrologic Analysis, Appendix 1.) At 1986 price levels, damages in the overflow area created by the 32nd and 40th Streets breaks totalled $15.6 million. As discussed in Appendix 2, the hydrologic-hydraulic model was calibrated to reconstitute the 1972 flooding conditions by adjusting the depth of flooding at various points within the Reach 4 floodplain. Without-project damages for the 50-year flood determined by this analysis within the June 1972 overflow boundary total $17.0 million. However, this analysis calculates 50-year damages based on the current (1986) level of development within the floodplain. Comparability of this analysis' damage projections to the 1972 flood damage figure is apparent given the increase in development between 1972 and 1986.

VIII. ANNUAL PROJECT COSTS

Annual project costs were computed using the discount rate of 3-1/4 percent and an economic life of 100 years. Included in first costs were supervision and administration (S&A), engineering and design (E&D), and interest that would accrue during construction. Total first costs were discounted from 1991 to 1981, then annualized over 100 years. (See discussion early in this appendix under the heading "Methodology"). Added to this figure were annual operation and maintenance (O&M) costs[1], resulting in total annual project cost.

Table 3-10 displays computed interest during construction and table 3-11 displays the estimated annual charges. The non-Federal cost share includes lands, easements, rights-of-way, relocations of utilities and bridges, and the annual O&M cost. S&A and E&D costs were each computed as 10 percent of the construction first cost. Costs in the ACDC downstream (Dreamy Draw to Cave Creek) attributable to Reach 4 are also included in the Federal cost share estimate.

1. The present value of the stream of annual O&M costs from 1991 to 2080 ($123,100 yearly) was also discounted to 1981, then annualized over the 100-year system life.

IX. BENEFIT-TO-COST RATIO

A project is considered economically justified or feasible to build when the benefits equal or exceed costs. This is indicated by the benefit-to-cost (B/C) ratio. When annual project benefits are divided by annual project costs, a B/C ratio of 1.0 or greater indicates a justified project. A B/C ratio of less than 1.0 indicates the project is not justified under the National Economic Development (NED) criteria and participation by the Federal Government is not feasible unless there are other considerations which outweigh lack of economic feasibility.

Table 3-12 presents the economic justification of the project, listing the net benefits as well as the benefit-to-cost ratio.

X. REMAINING BENEFIT-REMAINING COST RATIO (RB/RC)

The RB/RC ratio displays the proportion of remaining annual benefits to remaining annual costs. Actual and scheduled expenditures through the 1987 fiscal year are subtracted from total Reach 4 costs before they are annualized (using the same methodology described on page 3-15) to derive remaining annual costs. Remaining costs for Reach 4 do not include design costs incurred and the costs of rights-of-way that have already been acquired. To compute remaining annual benefits, any benefits already accruing to completed functional units of the project are excluded from total Reach 4 benefits before they are annualized. Since Reach 4 will not be completed until 1991, and has no independent operational units scheduled for completion prior to that date, remaining annual benefits are the same as total Reach 4 annual benefits. Table 3-13 displays the RB/RC at both the applicable discount rate of 3-1/4 percent (1.3 to 1) and the fiscal year 1987 discount rate of 8-7/8 percent (0.5 to 1).

XI. SENSITIVITY ANALYSIS

As a test to determine the sensitivity of costs and benefits to changes in the discount rate, a benefit to cost ratio was calculated using 8-7/8 percent (the Fiscal Year 1987 rate of return for analyzing the economic justification of Federal water resources projects as established by the Water Resources Council). Average annual benefits resulting from total flood damages prevented reduction using this discount rate equal $1,329,000. Average annual costs for the project totaled $3,239,000. The resultant benefit-to-cost ratio is 0.4 to 1, indicating the project is not economically justified under this condition. Net project benefits equal -$1,910,000.

Table 3-1. Land Use in the SPF Overflow Area Reach 4, ACDC, Under Existing Conditions.

Land Use	Estimated Number of Units
Single-Family Residential	21,300
Multi-Family Residential	2,400
Commercial	1,300
Industrial	20
Public	70
TOTAL	25,090

Table 3-2. Value of Development in SPF Overflow Area Under Existing (1986) Conditions. Reach 4, ACDC.

(Thousands of Dollars)

Land Use	Value
Residential	
Structures	$ 968,582
Contents	453,435
Commercial	
Structures	402,307
Contents	451,506
Industrial	
Structures	1,176
Contents	1,176
Public	
Structures	73,325
Contents	33,278
TOTAL	$2,384,785

Table 3-3. Flooding Characteristics in Overflow Area, Reach 4, ACDC.

(With and Without Project)

Overflow Area	Frequency of Event	Peak Flow (ft^3/s)	Maximum Depth (ft)	Average Depth (ft)
Between Ariz. Canal & Grand Canal				
W/O Proj.	SPF	12,000	3.8	1.6
W/Proj.	SPF	4,300	2.5	1.2
Below Grand Canal				
W/O Proj.	SPF	16,000	1.6	1.4
W/Proj.	SPF	8,300	1.1	1.1
Between Ariz Canal & Grand Canal				
W/O Proj.	100 Yr	5,000	2.8	1.1
W/Proj.	100 Yr	1,200	0.7	0.3
Below Grand Canal				
W/O Proj.	100 Yr	6,600	1.2	1.1
W/Proj.	100 Yr	3,700	0.7	0.7
Between Ariz. Canal & Grand Canal				
W/O Proj.	50 Yr	3,500	2.6	1.0
W/Proj.	50 Yr	400	0.6	0.2
Below Grand Canal				
W/O Proj.	50 Yr	2,700	0.7	0.7
W/Proj.	50 Yr	2,700	0.6	0.6
Between Ariz. Canal & Grand Canal				
W/O Proj.	25 Yr	2,300	2.5	0.9
W/Proj.	25 Yr	580	0.4	0.2
Below Grand Canal				
W/O Proj.	25 Yr	1,700	0.6	0.6
W/Proj.	25 Yr	1,700	0.4	0.4

Table 3-4. Damages As A Percent of Structure Value By Depth of Inundation, Reach 4, ACDC.

Depth[a] (Feet)	Residential		Commercial		Industrial		Public	
	Structure	Contents	Structure	Contents	Structure	Contents	Structure	Contents
-0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	2.0	0.5	0.0	3.7	1.0	0.0	1.0	0.0
1.0	10.0	17.0	8.9	16.0	4.0	3.0	4.0	5.0
2.0	15.0	30.0	16.0	27.0	15.0	10.0	13.0	16.0
3.0	28.0	52.5	25.0	40.2	25.0	22.0	25.0	28.0
4.0	32.5	63.5	31.9	58.9	35.0	40.0	40.0	45.0

a. Depths are relative to first-floor elevation.

Table 3-5. Flood Damage by Frequency of Event, Reach 4, ACDC.
(Thousands of Dollars)

Land Use	25-Year Flood		50-Year Flood		100-Year Flood		SPF	
	Structures	Contents	Structures	Contents	Structures	Contents	Structures	Contents
Without Project Condition								
Residential	16,954	6,990	24,473	12,422	39,836	24,688	82,290	64,065
Commercial	617	2,974	1,091	5,961	3,214	13,893	17,491	36,767
Industrial	0	0	0	0	12	2	21	9
Public	155	29	310	66	666	159	1,760	874
Total	17,726	9,993	25,874	18,449	43,728	38,742	101,562	101,715
With Project Condition								
Residential	2,185	85	4,640	410	6,969	1,305	47,246	29,787
Commercial	0	0	59	1,012	355	2,414	3,832	17,527
Industrial	0	0	0	0	4	0	12	0
Public	0	0	87	4	166	39	923	271
Total	2,185	85	4,786	1,426	7,494	3,758	52,013	47,585

Table 3-6. Annual Flood Damage, Reach 4, ACDC, Without Project.

(Thousands of Dollars)

Land Use	1986	1991[a]	2000-2080	Equivalent Average Annual (3-1/4% 90 Years)	Equivalent Average Annual (Discounted to 1981, 3-1/4% 100 Years)
Residential					
Structures	1,690	1,690	1,690	1,690	1,208
Contents	946	1,135	1,419	1,376	983
Commercial					
Structures	146	146	146	146	104
Contents	488	488	488	488	349
Industrial					
Structures	24	24	24	24	17
Contents	7	7	7	7	5
Public					
Structures	0	0	0	0	0
Contents	0	0	0	0	0
TOTAL	3,301	3,490	3,774	3,731	2,666

a. Project year one.

Table 3-7. Annual Flood Damage Prevented, Reach 4, ACDC. With 100-Year Protection.

(Thousands of Dollars)

Land Use	1986	1991	2000-2080	Equivalent Average Annual (3-1/4% 90 Years)	Equivalent Average Annual (Discounted to 1981, 3-1/4% 100 years)
Residential					
Structures	1,308	1,308	1,308	1,308	935
Contents	793	952	1,190	1,154	824
Commercial					
Structures	125	125	125	125	89
Contents	371	371	371	371	265
Industrial					
Structures	17	17	17	17	12
Contents	5	5	5	5	4
Public					
Structures	0	0	0	0	0
Contents	0	0	0	0	0
Total	2,619	2,778	3,016	2,980	2,129

Table 3-8. Annual Residual Flood Damages, Reach 4, ACDC, With 100 Year Protection.

(Thousands of Dollars)

Land Use	1986	1991	2000-2080	Equivalent Average Annual (3-1/4% 90 Years)	Equivalent Average Annual (Discounted to 1981, 3-1/4% 100 years)
Residental					
Structures	382	382	382	382	273
Contents	153	183	229	222	159
Commercial					
Structures	21	21	21	21	15
Contents	117	117	117	117	84
Industrial					
Structures	7	7	7	7	5
Contents	2	2	2	2	1
Public					
Structures	0	0	0	0	0
Contents	0	0	-	0	0
Total	682	712	758	751	537

Table 3-9. Displaced Households by Frequency of Event and Depth of Inundation, Reach 4, ACDC.

Without-Project Condition					
Depth of Inundation 1/	Days Evacuated	SPF Flood	100-Year Flood	50 Year Flood	25-Year Flood
0.0-1.0 foot	3	23,408	24,564	13,377	11,214
1.1-2.0 feet	9	3,714	1,140	804	484
2.1-3.0 feet	21	821	0	0	0
Total		27,943	25,704	14,181	11,698
With-Project Condition					
Depth of Inundation	Days Evacuated	SPF Flood	100-Year Flood	50-Year Flood	25-Year Flood
0.0-1.0 foot	3	20,079	2,082	286	0
1.1-2.0 feet	9	1,848	0	0	0
2.1-3.0 feet	21	0	0	0	0
Total		21,927	2,082	286	0

1/ Relative to first floor elevation.

Table 3-10. Interest During Construction, Reach 4, ACDC.

(3-1/4 %, Thousands of Dollars)

Total Project Cost	Construction Period	Average Cost Balance During Construction Period	Accumulated Interest & Principal	Accumulated Interest
$79,850.00	1-1/2 years	$32,000.0	$33,577.0	$1,570.0

Table 3-11. Estimated Annual Charges, Reach 4, ACDC.

(1986 Price Levels, Thousands of Dollars)

Federal		
First Cost, Construction[a]	$39,040.0	
Engineering & Design	3,900.0	
Supervision & Administration	3,860.0	
ACDC Downstream Cost Attributable to Reach Four	7,090.0	
Interest During Construction	1,064.0	
Gross Investment	54,954.0	
Discounted Gross Investment (from 1991 to 1981 3-1/4%)	39,911.6	
Amortization = 0.0338835 (3-1/4%, 100 Years)	1,352.3	
Annual Operation & Maintenance	0.0	
Subtotal		1,352.3
Non-Federal		
Lands and Damages	17,385.0	
Relocations	8,575.0	
Interest During Construction	513.0	
Gross Investment	26,473.0	
Discounted Gross Investment (from 1991 to 1981 3-1/4%)	19,226.6	
Amortization = 0.0338835 (3-1/4%, 100 Years)	651.5	
Annual Operation & Maintenance	88.0	
Subtotal		739.5
Total Annual Charges		$2,092.0

a. Includes all costs associated with construction of Cudia City Wash Sediment Basin.

Table 3-12. Summary of Annualized NED Benefits and Costs, Reach 4, ACDC.

(3-1/4%, 100-Year Life)

Flood Hazard Reduction Benefits:	$2,129,000
Emergency Cost Savings	$ 125,000
Total Project Benefits	$2,254,000
Project Costs:	$2,092,000
Net Benefits:	$ 162,000
B/C Ratio:	1.08

Table 3-13. Summary of Remaining Annualized Benefits and Costs, Reach 4, ACDC.
(100 Year Life)

	3-1/4%	8-7/8%
Remaining Benefits	$2,254,000	$ 1,329,000
Remaining Costs	$1,803,000	$ 2,679,000
Net Remaining Benefits	$ 451,000	$-1,350,000
RB/RC Ratio	1.3	0.5

NOTE:

DISCHARGES SHOWN WERE DERIVED BY AVERAGING FLOWS AT VARIOUS POINTS WITHIN EACH FREQUENCY'S OVERFLOW AREA.

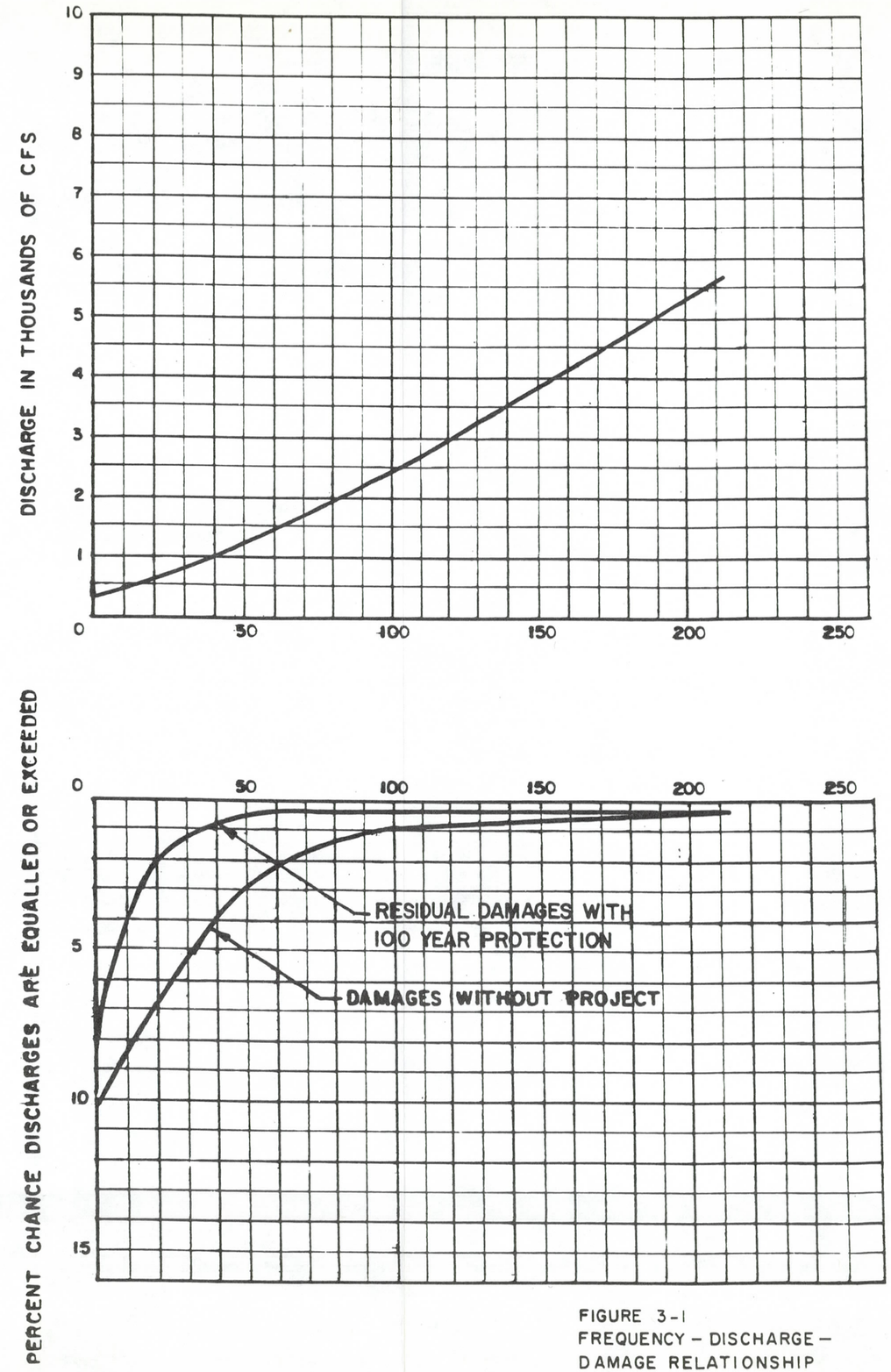

FIGURE 3-1
FREQUENCY - DISCHARGE - DAMAGE RELATIONSHIP