DESIGN & COST APPENDIX

Phoenix Urban Study Final Report

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

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ENGINEERING INVESTIGATIONS, DESIGN AND COST

APPENDIX

PHOENIX URBAN STUDY
FINAL REPORT

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
DECEMBER 1981

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PREFACE

Water has been the single most important factor contributing to the phenomenal growth of the Phoenix metropolitan area. A century ago, planners in the Salt River Valley were laying the groundwork to develop the limited water resources of the area to the maximum extent possible. In so doing they provided the most feasible location for development of a large population center in the lower Colorado River Basin. The successful development that resulted from the efforts of these pioneers in water resource planning, however, has placed an even greater demand on current available water resources. In recognition of the need to extend and refine water resource planning, the U.S. Army Corps of Engineers undertook the Phoenix Urban Study in cooperation with local authorities.

THE STUDY

During the course of the Phoenix Urban Study, water resource plans formulated were consistent with other urban programs and flexible enough to allow accommodation of changing social and economic conditions. Because the study interfaced closely with water resource programs of other agencies, special attention was devoted to insuring that the Urban Study did not duplicate the efforts of other agencies, but rather that it served as an extension and a coordination of these efforts.

STUDY REPORT

The Engineering Investigations Design and Cost Appendix of the Phoenix Urban Study Final Report provides the Technical detail to support the engineering decisions made during the course of the program. The organization of the Final Report and relation of the Engineering Investigations Design and Cost Appendix to it are shown in Figure P-1.

CHAPTER I

INTRODUCTION

The Engineering Investigations Design and Cost Appendix presents discussions of the technical data generated during the course of the Phoenix Urban Study. Included in this document, where applicable, are the preliminary designs of various components, all assumptions, cost curves, or other estimating devices used, and engineering data which influenced the acceptance or rejection of components.

The Urban Study performed work in the fields of water quality, flood control, and water conservation. Because the examination of issues and problems relating to water quality comprised much of the Urban Study's effort and produced implementable plans, information from these investigations makes up the largest portion of this appendix. Engineering, design, and cost data are shown for the wastewater management alternatives developed for the Phoenix metropolitan area by the Urban Study. These data were generated largely through the Urban Study's efforts as the metro planning agency for the Maricopa Association of Government's program to implement Section 208 of Public Law 92-500, the Federal Water Pollution Control Act Amendments of 1972.

All but one of the flood control projects examined by the Phoenix Urban Study (control of flooding along the Salt River through the Phoenix metropolitan area) did not warrant further study or action by the Corps of Engineers or other federal agency. Hydrologic data on which these negative flood control investigations were based are included in this appendix. Preliminary conceptual designs for the negative projects and reasons for their abandonment can be found in the Summary Report and Plan Formulation Appendix of the Final Report. Basic economic information for the Salt River flood control alternatives also is included in these documents, although advanced engineering, design, and cost data for the plans are yet to be generated by the Bureau of Reclamation with the assistance of the Corps of Engineers in the course of the Central Arizona Water Control Study.

An important facet of the Phoenix Urban Study's examination of flood control alternatives involved the feasibility of flood warning systems for the metropolitan area. A summary of the findings and tentative recommendations resulting from this effort are presented in this report.

Planning for the water conservation by the Phoenix Urban Study involved examinations of the possibilities of achieving conservation through

CHAPTER II

208 WATER QUALITY PLAN

BACKGROUND

The 208 Water Quality Planning portion of the Phoenix Urban Study was undertaken in 1976 to fulfill the requirements of Section 208 of Public Law 92-500, as amended by the Federal Clean Water Act of 1977 (P.L. 95-217). Under the requirements of Section 208 of the Act, the Maricopa Association of Governments (MAG) was designated by the Governor of Arizona as the agency responsible for development of an areawide wastewater management plan for Maricopa County. To accomplish this task, MAG requested that the Corps of Engineers, as part of the Phoenix Urban Study, be responsible for 208 planning for the Phoenix metropolitan Area. 208 planning efforts have been directed toward the following problems in the metropolitan area.

1. The Phoenix area is expected to continue to grow rapidly over the next 20 years. This population will require a significantly enlarged wastewater treatment system to handle increased flows.

2. Water resources are being depleted in the area and reuse of wastewater could help conserve these resources.

3. Water quality problems in some areas are presently being caused by discharges from wastewater treatment plants.

4. The existing wastewater system is operating at capacity, and most facilities are in need of upgrading to handle increased flows and to improve water quality. Future growth will place additional stress on the system.

The first step toward solution of these problems was accomplished by identifying the best areawide wastewater collection and treatment systems for the Phoenix metropolitan area. The process for developing and selecting a wastewater plan involved examination of many elements simultaneously. These elements include treatment plant locations and processes, collection systems, sludge handling, and effluent disposal or reuse. In each there were a number of alternative approaches and they all were analyzed to ensure the best became part of the final plan.

Based on a study of population, natural drainage, availability of wastewater reuse sites, and the extent of the existing sewer system, Report," May, 1977. These summary reports were presented at a public meeting in May, 1977, and the MAG Regional Council adopted seven areawide alternatives for further study in June of 1977.

Small Array of Alternatives

The following seven regional alternatives (small array) were selected for further refinement:

1-1 Regional Plant at 91st Avenue
2-3 91st Avenue + Chandler
2-4 91st Avenue + Northeast
3-1 92st Avenue + Northeast +Chandler
3-2 91st Avenue + Citrus Road + Chandler
4-1 91st Avenue + Citrus Road + Northeast + Expanded Chandler
5-1 91st Avenue + Citrus Road + Northeast + Chandler + 48th Street

These alternatives were selected for the cost, flexibility, and assurance that all the major plants were included.

The study area was then divided into the east and west subregions. The selected areawide alternatives were analyzed in greater detail for the two subregions. Advanced (tertiary) treatment levels were used for evaluation of some of the proposed treatment plants based on the nature of the proposed effluent reuse system. These levels were developed to meet the 1983 goals of PL92-500 and to examine the possibilities of wastewater reuse for parks, golf course, recreational lakes, groundwater recharge, and unrestricted agricultural irrigation. The alternatives also were evaluated using the following criteria: energy consumption, costs, implementation time, facility life, reliability of treatment, Federal funding, impact on groundwater, impact of emergency discharges, feasibility, environmental and socio-economic impacts, and ability to meet local reuse options.

Following this analysis the alternatives were narrowed to four final areawide alternatives described as follows:

Alternative 1 - 91st Avenue, 23rd Avenue, Tolleson, Gilbert, Chandler: Under this alternative, six plants serve the Phoenix metro area to the year 2000. The existing 90 million gallons daily (mgd) 91st Avenue plant would be expanded to serve all service areas except Tolleson/Peoria, portions of Gilbert, and Chandler which have their own treatment facilities.

The 91st Avenue plant would be expanded immediately to handle flows from the contributing service areas. Between 1990 and 1995 an additional

Flows from the northeast area (portions of Scottsdale, Phoenix, and Paradise Valley) would be delivered to a new facility located on the Salt River Indian Community land. A new pump station at Indian Bend Road and Hayden and force main would be required to lift flows to the proposed treatment plant site. The remaining service areas would be served as described under Alternative 1.

Alternative 4 - 91st Avenue, 23rd Avenue, Tolleson, Gilbert, Chandler, Northeast Area, Reems Road: Under this alternative, eight treatment facilities serve the Phoenix metro area. The 91st Avenue plant would be expanded to handle flows from El Mirage, Glendale, Luke AFB, Phoenix, Sun City, Surprise, and Youngtown. Staging of construction would be as previously described with expansion as required.

The remaining service areas would be served as previously described with plants serving Tolleson/Peoria, Chandler, portions of Gilbert, Goodyear/Avondale/Litchfield Park, and portions of northeast Phoenix/Paradise Valley/Scottsdale.

These integrated alternatives were analyzed, evaluated and presented to the advisory groups, communities, and MAG in a series of meetings and a public hearing in October, 1978. A summary brochure entitled "Metro 208 Areawide Alternatives," October, 1978, also was prepared and presented to the various entities during this period.

Selection of the Final Plan

Based on the recommendations of the advisory groups and the decisions of the individual communities concerning the final four areawide alternatives, the MAG Regional Council adopted Alternative 2 as the final areawide wastewater management plan for the Phoenix metropolitan area on November 1, 1978. (See Figure II-1)

The concept developed for the selection of the areawide wastewater management plan as described above shows a gradual transition from 36 conceptual alternatives through the large (20) and small (7) array of regional alternatives. At this point the area was divided into the east and west subregional areas and alternatives developed for these. Ultimately two alternatives from the east and west areas were selected for integration into four areawide alternatives. The final plan was then selected from these four.

The level of detail for each step of the process also varied. At the conceptual array stage, 1976 population numbers were used along with preliminary flow information from the communities to develop the total flows for each of the alternatives. Potential reuses were identified without any costs being generated. Land treatment alternatives were treated in a similar manner. In the large array,

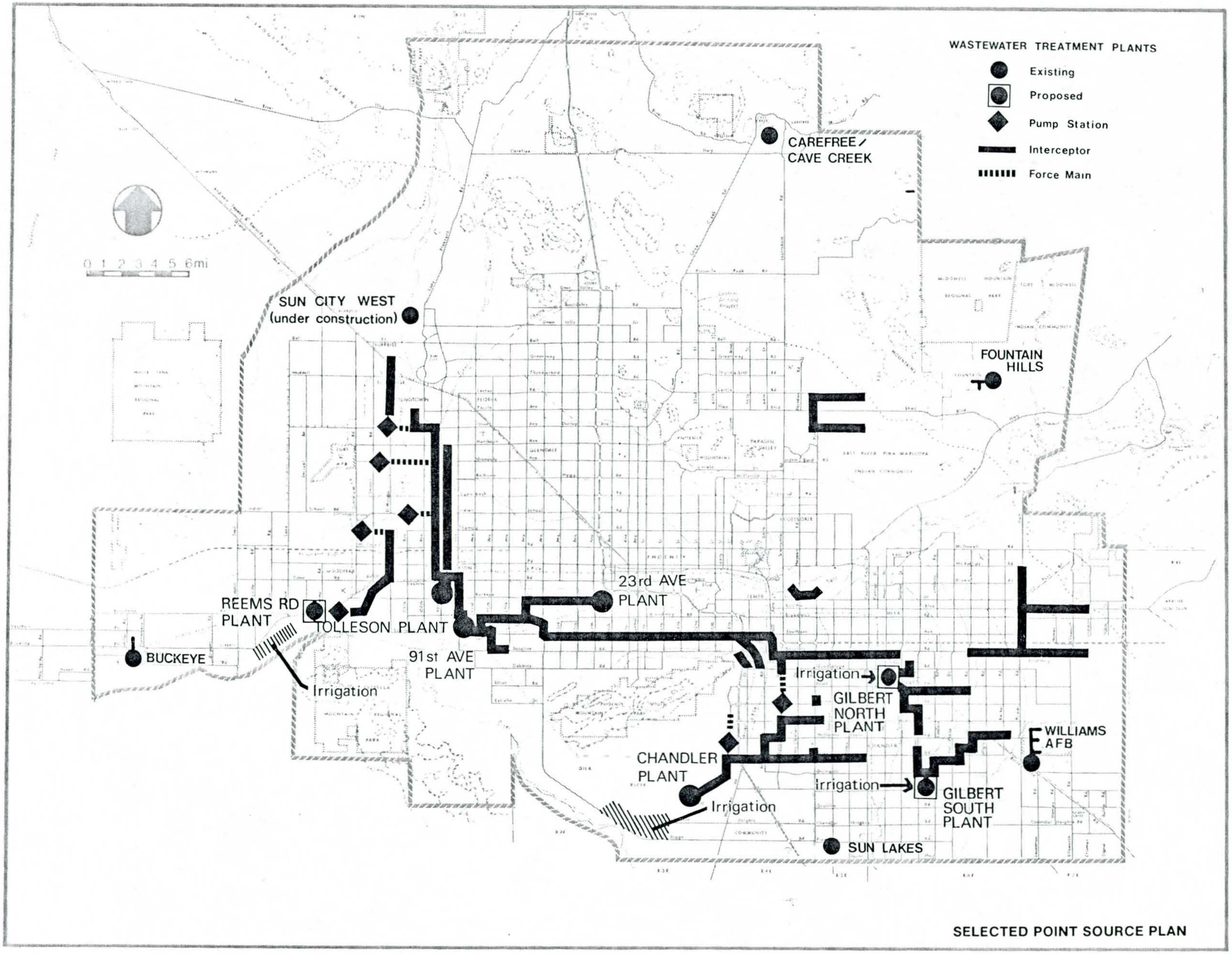

WASTEWATER TREATMENT PLANTS
Existing
Proposed
Pump Station
Interceptor
Force Main
0 1 2 3 4 5 6mi
CAREFREE / CAVE CREEK
SUN CITY WEST (under construction)
FOUNTAIN HILLS
REEMS RD PLANT
TOLLESON PLANT
23rd AVE PLANT
91st AVE PLANT
BUCKEYE
Irrigation
Irrigation
GILBERT NORTH PLANT
CHANDLER PLANT
Irrigation
Irrigation
WILLIAMS AFB
GILBERT SOUTH PLANT
SUN LAKES
SELECTED POINT SOURCE PLAN

FIGURE II-1

CHAPTER III

DESIGN AND COST DATA FOR WATER QUALITY

This chapter presents the background design and cost data which were utilized to develop and to evaluate the technical aspects of the final four areawide 208 plans. These data include population totals and distributions, wastewater flows, wastewater loads, design criteria, and costs. Background data have not been included for the conceptual, large, small, and subregional alternative array analyses for the sake of brevity and because these data are well documented in prior reports as listed in the Bibliography.

POPULATION

Maricopa County is one of the fastest growing areas in the United states and one of the few metropolitan areas of the nation that has continued to grow in recent years.

The population of Maricopa County, of which about 93 percent is presently in the Phoenix metropolitan area, increased from 187,000 in 1940 to 1,173,000 people in 1974, a 630 percent increase. This represents an annual growth rate of 5.6 percent since 1940. Contributing to the population growth are the migration to the west and the increasing importance of manufacturing and industrial operations in the area. Climate, job opportunities, nearby major recreational facilities and a strong retirement appeal also have contributed to the population surge in the study area.

Development of Population Projections

In the past, a number of organizations, both public and private, have made population projections for Maricopa County.

Valley Area Traffic and Transportation Study (VATTS): The Maricopa Planning and Zoning Department published, in a 1970 report, population projections for 1980 and 1995. This study was primarily for traffic and transportation analysis, but it did develop the small geographic units referred to as Traffic Analysis Zones (TAZ).

MAG Population Projections: MAG decided in 1972 that the VATTS population projections needed to be updated and projected further into the future. Also the number of Traffic Analysis Zones were responded and the MAG staff totaled the population figures. The total exceeded the 2.3 million control total set by DES. The managers then decided to meet and work out the differences in the totals. On August 17, 1977, the MAG Management Committee discussed the population projections for each planning area and came to a consensus on how the population would be distributed. On November 7, 1977, the Management Committee adopted the population projections. MAG then allocated these adopted population figures to smaller areas, working closely with each member jurisdiction to ensure conformance with local zoning plans and objectives as agreed to by the MAG Management Committee and the Regional Council.

Total Maricopa County populations were allocated by Municipal Planning Areas and further allocated to Community Aggregate Planning Model (CAPM) zones within MPA's. CAPM zones are smaller areas within the municipal planning areas which ignore city limits and were originally delineated for use in transportation planning studies.

Table III-1 presents the total planning area population for each community to be served by existing or proposed facilities under the areawide alternatives. Populations for the various service areas were developed in five-year increments from 1980 to the year 2000.

FLOWS

The projection of future wastewater flows are dependent upon two factors: 1) future projections of population, and 2) estimates of the future contribution of wastewater from each individual. This latter estimate is commonly called "unit flows" and can be expressed in gallons per capita per day (gpcd).

In the 208 program considerable work was devoted to the analysis and verification of these unit flow estimates. All estimates were reviewed by the MAG and the Corps staff, the 208 advisory groups, and communities prior to adoption for use in the 208 program. The following discussion is a brief presentation of the methodology used and the results obtained in the development of the unit flows.

The unit flows which were utilized in the 208 study were developed by community service areas to correspond with the availability of accurate flow and population data. The procedure used to develop these flows was to compare 1975 census data for resident population with the recorded 1975 average annual flows. A detailed investigation of the existing sewer system also was accomplished to estimate the percent of the population which was connected to the system. Only connected resident population was used in the development of the unit flows. These estimates of unit flows were then compared to

TABLE III-1
PROJECTED POPULATION BY PLANNING AREA

Planning Area	1980	1985	1990	1995	2000
Avondale	11,700	14,100	21,300	28,600	36,030
Buckeye	3,000	3,800	5,100	6,500	8,000
Carefree - Cave Creek	2,800	4,045	5,800	8,300	9,000
Chandler	30,000	42,500	58,800	75,200	92,700
El Mirage	5,700	7,500	9,400	11,400	13,500
Fountain Hills	5,000	7,005	10,000	15,000	22,500
Gilbert	10,800	14,700	24,800	34,800	45,500
Glendale	80,000	97,700	115,800	134,400	154,800
Goodyear	3,750	5,260	9,800	14,250	19,000
Guadalupe	4,500	5,000	6,000	6,900	8,000
Litchfield Park	3,250	4,140	8,300	12,550	16,900
Luke AFB	4,900	5,000	5,000	5,000	5,000

TABLE III-1
PROJECTED POPULATION BY PLANNING AREA

Planning Area	1980	1985	1990	1995	2000
Mesa (includes East Mesa)	162,777	189,605	213,799	237,880	265,144
Paradise Valley	13,500	15,800	16,200	16,700	17,400
Peoria	19,800	23,400	37,900	52,300	67,700
Phoenix	741,000	802,200	875,900	952,100	1,042,100
Scottsdale	84,500	92,700	96,600	100,700	106,400
Sun City	40,192	47,817	48,310	48,439	48,755
Sun City West	6,265	14,550	24,276	32,836	42,000
Sun Lakes	1,800	3,300	4,800	6,200	7,500
Surprise	3,600	3,700	4,700	5,700	6,800
Tempe	126,800	162,700	168,600	175,100	184,000
Tolleson	4,100	4,700	9,400	14,100	19,000
Williams AFB	3,338	3,400	3,469	3,472	3,507
Youngtown	2,000	2,000	2,000	2,000	2,200
Remainder of Maricopa County Inside Urban Planning Area	13,528	14,823	17,846	20,773	24,294
TOTALS	1,388,600	1,591,445	1,803,900	2,021,200	2,268,000

historical estimates used in other recent studies as well as to typical national values.

Since these unit flows were developed using the total recorded flows, they contained allowances for both commercial and industrial as well as residential flows. Based on discussions with the staffs of the cities in the Phoenix metropolitan area, it appeared that within the metropolitan Phoenix area the "mix" of flows was about 70 percent residential, 20 percent commercial and 10 percent industrial.

As long as this "mix" of flows remained relatively constant, the use of unit flows based on population only served as a reasonable estimating device. In two cases, however, additional work was required to estimate the needs of the non-residential users. The first of these special cases occurred in Tolleson where the treatment system was small and there existed a large water-using industry, the Swift Meat Packing Plant, tributary to the treatment plant. The second case was in Tempe where the "mix" of flows was expected to change in the future because of a projected major change in the ratio of commercial/industrial flow to residential flows. In the latter case the 208 program used previous studies and the adopted land-use plan to develop special unit flows for Tempe. For Tolleson, it will be necessary for future 201 studies to give special attention to the development of the existing and potential new industrial connections to the system.

Treatment systems normally are designed on an average daily flow and loadings basis. Because of the area's influx of winter visitors, the 208 program investigated the peak monthly flows to see if they were unusually high compared to the average annual flows. This investigation indicated that the peak monthly flows varied anywhere from about 5 to 15 percent higher than the average annual flows for each of the service areas. In addition, it was found that peak monthly flows for the service areas occurred at different times as shown in Table III-2. At the 23rd and 91st Avenue wastewater treatment plans the peak monthly flows were 10 percent and 5 percent greater respectively. Since these were not abnormally high peaks, the 208 planning effort used the average daily flows on an annual average basis as the basis of design for the treatment plants except for Mesa and Peoria. In these two cases it was found that average monthly flows were sustained over three consecutive months at a flow significantly higher than annual average flow. Unit flows which are higher than annual average unit flows were selected for these communities to reflect their actual sustained flow production during the peak months.

The unit flows developed for the 208 program are listed as follows:

Service Areas	Unit Flows (gpcd)
Glendale	110
Luke AFB	1.5 MGD
Mesa	85
Phoenix - 23rd Avenue	105
Phoenix - 91st Avenue	100
Scottsdale	105
Tempe Commercial & Industrial	1,760 g/ac/day
Residential	65
Tolleson	110
Williams AFB	1.0 MGD
Youngtown	70
Sun City	70
All other communities	100

Theoretically, these unit flows include an allowance for inflow and infiltration since they were developed on yearly average flows experienced at the area treatment plants. Two studies of infiltration /inflow conditions in the area, however, have concluded that infiltration and inflow are non excessive and negligible. For further information see "Phoenix Urban Study, Plan of Study, Appendix C, Water and Wastewater Technical Report," Section 1, U.S. Army Corps of Engineers, August 1975 and "Metropolitan Phoenix Facility Plan - 1978, Current Situation, Appendix A4.1, Infiltration/Inflow Analysis," John Carollo Engineers, 1979.

Based on the current "mix" of residential, commercial, and industrial flows in the metropolitan area, most of the communities in the area meet the Environmental Protection Agency's limit of 70 gpcd residential flow. Exceptions to this are Glendale, Phoenix at 23rd Avenue, Scottsdale, and Tolleson. The implementation of the water conservation and flow reduction program described later in this section, however, will bring all of the communities within the EPA limit. An additional adjustment to the unit flows was made to provide for the impact of proposed future water conservation and flow reduction programs.

These water conservation and flow reduction targets were estimated and adopted based on the following information:

1. Analysis of possible flow reduction measures and devices by the 208 staff and the Corps of Engineers.

2. Review and recommendation by the MAG Advisory and Management Committees, city staffs, and state staffs.

3. Decision by the Home Builders Association of Central Arizona to promote building of water conserving homes.

TABLE III-2
UNIT FLOWS CALCULATED FROM 1975 DATA

Community Service Area	UNIT FLOWS (gpcd) Average Annual	Peak Month
Phoenix 91st Avenue	100	116
Phoenix 23rd Avenue	105	118 (Dec)
Mesa	79	86 (Dec)
Tempe	90	95 (Mar)
Scottsdale	105	114 (Jan)
Glendale	108	116 (Dec)
Sun City	68	82 (Mar)
Peoria	70	101 (Oct)
Tolleson	110	143 (Aug)

After extensive review by the Corps and MAG staffs, and the 208 advisory groups, MAG adopted a flow reduction estimate of 4 percent for existing connections and 15 percent for all new connections after 1980. The 4 percent reduction was applied to existing connections at the rate of 1 percent for each five year period after 1980. For example, a service area having an existing unit flow of 100 gpcd would have the adjusted future unit flows as follows:

	1980	1985	1990	1995	2000
Existing Population	100	99	98	97	96
New Population after 1980		85	85	85	85

These adjustments resulted in a net overall reduction of total flows of approximately 10 percent by the year 2000.

The adopted flow reduction targets will be met through a combination of the following:

1. A public education program to make the general public aware of the need for water conservation and the measures available to save water. This education program already has been implemented by MAG communities and includes distribution of water conservation literature along with monthly utility bills, television commercials promoting water conservation, and newspaper articles which explain the need for and available measures for saving water.

2. The adoption and implementation of the plan by the Home Builders Association of Central Arizona to build water conserving homes. As in the case of the public education program, this plan has already been implemented by the Association in the Phoenix area.

3. Possible revision of plumbing codes in the area to promote use of water conserving devices.

For the design of the inteceptor system, the peak hourly flows must be determined. The 208 program examined the flow records by areas tributary to the major interceptors back to 1970. The largest flow event for each system over the past eight years were used as a basis of computing the peaking factor. This analysis resulted in the use of peaking factors which were in some cases different from those previously used for design in the area.

These 208 peaking factors are based on analysis of actual flow data for the system and are supported by flow records. The peaking factors utilized in the 208 program are shown below:

Range of Average Annual Flow Rates	Ratio: Peak Hour Flow to Average Annual Flow Rate
Less than 0.5 MGD	2.5*
0.5 to 1.0 MGD	2.3
1.0 to 40 MGD	2.2
More than 40 MGD	1.9

*Required by Arizona Department of Services regulations where no records exist of a lower ratio.

Table III-3 presents the total projected Urban Study average daily flows for each community to be served by existing or proposed facilities under the areawide alternatives. These flows for the various service areas were developed in five-year increments from 1980 to the year 2000.

LOADINGS

In order to analyze the existing sewage treatment plants and to provide a basis for development and analysis of alternative areawide treatment plant concepts, it is necessary to develop or estimate unit wastewater loading rates from present or future contributing sources in the study area. These loading rates are used in conjunction with contributing popualtion estimates to determine capacity requirements in certain processes within wastewater treatment plants. In the 208 study, the development of unit loadings depended on several data sources and included a review of historic loadings and a comparison of records of actual loadings with contributing population totals for existing treatment systems in the Phoenix metropolitan area. Even though future total waste loads were not projected to change, projected reductions in wastewater flows rates through water conservation practices would increase the loading factor concentrations from the contributing population. It was determined also that an analysis of areawide treatment concepts should include both annual average and peak monthly average loading rates for biochemical oxygen demand (BOD) and suspended solds (SS). Treatment facilities normally are designed to provide a specified percent removal of these constituents.

The review and development of unit loadings involved discussions with the staff of the City of Phoenix and calculations and verification by the engineers for the City of Phoenix. Based on these factors, the unit BOD and suspended solids loading factors for existing and future population sources were selected as follows:

TABLE III-3
PROJECTED FLOWS (MGD)

Planning Area	1980	1985	1990	1995	2000
Avondale	0.7	0.9	1.5	2.1	2.8
Buckeye	0.3	0.4	0.5	0.6	0.7
Carefree-Cave Creek	0.3	0.4	0.5	0.7	0.8
Chandler	3.0	4.0	5.4	6.8	8.2
El Mirage	0.4	0.5	0.5	0.5	0.6
Fountain Hills	0.5	0.7	0.9	1.3	2.0
Gilbert	1.0	1.3	2.2	3.1	4.0
Glendale	8.6	10.0	11.5	12.9	14.5
Goodyear	0.3	0.4	0.7	1.1	1.4
Guadalupe	0.5	0.5	0.6	0.6	0.7
Litchfield Park	0.3	0.3	0.7	0.9	1.3
Luke AFB	1.5	1.5	1.5	1.5	1.4
Mesa	13.9	15.7	17.2	18.9	20.7
Paradise Valley	1.4	1.5	1.5	1.5	1.7
Peoria	1.8	2.0	3.1	4.2	5.4
Phoenix	75.9	80.4	86.0	91.7	98.7
Scottsdale	8.9	9.5	9.8	10.0	10.5
Sun City	2.8	3.2	3.2	3.2	3.2
Sun City West	0.4	0.9	1.5	2.0	2.6
Sun Lakes	0.2	0.3	0.4	0.5	0.7

TABLE III-3
PROJECTED FLOWS (MGD)

	1980	1985	1990	1995	2000
Surprise	0.4	0.4	0.5	0.5	0.6
Tempe	12.7	15.9	17.5	19.2	21.1
Tolleson	0.5	0.5	0.9	1.4	1.8
Williams AFB	1.0	1.0	1.0	1.0	1.0
Youngtown	0.1	0.1	0.1	0.1	0.1
Remainder of Maricopa County Inside Urban Planning Area	1.4	1.5	1.7	1.9	2.2
TOTAL	138.8	153.8	170.9	188.2	208.7

1. Existing Sources

	Annual (mg/1) Average		Peak Monthly (mg/1) Average	
	SS	BOD	SS	BOD
1980	210	190	290	270
1985	210	190	295	270
1990	215	195	295	275
1995	215	195	300	280
2000	220	200	300	280

2. Future Sources

1980	250	225	340	320
1985	250	225	340	320
1980	250	225	340	320
2000	250	225	340	320

Since these loading rates, developed from actual plant records, are fairly typical of medium strength domestic sewage, it was assumed in the 208 analysis that no unusual or special treatment process design provisions would be required to accommodate the wastewater loadings and that conventional or land treatment systems designed according to ADHS requirements or to accepted design criteria would result in a treatment system which would accommodate existing and future wastewater loads.

DESIGN CRITERIA

As the 208 study progressed, the criteria for conventional and land treatment plant design, effluent reuse, and cost estimating became more refined. At first, with a large number of plants and systems, the costs and criteria were general and the specifics of each site were not considered in detail. As specific plant locations were identified, the design and cost criteria were tailored to the individual systems. The following describes the criteria used in developing and costing the alternatives.

Reuse Criteria

One of the key elements throughout the 208 study in Phoenix was the concept of wastewater reuse. Prior to initiation of the Urban Study, several reuse systems were in operation and others were being planned. Some of these major reuses were as follows:

		mgd
o	Buckeye Irrigation District (existing)	26.8
o	Arizona Game and Fish (existing)	6.5
o	Arizona Nuclear Power Plant (future)	125.0
o	Roosevelt Irrigation District (future)	17.9
o	Irrigation at Chandler (future)	8.2
o	Irrigation (future)	3.6
o	Sod growing at Tolleson (existing and future)	7.2
		195.2

Other reuses in the area included park or golf course irrigation at Fountain Hills, Sun Lakes, Williams AFB, Leisure World, and Cave Creek.

The Urban Study, however, endeavored to look at other possible reuses around the area and to look at possible multiple uses of water. During the initial study phases, possible reuses identified included nuclear power plant cooling, increased irrigation in the western portion of the study area, recreational reuse in the Indian Bend Wash System in Scottsdale, irrigation to the east and recreational reuse on the proposed Rio Salado Greenbelt concept, and groundwater recharge. These reuses and their corresponding water quality requirements were utilized in the study to identify and select treatment processes and in some cases possible treatment plant sites.

Concurrent with development of the collection and treatment alternatives, reuse options were identified and costed for all of the alternatives throughout the study. These proposed reuses included:

- o Existing Commitments
- o Rio Salado
- o Recreational use in Lower Indian Bend Wash and golf course irrigation near Indian Bend Wash
- o Recreational use in Upper Indian Bend Wash
- o Crop irrigation on the Salt River Indian Community Land
- o Crop irrigation on Gila Indian Community Land

- Increased supply of effluent to the west for crop irrigation

- Groundwater Recharge

The analyses indicated that the reuse alternatives were feasible, but, in order to be economical they needed to be near the treatment facility to avoid prohibitive transportation costs.

During a later phase of the 208 study, a detailed analysis was carried out on reuse alternatives for the eastside and westside. Before the reuse options could be developed in detail, however, the standards for effluent reuse were reviewed. The State of Arizona requires that wastewater be treated to a certain effluent quality depending on specific reuse, and the ADHS has established effluent quality standards as presented in Table III-4.

Although the information in Table III-4 is accurate as a general guide to the Arizona requirements, several of the reuse designations must be explained in more detail to cover the exceptions as follows:

1. Restricted agricultural use includes irrigation of non-edible crops and stock watering. These uses include fiber and forage crops not intended for human consumption and watering of farm animals other than producing dairy animals. In addition, orchard crops can be irrigated if the method of irrigation does not result in direct application to fruit or foliage.

2. Partially restricted agricultural use includes irrigation of any food crop where the product is subject to physical or chemical processing sufficient to destroy pathogenic organisms, irrigation of orchard crops regardless of the irrigation method, and watering of all farm animals including producing dairy animals.

3. Unrestricted agricultural use includes the uses mentioned above plus irrigation of crops which can be consumed in their raw or natural states.

4. Secondary contact recreation involves direct human body contact with the water, but normally not to the point of complete submergence. It is very unlikely that this water will be ingested, nor will critical organs such as eyes, ears, and nose normally be exposed to the water. This water may be used for fishing, hunting, trapping, boating, and other similar activity.

5. Primary contact recreation involves direct human body contact with the raw water to the point of complete body submergence.

TABLE III-4
STATE OF ARIZONA
EFFLUENT REUSE REQUIREMENTS

PARAMETER	Agricultural Reuse			Impoundments**		Turf Irrigation	
	Restricted	Partially Restricted	Unrestricted	Full Body Contact	Partial Body Contact	Restricted	Unrestricted
BOD mg/1	*	*	10	10	*	*	10
SS mg/1	*	*	10	10	*	*	10
Fecal Coliform #100 ml	N /A	1,000	200	200	1,000	1,000	200

* Secondary level of treatment required - concentrations not defined.

** Where effluent provides substantial portion of water supply to impoundment.

The water may be ingested accidentally and certain sensitive body organs, such as the eyes, ears, and nose, may be exposed to the water. Although the water may be ingested accidentally, it is not intended to be used as a potable supply. This water may be used for swimming, water skiing, skin diving, and other similar activities.

6. Restricted turf irrigation involves irrigation of turf areas at golf course, cemeteries and similar areas where children are not expected to congregate and play.

7. Unrestricted turf irrigation of school grounds, playgrounds, lawns, parks or any other areas where children are expected to congregate or play.

There is one additional reuse alternative, industrial use, which does not have specific quality standards. This alternative is considered by the Arizona Department of Health Services on a case by case basis because the variety of uses is so extensive that establishing specific criteria governing all uses is not possible. In fixing treatment requirements and quality criteria, the Arizona Department of Health Services considerations include, as a minimum, degree of potential contact with the reclaimed wastes by the general public and the degree of potential contamination of the products or by-products being produced or handled in the industrial process.

Based on established standards and discussions with ADHS and EPA, effluent reuse standards were proposed for use the Phoenix Urban Study. These standards, shown in Table III-5 are preliminary in nature and have not been adopted by the State Water Quality Control Council.

In the final areawide alternative analysis, it was determined by the Arizona Department of Health Services that lagoon (photo-synthetic or aerated) effluent meets the State standards for restricted agricultural irrigation. Further, the ADHS determined that disinfected lagoon effluent could be utilized for partially restricted agricultural irrigation on processed food crops such as grains and sugar beets.

The detailed analyses of reuse options at the subregional level, identified as potential reuse demand of over 600 mgd in the eastside and 300 mgd on the westside of the Urban Study Area. Agricultural reuse appeared to be an attractive option and as a result a total of 15 agricultural areas were identified as potential users in the study area.

Basically, all of the agricultural reuses were acceptable, however, the additional costs for advanced waste treatment suitable for unrestricted agriculture made it uneconomical. Therefore, in the final areawide

TABLE III-5
PROPOSED EFFLUENT REUSE STANDARDS

Reuse Options	Standards[2] (Existing & Proposed) BOD	SS	P	Fecal	Treatment Level
Agricultural					
Restricted	30	30	-	-	Secondary
Partially Restricted	30	30	-	1000	Secondary + Partial Disinfection
Unrestricted	10	10	-	200	Advanced Waste Treatment I (AWT I-Secondary + Filtration + Disinfection)
Recreational Lakes					
Partial Body Contact[1]	10	10	0.15	200	Advanced Waste Treatment III (AWT III phosphorous and nitrogen removal)
Full Body Contact	5	5	0.15	2.3	Advanced Treatment III (AWT III)
Municipal					
Golf Course	30	30	-	1000	Secondary + Partial Disinfection
Parks	10	10	-	200	Advanced Waste Treatment I
Groundwater Recharge[1] (low)	30	30	-	200	Secondary + Disinfection
Groundwater Recharge[1] (high)	5	5	-	-	AWT II (Filtration + Organic + Organic Removal)
Industrial					
Nuclear Cooling Water	30	30	-	-	Secondary

1. Proposed Standard
2. mg/l except for Fecal No./100ml

alternatives only reuse alternatives using effluent from a treatment secondary plus disinfection system were considered. An exception to this was reuse on the Salt River Indian Community where exchange for fresh water was still a viable option. Recreation and direct recharge with effluent were eliminated because of high costs and potential health hazards.

Residual Solids Management

Sludge is the residual solid which remains after sewage has been treated. The management of sludge is an important part of the waste treatment process because it requires looking at ways to handle, treat and dispose of the sludge. Disposal is extremely important since sludge can contain heavy metals, virus and other materials dangerous to health. The amount to be disposed also poses a problem.

Sludge management options available for the 208 areawide alternatives for the Phoenix area were investigated. Digested sludge production for the year 2000 was estimated at 139 tons/day. For each one million gallons of raw sewage, approximately 1,700 pounds of sludge are produced. Options for sludge disposal and treatment processes were examined for the proposed facilities, as were costs associated with each option.

Sludge Disposal: The options for sludge disposal were examined prior to treatment processes because the form of disposal has a direct influence on the process used. There are four possibilities for the disposal of sludge: water disposal, incineration, land application or landfill.

Water disposal was rejected immediately as a possibility because of the limited water supplies and because of health impacts on the Phoenix area. Incineration reduces sludge to a sterile ash. The release of particulates and gases into the atmosphere, however, would create air quality problems. Imposed stringent standards on discharge into the air would seem most likely under this option, making costs rise substantially. Although incineration remains an option, its benefits are minimal compared to costs.

Landfilling is the simplest method for disposing of sludge because of controls and proximity of landfill sites to proposed wastewater treatment plants. By the year 2000, the annual landfill volume required for sludge disposal would be 215,000 cubic yards. Between 1980 and 2000, 32,598,000 cubic yards would have been used for sludge landfilling. The principal problems involved with this form of disposal were threefold. First, the volume of sludge produced by the year 2000 would account for approximately 15 percent of all solid wastes, thereby reducing the useful lives of sanitary landfills by about 10 percent. Secondly, the impacts of sludge landfilling on groundwater quality were unknown. The third problem with this form of disposal resulted from the fact that sludge is considered as waste material rather than as a potential resource.

Land application of sludge to agricultural or reclaimed lands appeared to be a viable option of sludge disposal. The disposal of combined sludge types would affect between 3-7 percent of all agricultural lands by the year 2000. Approximately 26,700 acres would be required for land application. The closeness of agricultural lands to proposed wastewater treatment plants made this option attractive on the basis of cost and limited environmental impact. Several problems needed to be examined for each facility and it was determined that this would best be accomplished under 201 facilities planning. Soil conditions, crop selection, the presence of heavy metals in the sludge or soil, and application rates were all of serious concern. Nickel was identified as the most limiting metal in the Phoenix area (allowing an application rate based upon EPA standards of only 1.9 tons/acre/year). The application rate could be increased, but this would decrease the site life of the land area.

Nitrogen was identified as the major nutrient acting as a limiting factor in sludge disposal. This problem could be mitigated by not applying sludge at a rate to exceed the nitrogen uptake capacity of the crops. An average 1.5 tons/acre annually was estimated as a suitable application rate. This could be increased depending on the sites chosen for land application. Specific site tests and pilot projects are necessary to determine actual field application rates. The necessity of negotiating several contracts with farmers was another disadvantage associated with land application of sludge. Land application was determined to have the potential for being a practical and cost-effective method of disposal.

Sludge Treatment: A review was made of the many unit processes available to stabilize and reduce the volume of sludge for disposal. Based upon preliminary analysis the processes most applicable to the study area were selected for further analysis and are summarized in Table III-6.

Figure III-1 shows the sludge processing alternatives for new facilities. These alternatives were analyzed for three different types of sludges: primary, secondary, and tertiary. From a cost viewpoint, anaerobic digestion with liquid land spreading proved to be the most economical.

Additional Management Considerations: Location of sludge treatment was examined in addition to disposal and treatment processes. Local, regional, and combined sludge management options were considered

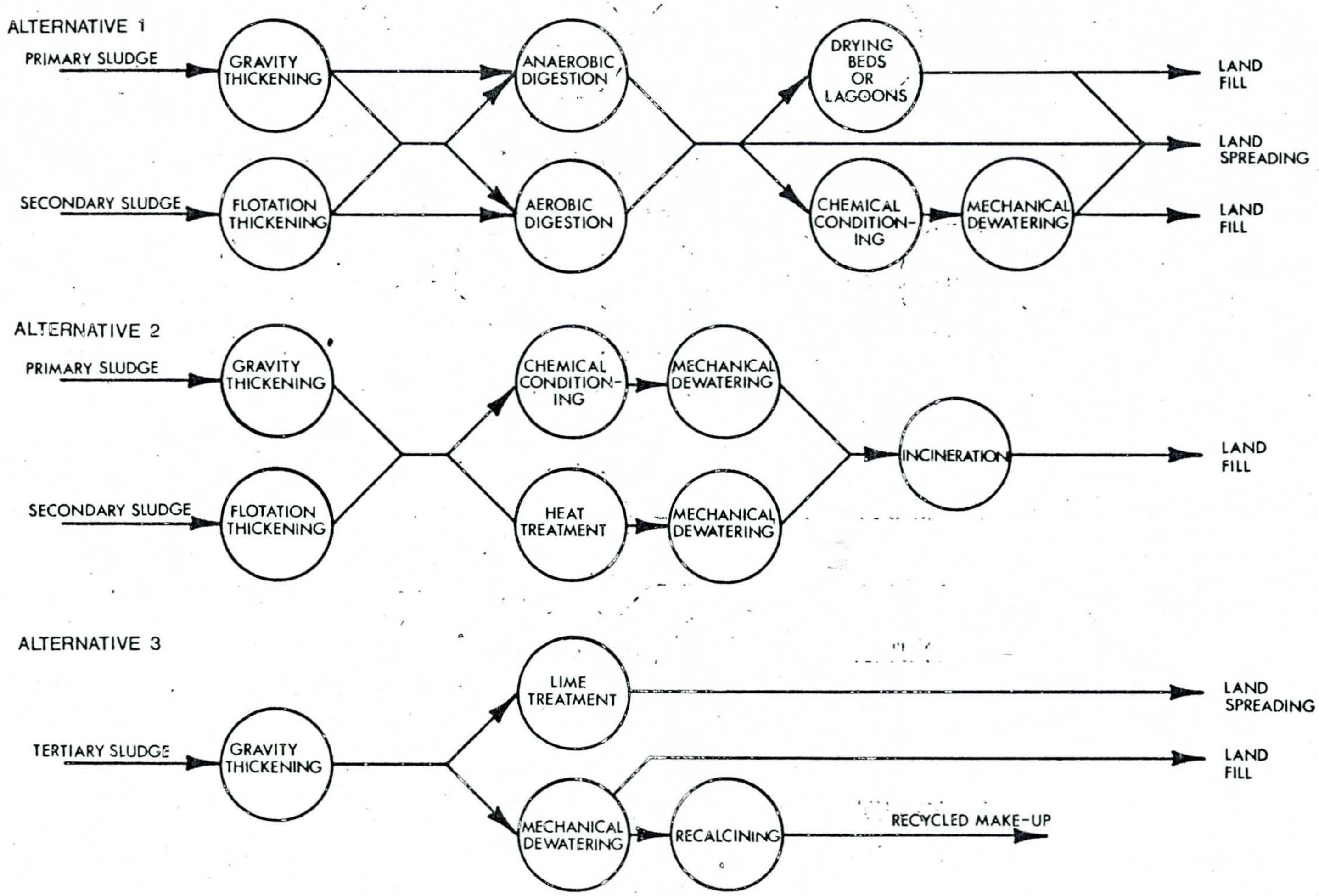


FIGURE III - 1
SLUDGE PROCESSING ALTERNATIVES

TABLE III-6

SUMMARY OF SLUDGE PROCESSING ALTERNATIVES SELECTED FOR FURTHER ANALYSIS

Sludge Treatment Process	Sludge Type: New Facility				Expansion of Existing Facility		
	Primary	Secondary	Combined	Tertiary	Primary	Secondary	Combined
1. Thickening							
a. Gravity	X			X	X		
b. Flotation		X				X	
c. Centrifugal							
2. Stabilization							
a. Chlorine Oxidation							
b. Lime Treatment				X			
c. Aerobic Digestion	X	X	X		X	X	X
d. Anaerobic Digestion	X		X		X		X
3. Conditioning							
a. Elutriation							
b. Heat Treatment			X				X
c. Chemical	X	X	X		X	X	X
4. Dewatering							
a. Drying Lagoons	X	X	X		X	X	X
b. Drying Beds	X	X	X		X	X	X
c. Mechanical	X	X	X	X	X	X	X
5. Conversion/ Reduction							
a. Heat Drying							
b. Composting		*	*			*	*
c. Incineration	X		X		X		X

* Can only be considered further after feasibility demonstration
X = Selected alternative

for primary and secondary sludge. Tertiary sludge would be a local management problem because it is produced only at advanced treatment plants.

Cost, proximity to development and disposal sites, and reliability all were studied. Based on the expansions which would be required by plants and the above criteria it was determined that total regionalization of sludge management would be a poor choice. Regional treatment at 91st Avenue would cost approximately 10-15 percent more than local treatment.

Areawide Sludge Alternatives: Sludge from a wastewater treatment plant, when digested and dried can be utilized as fertilizer. Treatment of the sludge to obtain a suitable fertilizer does not usually involve advanced or expensive processes. The sludge is usually stabilized by digestion and then air dried, drum flash dried, or simply placed in sand beds and dried by evaporation and percolation. Final preparation of the dried sludge such as shredding, windrow composting, or heat drying increases its ease of handling and transport, as well as its value.

A million gallons of wastewater will yield approximately 700 cu. ft. of wet sludge (95 percent moisture). Reuse of this sludge as fertilizer is a financial benefit to the wastewater treatment plant in that the cost of sludge handling, transport, and ultimate disposal can be saved and the dry sludge can be sold.

Low-cost bulk sludge fertilizer should be low in ash, free from pathogens, weed seeds, and odor. It should be uniform in texture and relatively dry, but not dusty. Anaerobically digested sludge contains 20 to 30 percent less nitrogen than does fresh sludge. Anaerobic sludge digestion processes cause this loss of nitrogen. The activated sludge process tends to concentrate the nitrogen from the wastewater and tie it up in the sludge as bodies of micro organisms. In the use of open drying beds and stockpiles, there is a 25 to 50 percent loss of nitrogen, phosphorous, and organic compounds, depending on the circumstances. Some of these losses are the result of rainfall leaching, while others are due to additional anaerobic digestion and bacterial action.

The City of Phoenix and participants in the Multi-City Agreement executed an agreement dated 9 July 1974 with Kellog Supply Inc., whereby Kellogg purchases and removes all sludge produced at the 23rd Avenue and 91st Avenue treatment plants. The sludge is transported by the purchaser out of the state for processing. The purchaser also has a location at 87th Avenue and Southern for stockpiling of sludge for future on-site processing. The agreement is for a period of 10 years. Payment is made monthly on the basis of $0.25/cu.yd. of air dried digested sludge. The sludge must contain no more than 25 percent moisture.

The cities retained the option to keep and use up to 6300 cu.yd. of sludge per year. If the municipalities do not exercise their options, then Kellogg must remove and pay for the sludge.

If performed by the City of Phoenix, the labor and other costs associated with the loading and transporting of sludge and the preparation of sludge drying beds was determined in 1975 to be approximately $3.50/cu.yd. During 1975, the Kellogg Supply Inc. removed a total of 15,675 cu.yd. of sludge from area treatment plants, thereby saving the City of Phoenix and other communities almost $55,000. Additionally, the cities were paid $3,918 for the sludge, bringing the total benefit for the year to almost $59,000.

Composting, if done properly can obtain a total kill of pathogens and weed seeds. This requires a minimum of 24 hours at a minimum temperature of 160°F. In the case of outdoor windrowing, care must be taken to insure that the bottom and outer edges of the windrowed pile reach the desired temperature.

There are several environmental effects that must be considered in the reuse of sludge for fertilizer production. In the removal and transport of dried sludge from the sludge beds, care must be exercised to insure the control of dust. There also is the possibility that odors would develop if the sludge material is stored when moisture is still present. Finally, as sludges are known to contain pathogenic organisms, heavy metals, and other toxic materials, fertilizers manufactured from treatment plant sludges must be analyzed for these items. The subsequent use of these fertilizers should be controlled in relation to their content of toxic materials.

The final determination of the sludge handling and disposal system for 91st and 23rd Avenue plants will be made after due consideration of the impacts of the industrial pretreatment requirements to remove heavy metals and other detrimental materials and in the impacts of the sludge in landfills and agricultural land.

The existing Tolleson plant will continue to use its sludge on the sod farm.

Treatment and Disposal Processess

Conventional Treatment: In the earlier phases of the 208 study, four levels of conventional treatment were selected to cover the range of treatment most likely to be required in the study area. These treatment levels were selected based on effluent requirements for existing or proposed reuse alternative or possible disposal methods identified for metropolitan Phoenix. The first level of treatment considered was primary treatment which would be used in conjunction with a land treatment system to provide an effluent which meets the EPA requirements for secondary treatment. The second level was secondary treatment which meets the EPA requirements for BOD and SS removals, and will give a 30-30 BOD/SS effluent quality. The typical secondary treatment process used to develop the cost curves consisted of screening and grit removal, primary sedimentation, activated sludge, sludge processing by anaerobic digestion and vacuum filtration, and disinfection. The next level considered was tertiary treatment (Tertiary I), which consisted of the above secondary treatment process plus two-stage lime clarification and multi-media filtration. This process will result in a "5-5" BOD/SS effluent and a 90-95 percent phosphorus removal. The fourth level of treatment (Tertiary II) evaluated for costs consists of carbon absorption and ammonia stripping in addition to secondary, lime clarification and filtration. This process will produce "1-1" BOD/SS effluent plus 90-95 percent phosphorous removal and 85-98 percent ammonia removal. Effluent from this process would bve suitable for reuse in full-body contact recreational lakes.

These conventional treatment levels were selected following a review of the existing treatment systems in the Phoenix area, the Arizona Department of Health Service's requirements concerning wastewater disposal or reuse, and the wastewater disposal or reuse alternatives which were proposed or identified in the Phoenix area.

As the 208 study progressed and the number of areawide alternatives decreased, however, it was decided that alternative processes should be considered. This decision was made in part because most of the proposed plants were small (less than 6 mgd) or existing facilities had systems which could be expanded. The Arizona Department of Health Service requirements for effluent reuse for restricted agriculture also were clarified and this allowed consideration of sewage lagoons as an acceptable treatment system.

Based on a review of possible treatment methods and discussions with the ADHS staff, the systems selected for the final set of alternatives were either new lagoons with mechanical aeration followed by stabilization ponds with disinfection, or expansion of an existing process. The effluent quality from the lagoon process would have BOD less than or equal to 30 mg/1, suspended solids less than or equal to 135 mg/1 and would meet State of Arizona requirements for irrigation of restricted crops such as fiber, forage, or orchard crops. With chlorination, the effluent also would be suitable for use on processed food crops such as grains or sugar beets.

The proposed lagoon systems were designed in accordance with the most recent ADHS Design Bulletin Number 11. The specific design criteria are:

Aerated lagoon detention time	- 10 days
Stabilized lagoon detention time	- 10 days

Aerated lagoon depth	- 10 feet
Stabilization lagoon depth	- 3-5 feet
Chlorine dosage	- 10 mg/1
Effluent Quality - BOD	- 30 mg/1
SS	- 90-135 mg/1

In making the decision to go with the lagoon system, certain factors had to be taken into account. One of these was the EPA stipulation that, except for plants of 2.0 mgd or less, all effluent to be discharged to the nation's waters must have a minimum quality of BOD and Suspended Solids less than or equal to 30 mg/1. This meant that the effluent from lagoon systems larger than 2.0 mgd could not be discharged to any of the rivers in the study area (Salt or Gila) or dry washes or irrigation canals which are tributary to the rivers. All plants greater than 2.0 mgd, therefore, had to have specific reuse or disposal options identified.

Land Treatment: In addition to conventional treatment, land treatment plus recovery processes received evaluation at the existing and proposed treatment plant sites throughout the Urban Study. The two land treatment methods which received the most intensive evaluation were infiltration/percolation and overland flow.

Infiltration-percolation or rapid infiltration is a land treatment system in which wastewater is percolated through the soil in a controlled and engineered system and employs recovery of nearly all of the applied water. Normally, partially treated wastewater is applied to the soil in spreading or percolation basins and wastewater renovation is achieved as the effluent travels through the soil matrix by natural physical, chemical, and biological processes. The renovated water is then recovered using wells or underdrain systems and can be reused for unrestricted agricultural irrigation purposes.

Overland flow is essentially a biological treatment process in which wastewater is applied over the upper reaches of sloped terraces and allowed to flow across the vegetated surface to runoff collection ditches. Renovation is accomplished by physical, chemical, and biological means as the wastewater flows in a thin sheet down the relatively impervious slope. Runoff from overland flow systems can be collected at the toe of the slope in cutoff ditches and can be reused for agricultural irrigation.

The land treatment systems were designed on criteria published by the EPA in the document entitled "Land Treatment of Muncipal Wastewater Effluents - Design Fators II," January, 1976. Local research conducted at the 91st and 23rd Avenue treatment plants provided information which was used to update and "localize" design factors developed by the EPA.

The three basic loading rates were considered for land treatment systems in the Phoenix area are: a) hydraulic, b) organic, and c) nutrient (nitrogen and phosphorus).

The hydraulic loading rate depends on site soil characteristics and upon the particular land application method. Infiltration-percolation hydraulic loading rates are dependent on soil percolation values. It should be noted, however, that the actually obtainable rate of application of effluent is much less than the percolation rate of soil. This is the result of soil clogging which occurs over time and the fact that percolation tests are conducted with clear water which will percolate much faster than typical wastewater or effluent. In addition, the infiltration beds must be allowed to dry periodically to let accumulated solids decompose thus restoring hydraulic capacity to the site. Water losses in overland flow systems are the result of plant evapotranspiration and percolation.

Maximum hydraulic loading rates for each treatment method are given in Table III-7 and these are the rates used in computer analyses of the land treatment systems.

TABLE III-7

Application Method	Maximum Loading Rate Feet/Year
Overland Flow	40
Infiltration/Percolation*	350

*Includes drying periods

Organic loading rates are the rates of application of biochemical oxygen demand (BOD) and suspended solids (SS) to the soil. The addition of these elements influences the rate of oxygen diffusion in the soil and thus the existence or non-existence of aerobic conditions in the soil. As the presence of oxygen is necessary to bring about the decomposition of organic matter in wastewater, limits must be applied to the rate of application of BOD and SS.

BOD and SS are removed to a very high degree by land treatment of the applied wastewater. When applied to the land at acceptable loading rates, almost complete removal can be expected.

Acceptable organic loading rates depend upon the type of soil, the drying period, and the temperature. The permissible loading rates used in analyzing land treatment alternatives are given in Table III-8. These rates were obtained from EPA and other literature.

TABLE III-8

PERMISSIBLE BOD LOADING RATES

Method	BOD Loading Rate
Overland Flow	25,000
Infiltration	25,000

Nutrient loading rates are concerned with the addition of nitrogen and phosphorus to the soil. Applied nitrogen may be removed by a combination of five means:

1. crop uptake
2. denitrification
3. volatilization
4. addition to ground or surface water, and
5. storage within the soil

Denitrification is important in overland flow systems and is the only significant means of nitrogen removal in infiltration percolation systems.

Phosphorus is removed from the applied effluent by the orthophosphate (PO_4) reacting with metals in the soil to form metal salts. The most common reacting metals are usually iron, aluminum, and calcium. The resulting metal phsophate is very soluble. The phosphates are immobilized in the soil by a combination of adsorption, fixation, and precipitation. Crop uptake also results in the removal of phosphorus from the soil.

Nutrient loading rates were established to optimize the various natural removal mechanisms of the crops and soil. Table III-9 shows the nutrient loading rates used in the final array for the various land treatment methods.

TABLE III-9

PERMISSIBLE NUTRIENT LOADINGS

Application Method	Loading Rate Nitrogen	lb/acre/yr Phosphorus
Overland Flow	500	150
Infiltration/Percolation	22,000	10,000

In land treatment systems, efficiency is a variable and is dependent on numerous factors including soil type, climate, type of crops, and wastewater quality. In contrast to conventional tratment systems, pollutant removal efficiencies cannot be expressed as absolute expected values. At best, land treatment systems can be expected to perform through a range of treatment efficiencies. Research by the EPA (cited in Table III-10) has established the removal efficiency ranges for properly managed land treatment sites using proper loading rates as previously described in this report.

TABLE III-10

EXPECTED TREATMENT PERFORMANCE

Application Method	BOD	Nitrogen	Phosphorus
Overland Flow	92+%	70-90%	40-80%
Infiltration/Percolation	90-99%	20-40%	70-99%

Land treatment sites receiving primary or secondary effluent may sometimes have an odor problem. For these reasons, it is desirable to establish a "buffer zone" around a land treatment site to reduce the potential of odors reaching populated areas. Recommendations in the literature for buffer zones range from 200 feet to 1/4 mile. Exact requirements will vary from site to site and will depend on climate, location, prevailing wind direction, treatment method and proximity to population centers. For the 208 analysis, buffer zones were assumed to be 200 feet wide on three sides of each land treatment site, with the fourth side (prevailing wind side) having a 500 foot wide zone.

Overland flow systems must be provided with storage facilities to allow the site to be shut down periodically. Infiltration/percolation sites are designed with more than one bed to allow alternate drying periods. Additional storage, therefore, is not required. Overland flow systems require only minimal storage to allow for occasional site maintenance.

For the 208 alternative analyses the following storage requirements were assumed:

Treatment Type	Days Storage
Overland Flow	1
Infiltration/Percolation	0

Land treatment techniques and sites were reviewed by advisory groups, which determined that for environmental, socio-economic, and groundwater reasons, most of the land treatment options should be abandoned as part of 208 planning. At the time the final areawide plans were being evaluated, only seven land treatment alternative sites were still viable. These were located at Chandler, Gilbert, Williams AFB, 23rd Avenue, 91st Avenue, and Northeast Scottsdale, and the proposed Reems Rd. site. During the final selection process, however, MAG planners and advisory groups indicated a preference for conventional treatment over land treatment alternatives. This decision was based on capital and annual costs for:

- o treatment facilities
- o transmission systems
- o site clearing
- o distribution systems
- o recovery systems
- o service roads
- o additional fencing.

The possible effect of land treatment on groundwater quality, particularly at the Northeast Scottsdale site was viewed as a problem. This influenced the decision to eliminate the Northeast Scottsdale site with its land treatment option from the final selected plan.

It also was determined that adoption of land treatment alternatives would require pilot projects, thereby adding considerable time and expense to the implementation of the final 208 Areawide Water Quality Plan.

Results of recent studies by the Corps of Engineers, however, have shown land treatment to be a cost effective wastewater renovation process which poses no greater health or environmental hazards than any conventional treatment method. Land treatment, therefore, will receive further examination as part of 201 facility planning in the Phoenix metropolitan area. This 201 planning effort will be carried out by local agencies.

Irrigation and Disposal: In conjunction with conventional or land treatment systems, irrigation/disposal systems were evaluated at several sites. While these systems provide additional treatment, they differ conceptually from land treatment and recovery processes in an important way. In land treatment systems, the primary objective is to treat the wastewater and recover the renovated water for some beneficial reuse. Irrigation/disposal systems, as analyzed in this study, have been evaluated simply as a method to dispose of the treated wastewater and not to recycle or recover any renovated water Irrigation/disposal systems have been evaluated at the following sites: Northeast, Gilbert-North and South, Chandler, and Reems Road. The development of criteria for the irrigation systems closely follows accepted practices as presented by the EPA in the recent design manual for land treatment systems. More site specific information has been developed from information obtained from the University of Arizona Agricultural Extension Service. The following sections present specific design criteria and information which was used to develop the storage and irrigation disposal or effluent reuse systems for each alternative. They also identify some of the problems to be considered in their implementation.

The general benefits of irrigation or land disposal include the following:

1. No discharge to receiving streams which results in improved areawide water quality;

2. Potential increased Federal funding for the system;

3. Possible improvements in groundwater quality and quantity through recharge depending on the location of the system;

4. Local reuse of effluent which may result in local economic benefits from crop production or improved groundwater conditions.

Cropping Patterns: In September, 1978, a report was issued as a part of the Phoenix Urban Study entitled, "Wastewater Irrigation/Disposal Sites." This report presented various cropping patterns utilized in the Phoenix area. These patterns included various rotations of cotton, alfalfa, and small grains. Further discussions with various individuals also suggested the use of so-called "permanent pasture" as a possible means of disposing of wastewater. In order to utilize commonly grown crops and to maximize the disposal aspect of the wastewater application, the cropping patterns in Table III-11 were investigated.

With cropping Pattern I it is expected that the total area would be divided into six equal fields: 2-cotton, 3-alfalfa, and one small grains. These cropping patterns would be rotated yearly. Cropping Pattern II would have eight fields: 2-cotton, 3-alfalfa, and 3-small grains. Cropping Pattern III would require multiple fields to allow some fields to be irrigated while some are drying and others are being grazed. Grazing could be a year-round operation with cattle being pastured from May through August and sheep from October through April, with the month of September used for tilling and reseeding if necessary.

Consumptive Use: In order to design an irrigation system, it is necessary to know how much water to apply, when to apply it, and what effect it has on the crop when it is applied. This information was extracted from a technical bulletin (No. 169) of the Agricultural Experiment Station of the University of Arizona entitled "Consumptive Use of Water by Crops in Arizona." The consumptive use is defined as the unit amount of water used on a given area in transpiration, building of plant tissues, and evaporation from adjacent soil. For the purposes of the following analysis, consumptive uses, as listed in Table III-12 have been employed.

TABLE III-11

CROPPING PATTERNS

Pattern I	
2 years	cotton
3 years	alfalfa
1 year	small grains (wheat and sorghum)
Pattern II	
2 years	cotton
3 years	alfalfa
3 years	small grains (wheat and sorghum)
Pattern III	
Bermuda	(April - September)
Rye	(October - March)

Nutrient Uptake: Many factors must be considered in the planning, analysis, and design of a wastewater irrigation system. One of these factors is the fate of certain constituents in the wastewater, especially nitrogen and phosphorus. Because the nutrients are essential to plant growth, the plants utilize these constituents and remove them from the soil system. Groundwater contamination may take place as the result of over-application of these constituents. Crop yield increases ranging up to two to fourfold have been achieved when wastewater effluent is used to irrigate. Typical uptake rates are listed in Table III-13.

TABLE III-12

CONSUMPTIVE USE

Cotton		3.43 ft/year
Alfalfa		6.19 ft/year
Wheat		1.92 ft/year
Bermuda & Rye Grass	Summer	2.5 inches/week
	Winter *	0.5 inches/week

*December - March

In addition to the crop uptake, it has also been assumed that 20 percent of the applied nitrogen will be lost to denitrification.

Wastewater normally contains approximately 40 ppm of total nitrogen (expressed as N). In order to satisfy the plant's water needs, more nitrogen than can be utilized by the plants will be applied if wastewater is used solely for irrigation. Thus, leaching of nitrogen to the groundwater can occur, resulting in groundwater quality degradation. If effluent is mixed with groundwater which contains little or no nitrogen, the wastewater is diluted and the amount of nitrogen applied to the farming operation can be balanced with the plant's needs to mitigate leaching. To reduce the potential of nitrogen leaching into the groundwater, wastewater should be blended with groundwater in the following ratios:

Cropping Pattern	Wastewater: Groundwater
I	1.03 : 1
II	1.19 : 1

In the case of the "permanent pasture," supplying the consumptive use of the crop does not supply sufficient nitrogen. Therefore, wastewater is over-applied up to the limit as determined by the nitrogen uptake.

Although nitrogen application has been limited to uptake values, crops do not utilize it at a uniform rate. Many factors influence this uptake rate including stage of plant growth, ambient temperature and moisture conditions, and nitrogen form. It is possible, therefore, that at certain times of the year excess nitrogen will be applied and may be leached into the groundwater. Much more detailed studies are required to quantify this potential problem. The proposed systems will have to be studied in detail on a site-specific basis in the 201 facility plans.

Another problem requiring additional research is the possibility of nitrogen toxicity developing in grazing cattle as the result of the high uptake values of the bermuda. This appears to be a management problem and could be overcome.

Phosphorus is not a problem. In any of the alternatives, it would take in excess of 20 years to saturate the upper three feet of soil, assuming the level of 10 mg/l P continues in the treated effluent.

TABLE III-13

NUTRIENT UPTAKE RATES
(lbs/acre/year)

	Range		Design Value	
Crop	Nitrogen	Phosphorous	Nitrogen	Phosphorus
Cotton	66-100	15	100	15
Alfalfa*	155-480	20-35	400	35
Wheat	50- 81	15-29	75	30
Sorghum	250	40	250	40
Bermuda	350-600	30-40	600	40
Rye	180-250	55-75	250	75

*Legumes also take nitrogen from atmosphere.

Irrigation Requirements: The analysis of water requirements also necessitates the evaluation of the efficiency of conveying the irrigation water to the fields. Irrigation efficiencies for ditch and furrow application range from as low as 50 percent to as high as 70 percent depending on soil type, time of application, and physical condition of conveyance facilities. If water is transported via unlined canals and allowed to flow onto adjacent border areas, resulting efficiency is low. If concrete-lined canals or pipes are used and only the cultivated area is wetted at appropriate times, however, efficiency can be high. Because of the general nature of the soil in the Phoenix area and the high ambient temperatures which result in high evaporation rates, a typical area irrigation efficiency of 60 percent was used in this study.

TABLE III-14

IRRIGATION REQUIREMENTS

Cotton	6.8 ft/year
Alfalfa	12.8 ft/year
Wheat	3.8 ft/year
Sorghum	4.6 ft/year
Cropping Pattern I	9.8 ft/year
Cropping Pattern II	9.4 ft/year
Cropping Pattern III	7.2 ft/year

Another factor considered when evaluating water requirements is the amount of water required for leaching of accumulated salts from the root zone. To take a conservative approach, an additional 10 percent leaching requirement has been used in this study, resulting in an overall irrigation efficiency of 50 percent. Actual irrigation requirements are listed in Table III-14.

It should be pointed out that leaching of salts from the root zone may have either an adverse or positive impact on groundwater quality depending on the location. Salts leached out of the root zone will travel further into the soil structure and may eventually reach the water table. If the existing groundwater is high in salts, however, the concentration in irrigation water reaching the water table may be less than the existing groundwater and dilution will occur. This condition has occurred in the Buckeye area and has resulted in improvement of the groundwater quality. On the other hand, leaching of salts in other areas such as Gilbert may add salts to the groundwater an degrade the quality.

Storage: Irrigation requirements, although given in quantities per year, fluctuate during the growing season. Each of the various crops grown in the three cropping patterns have their own monthly water requirements.

TABLE III-15

MONTHLY WATER REQUIREMENTS*
(feet/month)

Month	Cotton	Alfalfa	Wheat	Sorghum
January			0.8	
February		0.8		
March		1.5	1.2	
April	1.5	1.5	1.2	
May	0.8	1.5	0.6	
June		1.5		1.5
July	1.5	1.5		0.8
August	1.5	1.5		1.5
September	1.5	1.5		0.8
October		1.5		
November				
December				
TOTAL	6.8	12.8	3.8	4.6

*Including irrigation efficiency of 50%.

Table III-15 presents the various water requirements by month and by crop. These requirements vary with the plant cycle and time of year and fluctuate depending on antecedent moisture conditions. Water requirements for these systems vary substantially between summer and winter while wastewater flows are fairly constant year-round. In order to assure adequate water during the growing season as well as some method for accommodating wastewater flows during harvest or low water demand periods, an analysis has been prepared for each of the cropping patterns resulting in approximately a 3 month storage requirement for Patterns I and II. Wastewater can be applied continually to Pattern III, although a two-week storage facility is desirable to allow for convenience of operation.

Miscellaneous Requirements: An analysis of irrigation requirements for wastewater application results in gross acreages that are to be used for cropping. An operating farm, however, requires additional area for such things as fencing, equipment storage and field borders. After a brief analysis, it has been assumed that a 5 percent allowance for this acreage would be sufficient. An additional amount of area also must be set aside for the storage lagoons. This area has been calculated by assuming a fifteen foot deep reservoir and added on to the adjusted acreage. Total acreage, therefore, represents the land needed for crop requirements, operational allowance, and storage lagoons.

Process Summary

Presented in Table III-16 are the various treatment processes which were considered at each proposed or existing treatment plant site. Table III-17 lists the various intended effluent reuse and/or disposal methods and the required effluent quality at each plant site. At the Reems Road WWTP, two processes were considered. One was the lagoon system with disinfection and the other a system consisting of an aerated lagoon followed by an infiltration-percolation land treatment and recovery system.

For the northeast plant two processes were evaluated. One was a conventional activated sludge process followed by filtration while the other consisted of an aerated lagoon system followed by "overland flow" land treatment. These treatment levels are necessary for the proposed unrestricted agriculture reuse on the Salt River Indian Community lands.

Interceptor Sewers

Sewered Areas

In the portions of the Urban Study area which were sewered, the analysis of interceptor sewer needs, including sizes, routes, and staging was relatively straightforward. In each case, a comparison of total or owned capacities in existing interceptors against projected flows was made, and interceptor needs identified. These comparisons ar various points in the existing system were used to identify not only where, but also when, and how much capacity will be needed to accommodate the service areas through the year 2000. From these conclusions, the required interceptor layouts and sizes in sewered areas were developed. A Manning's "n" factor of 0.013 was utilized in all calculations of existing capacity and new interceptor sizes.

Unsewered Areas

The procedure which was used to identify interceptor sewer needs, routings, sizes, and staging in areas presently unsewered involved analyses of population density, flow projections, and natural drainage in each area. The population density and flow projections were based on the CAPM population totals adopted by MAG and utilized in the 208 study.

The basic approach in the analysis involved the following steps:

1. Identification of unsewered areas;

2. Estimation of total developable acres in each area;

3. Calculation of year 2000 average population density;

4. Identification of total acres presently developed;

5. Calculation of present population density in developed areas;

6. Calculation of projected year 2000 peak flows;

7. Development of proposed sewer routings based on natural drainage and existing community sewerage plans;

8. Sizing of required interceptor sewers (minimum size at 10 inch diameter).

For the 208 analysis the following criteria were used to determine if an area would be sewered:

1. A density of 1.5 persons per acre or greater was required within the entire service area, or;

2. A density of 1.5 persons or greater was required within a minimum one-square mile area, which is presently developed in the service area.

It should be noted that these criteria are assumptions used for planning purposes. They are based on engineering judgement and experience in the Phoenix area and meant only to be guidelines for planning. Decisions concerning areas to be sewered and areas to be accommodated by individual systems can not be made until detailed studies of soil conditions, depth to groundwater, environmental impacts, and costs have been completed.

These planning criteria can be justified as reasonable from the EPA's guidelines for analysis of individual systems versus community sewering which include a substantial human habitation criteria of 1.7 people per acre or 1 household per 2 acres (See "Funding of Sewage Collection System Projects," EPA Program Requirements Memorandum #77-8, August, 1977). According to the EPA memorandum, densities of less than one household for every two acres rarely result in serious localized pollution or public health problems from the use or properly operated on-site systems. At densities greater than one household for every 2 acres, however, an analysis should be done to determine the cost-effectiveness and environmental impacts of various types of individual systems and community sewerage.

Both of the criteria used in the 208 are conservative and were used only to identify areas in which it is reasonble to assume that a sewer system may be more cost-effective, or may eliminate possible localized pollution or public health problems resulting from human habitation.

COST CRITERIA

As described earlier in this chapter, the level of detail involved in the alternative analysis increased as the areawide plans were refined and reduced in number. At the conceptual array stage, many alternatives were involved and the criteria including design requirements, costs, and evaluations were fairly general. These criteria were utilized to provide a common basis for comparison of the alternatives at a level of detail sufficient to provide adequate data for selection or rejection of an alternative. As the Urban Study progressed and site specific data was developed, however, the analyses and evaluations became much more detailed. Also, decisions and clarifications of existing standards for reuse and treatment methodologies by the Arizona Department of Health Services during the study resulted in analysis and eventual selection of lagoon treatment of wastewater as a viable alternative for many of the smaller treatment facilities in the final areawide plans for the Phoenix area.

All capital cost estimates in the final alternative analysis were adjusted to an Engineering News Record Index of 2700. Ths index represents capital cost requirements for January, 1978, in the Phoenix area. All operation and maintenance costs were adjusted to January, 1978, levels by using actual Phoenix area unit costs for labor, power, and industrial commodities. The specific criteria used to adjust the operation-maintenance costs were as follows:

1. Labor Rate - $8 per hour for facilities accommodating flows greater than 10 mgd.

   - $9 per hour for all other facilities

2. Power Rate - $0.031 per kilowatt hour

3. Materials and Supplies - Adjusted to a Wholesale Price Index for the Industrial communities of 200.

All of these criteria and rates were developed following analyses and verification with the local community officials, power companies, and suppliers.

The final four areawide alternatives included treatment methods not previously addressed in the Urban Study. Also, since the final array of alternatives provided the basis for selection of a single alternative for implementation, it was felt that the level of detail required in the final cost analyses must be significantly greater than in previous evaluations. Detailed cost estimating procedures were developed for the final array as follows:

Lagoon Systems: The estimating procedure for costs, energy use, and land area requirements for the proposed lagoon systems in the final array was based on a preliminary design of each facility and local unit costs. The unit costs were developed in cooperation with the engineers for the City of Phoenix and were verified against recent construction bid tabulations and discussions with local suppliers. The estimating procedure also was reviewed and approved by representatives of the Arizona Department of Health Services and the Environmental Protection Agency. Table III-18 lists the various unit costs used in the lagoon system estimates.

Site specific preliminary designs for the existing or proposed lagoon facilities including the Northeast area, Chandler, Reems Road, Gilbert North, Gilbert South, Carefree-Cave Creek and Buckeye were prepared based on design criteria for lagoons described earlier in this chapter. Following these preliminary designs, capital and operation-maintenance costs were estimated for each system utilizing the unit costs shown in Table III-18 and the lagoon system estimating procedure forms shown in Tables III-19 and III-20.

TABLE III-18

COST CRITERIA - AERATED LAGOONS

Item	Unit Cost
Excavation, backfill, and compaction	$2.25/cu. yd.
Lagoon sealing or lining	$0.41/sq. ft.
Aeration equipment	lump sum estimate by local equipment supplier
Site fencing	$10.00/L.F.
Land acquisition	$4,500/acre
Service buildings	$100/sq. ft.
Chlorination building	$30/sq. ft.
Chlorination equipment	Lump sum per EPA cost curve in "Estimating Costs and Manpower Requirements For Conventional Wastewter Treatment Facilities," updated to ENR Index of 2700.
Site preparation and miscellaneous	20% of estimated construction cost.
Labor	$9.00 per hour
Electricity	$0.031 per KWH
Chlorine	$0.095 per lb.

TABLE III-19

PHOENIX URBAN STUDY
LAGOON SYSTEM ESTIMATING PROCEDURE

Capital Costs	Total Estimated Cost

1. Excavation, Backfill and Compaction (Balanced cut and Fill) ____________ cubic yards at \$2.25 per cubic yard

2. Lagoon Sealing or Linging (to top of berm) ____________ $ft.^2$ at \$0.41 per ft^2

3. Aeration Equipment (Including electrical equipment) ____________ HP units: Lump Sum per Equipment Supplier

4. Site Fencing ____________ ft. at \$10 per ft.

5. Land Acquisition (Does not include buffer zone) ____________ acres at \$4,500 per acre

6. Laboratory, Office, Shop Building (lab at 400 $ft.^2$ per ADHS Bulletin 11) ____________ ft^2 at \$100 per $ft.^2$

7. Chlorination Building (Per ASCE Design Manual 8) ____________ ft^2 at \$30 per ft^2

8. Chlorination Equipment and Contact Chamber

   Lump sum per EPA Cost Curve Number 66 in "Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities," March, 1973, updated to ENR Index of 2700.

Subtotal A ____________________

TABLE III-19

PHOENIX URBAN STUDY
LAGOON SYSTEM ESTIMATING PROCEDURE

Capital Costs	Total Estimated Cost
9. Site Preparation, Headworks, Yard Piping, Access Roadways, Electrical, Instrumentation Per 208 Assessment Manual, Appendix H	
20% of Subtotal A	________
10. Engineering, Legal, Fiscal, Administrative, and Contingencies Per 208 Assessment Manual, Appendix H	
30% of Estimated Construction Cost	________
Total Estimated Capital Cost (Estimated Construction Cost + Item 10)	________

TABLE III-19

PHOENIX URBAN STUDY
LAGOON ESTIMATING PROCEDURE

Operation and Maintenance Costs	Total Estimated Cost
1. Labor per EPA Manual "Estimating Staffing for Municipal Wastewater Treatment Facilities," March, 1973. (Staffing Curves D-1, D-2, D-3, D-4, D-13, D-32, D-33.) ____________ Hours at $9.00 per hour.	
2. Electricity __________ KWH per Aerator Manufacturer __________ Chlorination, Lighting and Misc. per Assessmen Manual, Appendix H Total _____ KWH per year at $0.031 per KWH	
3. Chemical (Chlorine at Average Dose of 10 mg/1) __________ Lbs. per year at $0.095 per lb.	
4. Materials and Supplies Lump Sum per 208 Assessment Manual, Appendix H, increased by wholesale price index ratio of 200/184.6	____________________
Total Estimated O & M Cost	

91st Avenue Treatment Plant Expansions: The final cost estimating procedures for the 91st Avenue Treatment Plant were developed in cooperation with the engineers for the City of Phoenix and the Arizona Department of Health Services. The cost procedures were selected following a detailed analysis of actual and projected operation and maintenance costs at 91st Avenue and a detailed capital cost estimate based on the design plans for the upcoming 30-mgd expansion at the plant. The estimated capital costs versus plant expansion design capacity at the 91st Avenue Plant, which was used for the final areawide plan cost estimate is as shown in Figure III-2. These capital costs include allowances for contingencies, legal requirements, fiscal requirements, and engineering.

The estimated annual operation and maintenance cost for the proposed expansions at 91st Avenue was selected to be $92 per million gallons of flow treated. This cost was developed from detailed study of required staffing, operation, and maintenance requirements for the existing 91st Avenue facility by the engineers for the City of Phoenix, the ADHS, and the EPA. The operation-maintenance cost represents the total expenditure per unit of flow required to provide adequate treatment and maintenance of the 91st Avenue Treatment Plant.

23rd Avenue Treatment Plant: Capital and operation-maintenance costs were not generated for the existing 23rd Avenue Treatment Plant in the 208 study because continued use of the existing facility was common to all of the areawide alternatives. Further, the City of Phoenix was planning to upgrade the facility as required to accommodate existing and projected flows to the plant regardless of the outcome of the 208 planning process.

Tolleson Treatment Plant: Under all four of the final areawide alternatives, the existing Tolleson facility will be expanded to accommodate a year 2000 domestic flow of 7.2 mgd from the Tolleson and Peoria service areas, with an allowance for additional flows from the packing plant in Tolleson. The basis for the decision by Tolleson and Peoria to expand and utilize the existing Tolleson plant included the following considerations:

1. The existing Tolleson plant served only Tolleson and had excess treatment capacity above the projected needs of Tolleson through the year 2000;

2. The existing Tolleson plant was well maintained and operated and produced a high quality secondary effluent;

3. The City of Peoria was experiencing growth, and increased flows, but, owned no interceptor or treatment capacity

Estimated Capital Costs For Expansion Of
The Existing 91ST. Avenue Treatment Plant

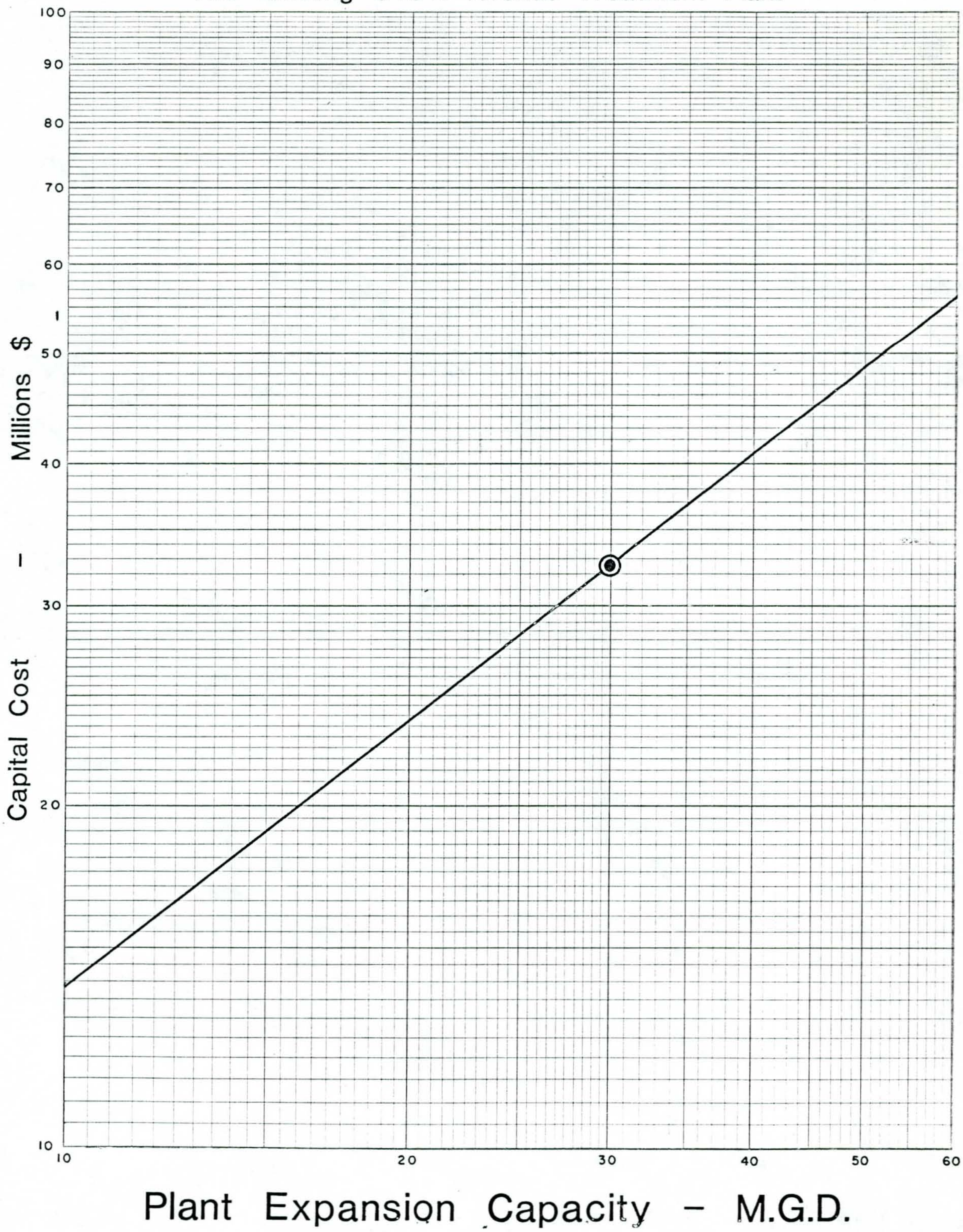


FIGURE III - 2
PHOENIX URBAN STUDY

in any of the area systems. The city was renting capacity from Glendale and Phoenix;

4. Prior to the 208 study, Peoria was planning to join with Glendale, Youngtown, Phoenix, and others in the proposed 99th Avenue Interceptor project and the 91st Avenue Plant expansion. However, when the EPA imposed an Environmental Impact Statement (EIS) requirement on the 99th Avenue Interceptor project, it became apparent to Peoria that probable delays associated with the EIS completion could place them in an untenable position if Glendale or Phoenix could not provide additional capacity in the interim to accommodate Peoria's growing wastewater needs;

5. The existing Tolleson Plant, with relatively minor modifications, could be expanded to accommodate the projected capacity needs of Tolleson and Peoria through the year 2000.

Based on the above considerations, time effectiveness, and cost effectiveness, Peoria and Tolleson entered into an agreement to expand and utilize the existing Tolleson facility.

Expansion of the existing plant would take the form of flow equalization facilities; added primary, intermediate, and final sedimentation tanks, as well as increased pumping and piping capacity and yard piping. The main treatment process will continue to be bar screens, grit removal, primary clarifiers, first and second stage trickling filters, and intermediate and final clarifiers. Expansion of the plant will take place at the existing site and the land area required for the necessary additions is negligible. Because of immediate community needs, construction of the additions to the Tolleson plant and interceptors will be staged to occur in 1980-85. Effluent from the plant will continue to be used on the adjacent sod farm or discharged to the Gila River.

The estimated costs for upgrading the Tolleson treatment plant to accommodate flows from Peoria were developed for the 208 analysis by the engineers and staff of the City of Tolleson. These costs were prepared from a preliminary design of the required facilities updated to an ENR Index of 2700 based on an analysis of the wastewater treatment plant capacity of the plant by the engineers for the City of Tolleson in 1977.

<u>Fountain Hills Treatment Plant</u>: Under all four of the final areawide alternatives, the existing Fountain Hills treatment plant will be expanded to accommodate year 2000 domestic flows of 2.0 mgd. The basis for the decision by Fountain Hills to expand and utilize the existing plant included the following considerations:

1. The community is geographically removed from the Phoenix metropolitan wastewater planning areas and is bounded on the south and east by Indian reservations, on the north by a state park, and on the west by a range of mountains;

2. The community had an existing collection system and secondary treatment facility at Fountain Hills. The treatment plant was constructed in 1974 with a design capacity of 0.5 mgd and utilized the modified activated sludge process;

3. Effluent from the plant was being reused for turf and golf course irrigation and discussions with the local sanitary district officials indicated additional turf and golf courses areas would be available for wastewater reuse as the flows increased;

4. A cost analysis performed in an early phase of the Phoenix Urban Study showed that it was significantly more cost-effective for Fountain Hills to operate its own system than to join in a regional system. This major cost difference was verified by the engineers for Fountain Hills and was due largely to extremely high pumping costs to convey the Fountain Hills flows to the regional system.

Based on the considerations described above, the Fountain Hills Sanitary District decided to expand and operate their existing collection, treatment, and reuse system.

The estimates of costs for the Fountain Hills treatment plant expansion were made from cost curves developed in the Phoenix 208 analysis as shown on Figures III-3 and III-4.

The capital cost curve shown in Figure III-3 was developed from a recent EPA publication entitled "An Analysis of Construction Cost Experience for Wastewater Treatment Plants" (1976). This publication presented capital cost curves for primary, secondary and advanced trreatment facilities ranging in size from 0.01 mgd to 1000 mgd. These curves were based on a survey of actual costs and were based on a specific ENR Index to allow escalation to present day values. Once the curve data from the publication were updated and adjusted to the Phoenix area, the resulting costs were verified against specific area treatment cost estimates or bids. The final curve, shown in Figure III-3, included costs for treatment units and appurtenances not included in the analysis, contingencies, legal requirements, fiscal requirements, interest during construction, engineering and land.

The operation-maintenance costs shown in Figure III-4 were developed from a recent EPA publication entitled "A Guide to Selection of

Estimated Capital Costs for Secondary Wastewater Treatment Plants in the Phoenix Study Area

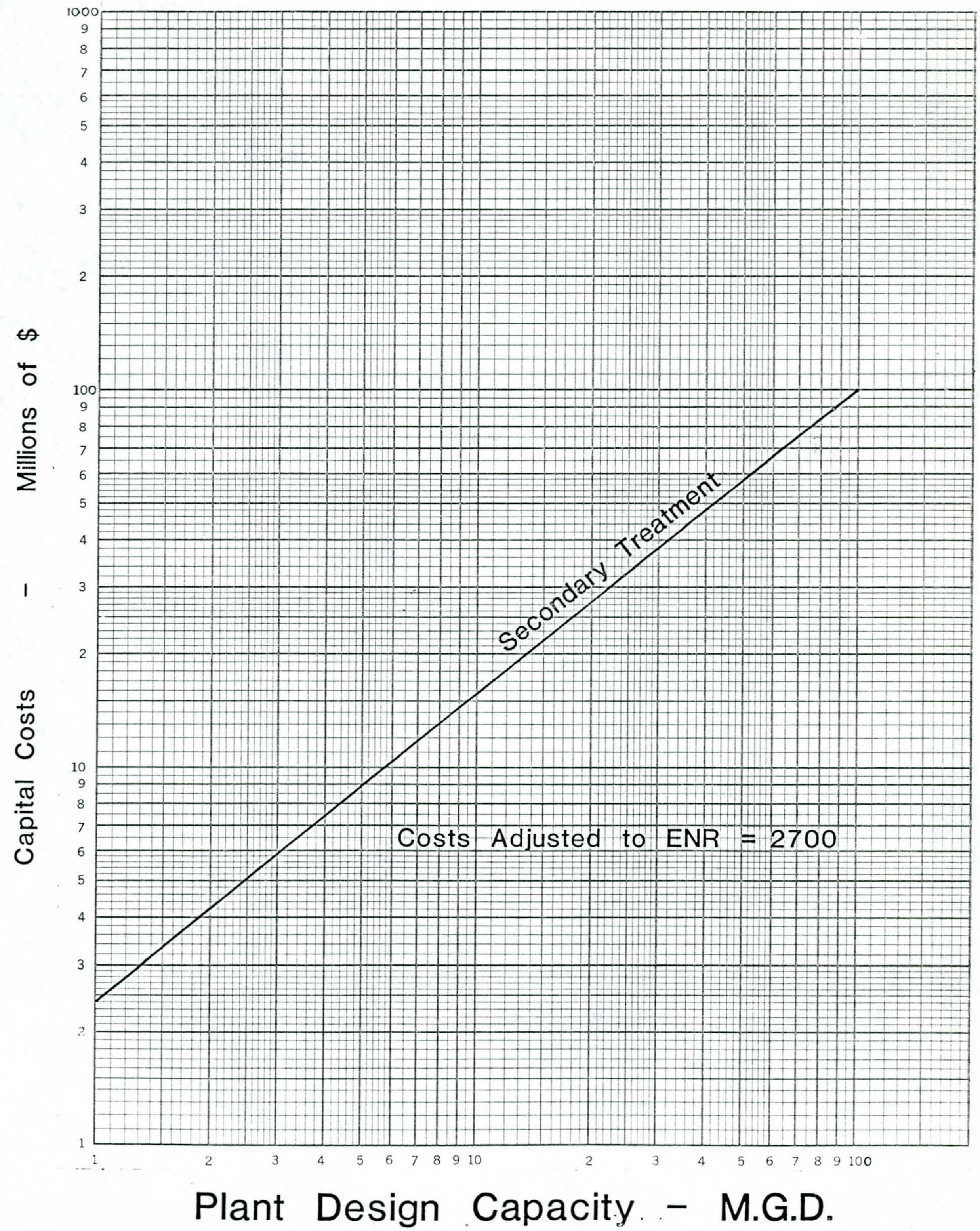


FIGURE III - 3
PHOENIX URBAN STUDY

Cost Effective Wastewater Treatment Systems," (1975). To verify the estimating curve and to adjust it to area prices, actual costs for the Phoenix area were obtained.

In reviewing the existing area cost information, it was found that the City of Phoenix maintained detailed and accurate information concerning expenditures for operation and maintenance at the 91st and 23rd Avenue treatment plants. These data were obtained for fiscal years 1966 through 1976 and compared with the estimated costs from the EPA publication for the same treatment process. In general, the correlation between the EPA 1975 source and the actual data for Phoenix was quite good.

The curve shown in Figure III-4 was developed by adjusting the estimated material-supply costs to a Wholesale Price Index for Industrial Commodities of 200. This index corresponds to an ENR index of 2700 for January, 1978. In addition, the labor costs in these curves were adjusted by using the actual average wage rates for the Phoenix area treatment facilities at $9 per hour for plants smaller than 10 mgd. The operation and maintenance costs as presented in Figure III-4 include all costs for materials, supplies, chemicals, power and labor.

Williams Air Force Base Treatment Plant: Under all four final areawide alternatives, the exiting Williams Air Force Base secondary treatment plant would be operated and maintained by the Base through the year 2000. This decision was reached by the Air Force because this treatment facility was recently remodeled and has a design capacity equal to the projected year 2000 average flow (1.0 mgd). No major capital outlay for construction or remodeling would be required at the plant through the planning period of the study.

Additionally, there is a proposed 9-hole expansion of the existing golf course to be irrigated with effluent. Existing and proposed reuses, therefore, are available in the vicinity of the plant to utilize all projected flows from the treatment plant. Costs were not generated in the final 208 analysis for the Williams AFB system.

Land Treatment: Since the latter part of 1976, the investigation of land treatment alternatives in the Phoenix area 208 study paralleled the investigation of conventional treatment alternatives. Initially, various land treatment systems and processes, and the design criteria, costs, and environmental factors to be considered in selecting land treatment sites, were investigated. Thirteen general areas that appeared to be suitable for land treatment sites were identified.

Based on these general sites, a large array of different land treatment

Estimated Annual Operation & Maintenance Costs for Secondary Wastewater Treatment Plants

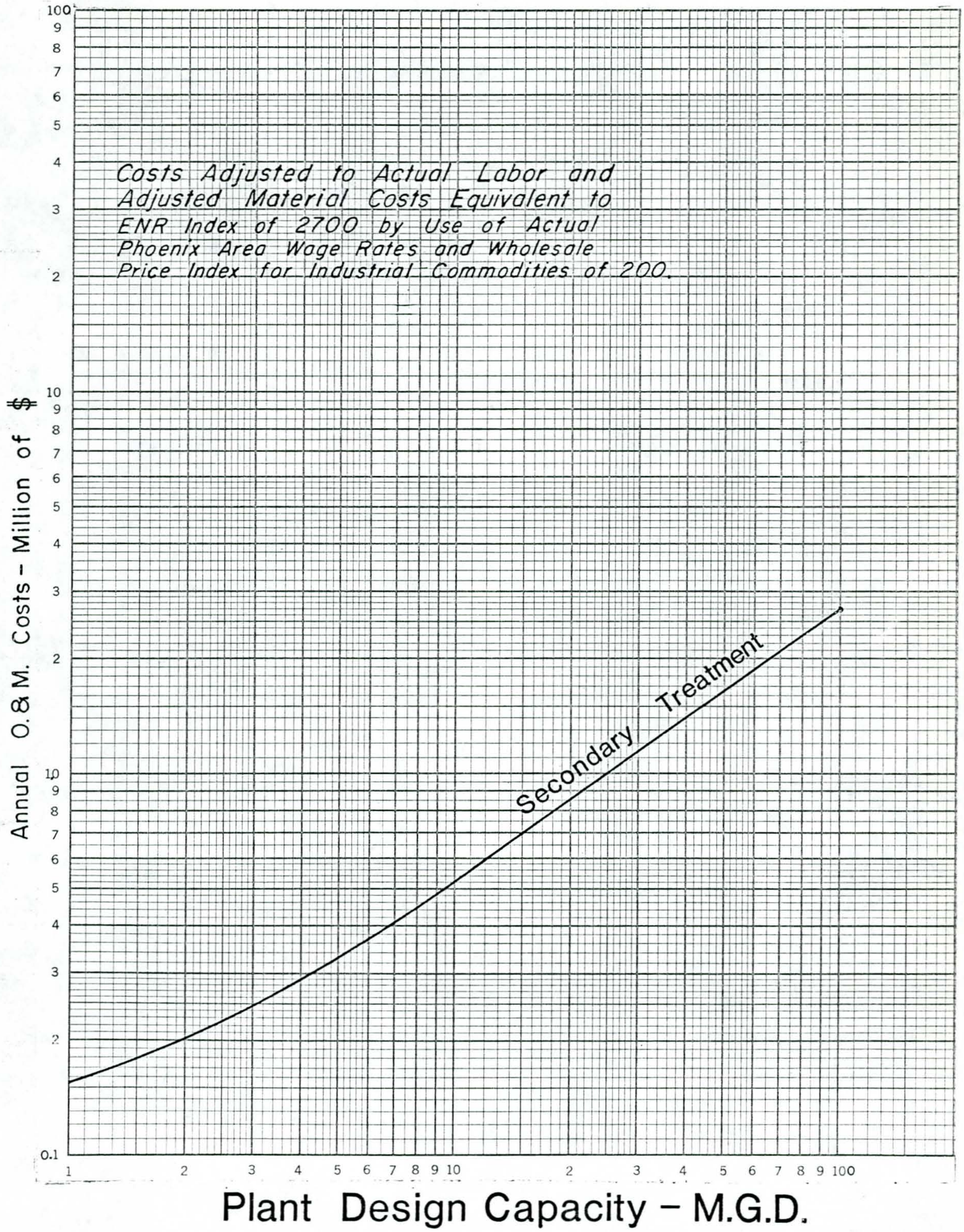


FIGURE III - 4
PHOENIX URBAN STUDY

alternatives utilizing overland flow or infiltration percolation were developed on a site specific basis, including costs, area requirements and power consumption. These alternatives were evaluated for relative feasibility. Although each of the alternatives were technically feasible, many of them had limitations such as conflicting projected land uses, institutional constraints, or poor location in relation to groundwater basins, restricting their implementation. The analysis resulted in the selection of a small array of land treatment alternatives for further study.

A more detailed analysis was then made evaluating the application of primary effluent as well as series infiltration/percolation systems. This analysis developed costs associated with producing effluent of various qualities from several land treatment sites utilizing a series concept. The concept was based on a treatment process including two infiltration/percolation systems, one following the other, thereby increasing the overall detention time and distance traveled through the soil matrix. This would result in increased removal efficiency for nitrogen and phosphorus. The combination provided a better quality effluent than the single pass systems. Although tehcnically feasible, these systems were not cost-effective and were eventually eliminated by the advisory groups.

During the later portions of the 208 analysis consultants reviewed the land treatment alternatives and eliminated all but five general sites. These systems were elimated primarily on socio-economic, environmental and groundwater issues as discussed or identified by advisory groups, governmental representatives, and tribal officials.

During the subregional analysis, separate analyses was done for all land treatment alternatives on the westside, and eight alternative sites with a total of 32 alternatives were developed and analyzed in detail. Screening of the alternatives reduced the number to 15 alternatives located at four sites. Advisory group review further reduced these various alternatives to three sites. Near the end of the 208 study, because of renewed interest by EPA and local agencies, two additional land treatment options were evaluated: an overland flow system for a site in the northeast portion of the metro study area and another rapid infiltration site at Cotton Lane near the proposed Reems Road Plant in the southwest area. By the time the final four areawide alternatives were evaluated, these two alternatives were the only two land treatment alternatives still viable. These, in turn, were eliminated largely on the basis of cost.

Throughout the land treatment alternative analyses in the Phoenix area 208 study, the engineering and cost analyses were performed in part by utilizing a systems approach with the benefit of a computer program package. The program was comprised of two distinct software packages, one providing engineering analysis and the other providing the cost analysis. Both programming packages were developed during the course of the Phoenix Urban Study. The cost analysis program was a modification of a portion of the Corps of Engineers Computer-Assisted Procedure for the Design and Evaluation of Wastewater Treatment System (CAPDET) Program. A block diagram of the program is shown on Figure III-5.

The analysis was performed utilizing FORTRAN IV. The system was a Computer Hardware Incorporated (CHI) 2130 Computer System. It consisted of CHI 2130 Central Processor, CHI 1114 Disc Controllers, two Memorex 660 Disc Drives, CHI 1103 Printer, IBM 1403 Card Read/Punch, and a UCC 2000 Plotter.

The analysis was initiated with data cards for the engineering analysis being input with the appropriate program. Output from this program was stored in a data matrix on magnetic disc in addition to the printed output. This information coupled with additional input data was used as input to the cost program which provided an annotated printed output for each site as well as two data summary tables. The computer program subroutines shown on Figure III-5 are as described in Table III-20.

TABLE III-20

LAND TREATMENT PROGRAM SUBROUTINES

1.	PLIM	Sets Pumping Heads
2.	TRANEX	Calculates Transmission Conveyance and Pumping Costs
3.	TRANOT	Prints Annotated Transmission Data
4.	OVEREX	Calculates Overland Flow Cost Table
5.	OVEROT	Print Annotated Overland Flow Data and Output
6.	RAPIDEX	Calculates Rapid Infiltration Cost Table
7.	RAPIOT	Print Annotated Rapid Infiltration Data and Output

Even though the engineering the cost analyses involved in the land treatment portion of the Phoenix Urban Study were detailed and comprehensive, evaluations and decisions by the 208 staff, the local communities and other entities eliminated all but two alternatives from consideration in the final alternative analysis. Further computer application for analysis of these remaining two alternatives was not utilized

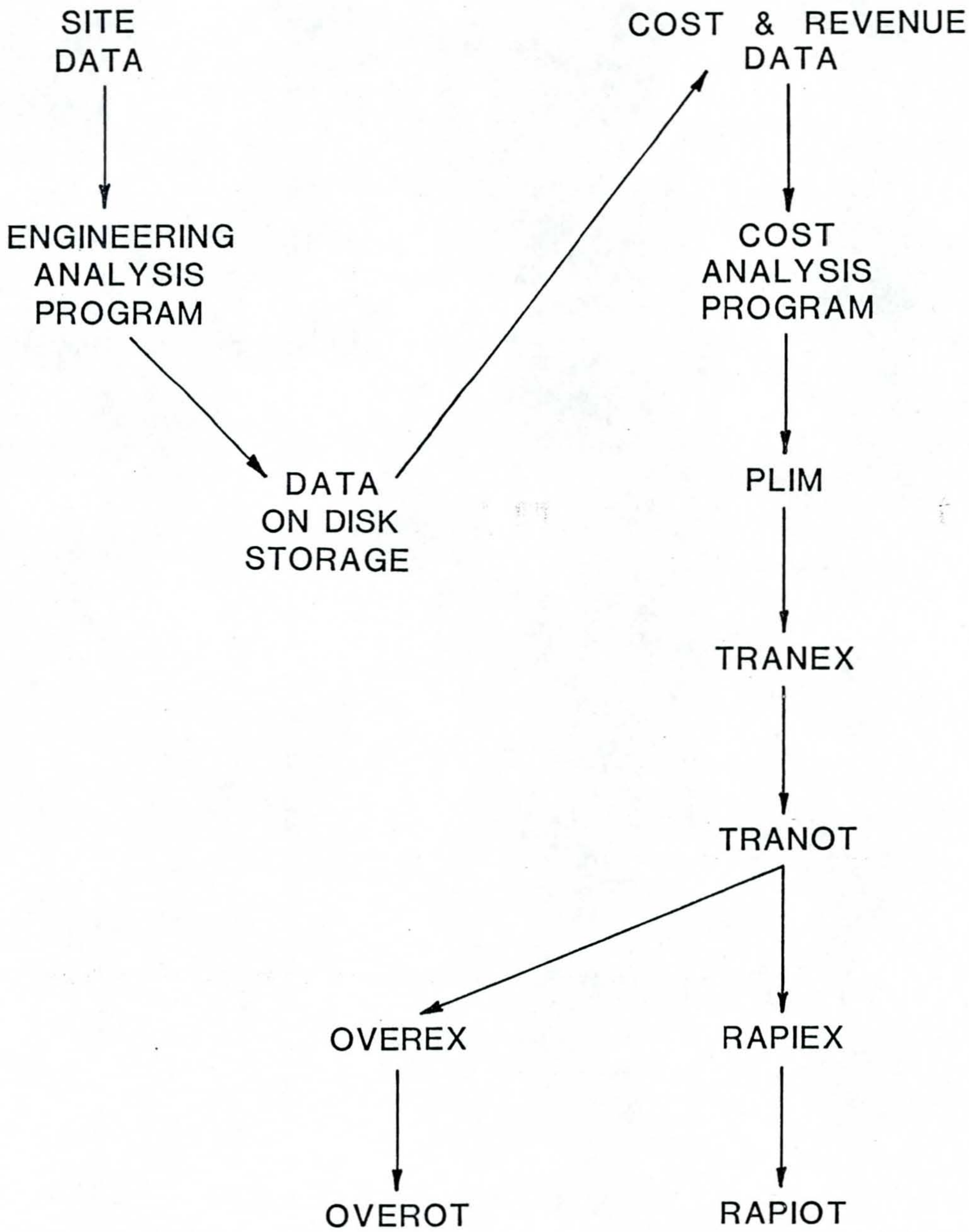


FIGURE III - 5
LAND APPLICATION COMPUTER
SOFTWARE PACKAGE

since preliminary designs and cost estimates for these land treatment options could be prepared based on site specific data and area specific cost factors more quickly and with less effort than through utilizing the computer programs.

Costs for the land treatment systems in the final areawide alternatives were based on data published in an EPA document entitled "Cost of Wastewater Treatment by Land Application," 1975. This document developed cost curves which yielded both capital and annual costs (were applicable) for the following items.

Treatment Facility
Transmission System
Site Clearing
Storage Facility
Distribution System
Recovery System
Service Roads
Fencing

The various unit costs used in the analysis were coordinated with those used for the conventional treatment analysis to insure accurate reflection of local conditions.

Irrigation/Disposal Systems: In the final areawide alternative analysis, several irrigation/disposal systems were identified as viable effluent handling facilities in the Phoenix area. These systems were analyzed, evaluated, and finally incorporated in the waste management plans based on local desires, cropping patterns, and irrigation water needs.

For the purposes of the final 208 analysis it was assumed that effluent would be supplied to the farmer in the middle of his field. It also was assumed that the farmer would own and operate the farm, and, therefore, costs associated with conveyance and storage were the only costs to be borne by the operating agency. Finally, it was assumed that the treated wastewater would be conveyed by gravity through a lined canal to a lined storage facility located adjacent to the irrigated acreage.

Based on these assumptions, site specific preliminary designs of conveyance and storage systems were developed for the proposed irrigation/disposal systems to be located at the Reems Road, Chandler, Gilbert North, Gilbert South, and the Northeast Area plants. The costs for conveyance to and storage at the irrigation/disposal sites which were identified in the final alternative analysis were developed from cost curves an EPA publication entitled "Cost of Wastewater Treatment by Land Application," 1975. The data from the cost curves was updated to reflect current local costs.

Pump Stations: The data source used to develop estimated capital and operation and maintenance costs for conventional sewage effluent pump stations was an EPA publication entitled "Costs of Wastewater Treatment by Land Applciation," (1975).

This data source was selected because it was relatively recent and it presented estimated capital and operation and maintenance costs for pump stations at three different total pumping heads. In addition, this publication broke down operation and maintenance costs including power, labor, and materials costs, to allow adjustment to local conditions in the Phoenix area. The cost curves utilized in the final areawide alternative analysis are as shown on Figures III-6 and III-7.

Interceptors and Force Mains: Interceptor sewer unit costs were developed for varying depths and pipe diameters from 12 to 84 inches with and without pavement replacement using actual bid tabulations and data developed for the Phoenix area. Table III-21 lists the unit prices used for interceptor construction.

The operation and maintenance costs for interceptors were taken from the EPA Publication "Costs of Wastewater Treatment by Land Application," (1975) and adjusted to correspond to an ENR Index of 2700. These are listed in Table III-22. The capital and operation and maintenance costs for force mains were taken from the EPA publication "Costs of Wastewater Treatment by Land Application," (1975) and adjusted to correspond to an ENR Index of 2700. These are listed in Table III-23.

COST METHODOLOGY

The costs which would be incurred by the implementation of any of the areawide alternatives can be classified as 1) capital cost, or 2) annual operation and maintenance cost. The capital costs include all the expenditures associated with the complete erection of a treatment facility or collection system including the actual construction costs and expenses incurred for administrative, legal, and engineering services. The annual operation and maintenance costs are those associated with maintaining the facility's operation, including administrative and operator salaries, power and chemical costs, and equipment repair and replacement.

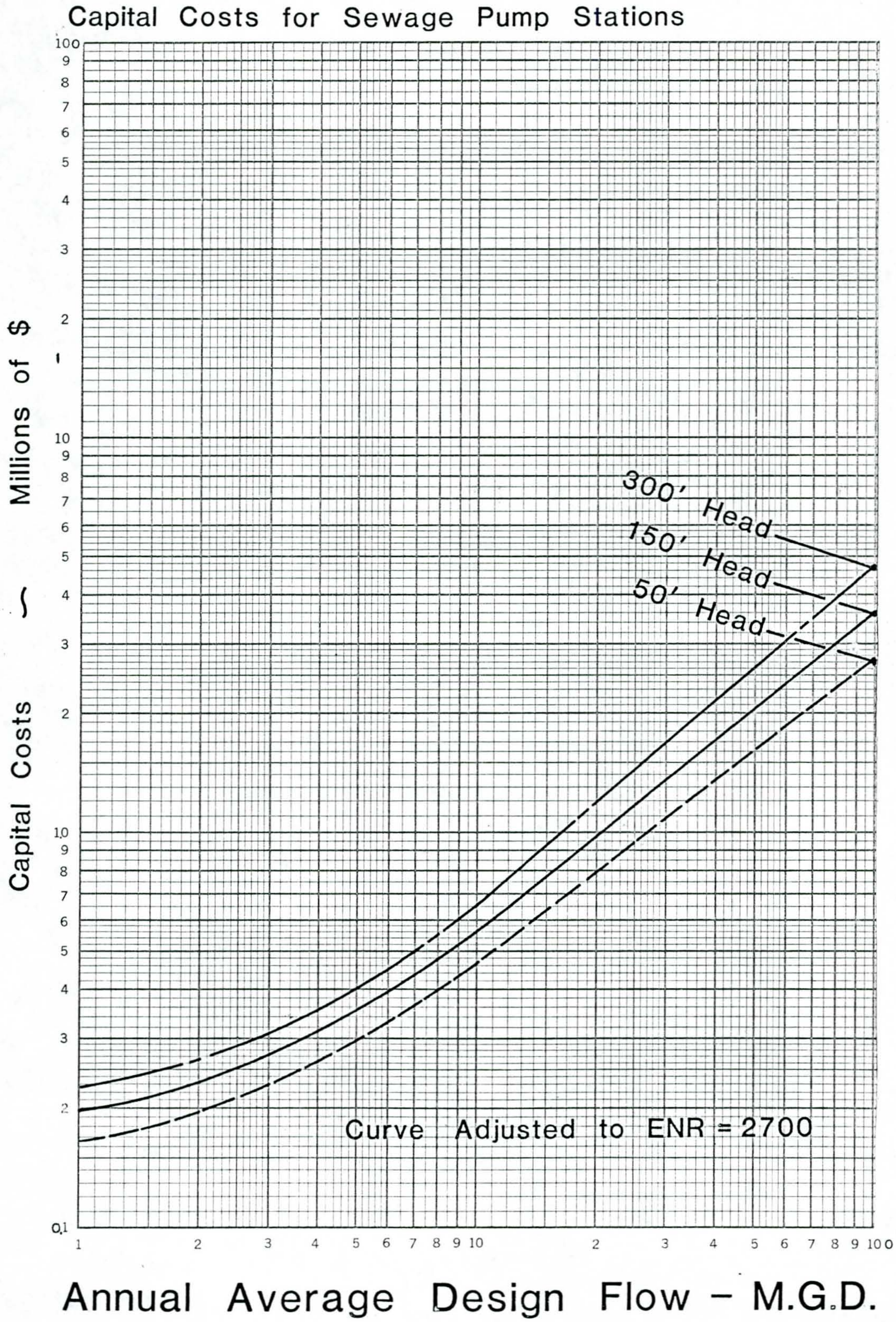
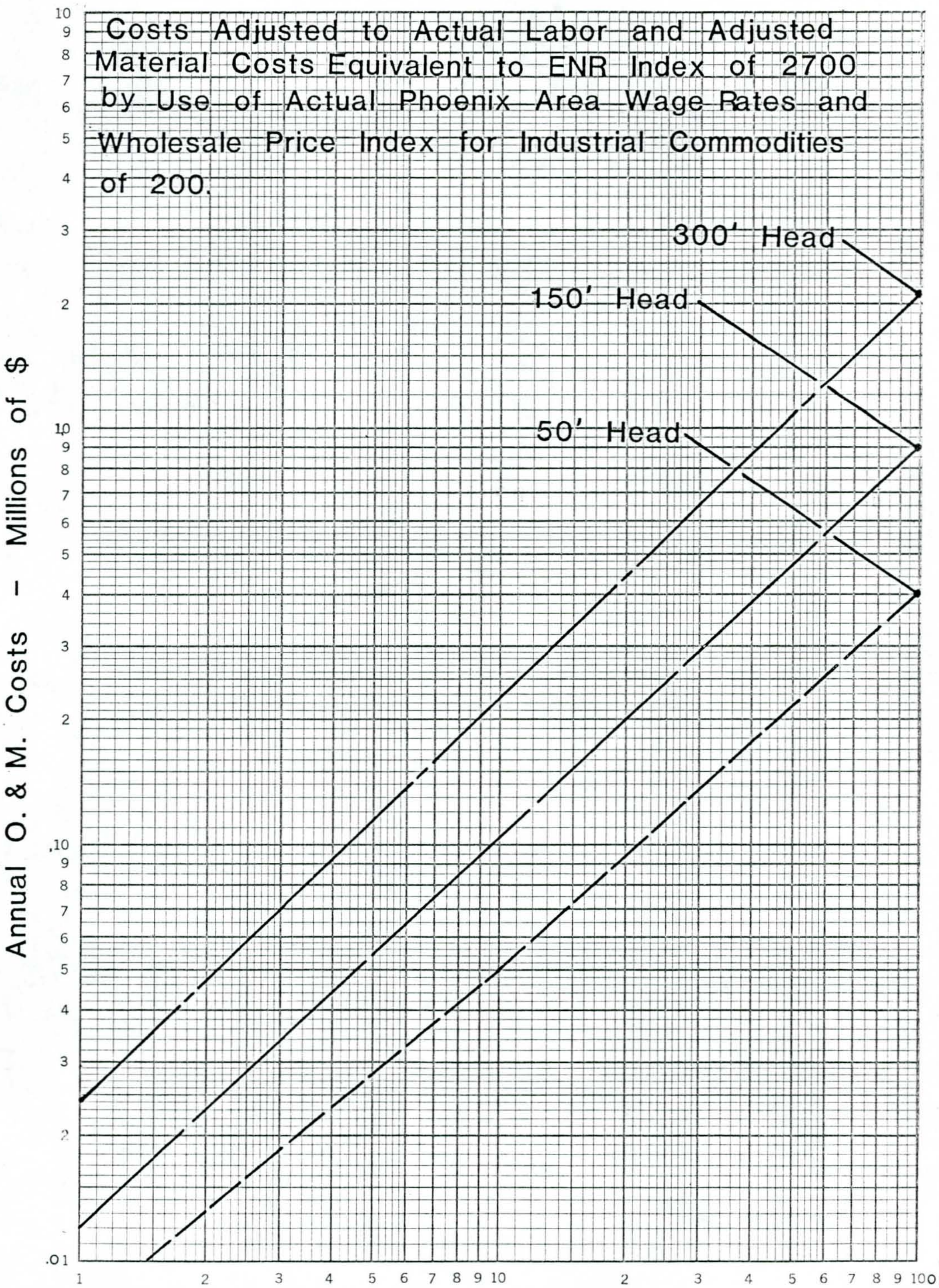


FIGURE III - 6
PHOENIX URBAN STUDY

Operation & Maintenance Costs for Sewage Pump Stations



Annual Average Design Flow - M.G.D.

FIGURE III -- 7
PHOENIX URBAN STUDY

TABLE III-22

INTERCEPTOR OPERATION AND MAINTENANCE COSTS

Pipe Diameter (Inches)	O & M Costs ($/Linear Foot/Year)
12	0.08
15	0.11
18	0.14
21	0.15
24	0.16
27	0.17
30	0.19
33	0.21
36	0.23
39	0.25
42	0.26
48	0.29
51	0.30
53	0.33
57	0.35
60	0.38
66	0.41
72	0.44
78	0.47
84	0.53

TABLE III-23

FORCE MAIN CAPITAL AND OPERATION AND MAINTENANCE COSTS

Pipe Diameter (Inches)	Capital Cost ($/Linear Foot)	O & M Costs ($/Linear Foot/Year)
8	20	0.05
10	24	0.05
12	26	0.06
14	29	0.07
16	33	0.07
18	37	0.09
21	43	0.09
24	49	0.09
27	55	0.11
30	62	0.13
33	69	0.14
36	77	0.14
39	83	0.17
42	92	0.19
48	109	0.20

In order to compare the total monetary outlay required by each regional alternative for the Phoenix Urban Study area, both annual cost, and in some cases, present worth analyses were performed. Based on the results of these cost analyses, the total monetary impact of each regional alternative could be evaluated relative to the other alternatives. For the purposes of analyzing the costs of the regional alternatives, certain assumptions and methods were used in the planning process.

The planning period for the alternatives were selected to be 20 years based on guidance from the Environmental Protection Agency. Also, the interest rate used in the analysis was selected to be 6 5/8 percent in compliance with the EPA's most recent guidelines for a cost-effective analysis. To comply with the EPA's guide-lines for a cost-effective analysis, the total monetary outlay required by each of the areawide alternatives was calculated by converting the total annual operation and maintenance costs to a present worth at an interest rate of 6 5/8 percent and combining it with the total capital cost to yeild a total estimated present worth.

The present worth analysis is a method to show, on a common basis, the theoretical amount of money required at present to build and operate a system over the planning period of 20 years. This type of calculation is used to compare different alternatives which have expenditures occurring at different rates (staged construction) and different salvage values. Salvage values were calculated based on straight line depreciation over the planning period.

The final cost analysis factors for the regional alternatives are summarized as follows:

Item	Factor
1. Interest Rate	6 5/8%
2. Planning Period	20 years
3. Service Lives for Calculation of Salvage Values	
a. Land	Permanent Life
b. Structures and Earthwork	40 years
c. Sewers and Force Mains	40 years
d. Process Equipment	20 years
e. Auxiliary Equipment	10 years

4. Capital Cost Allocation Factors

a. Mechanical Treatment Facilities and Pump Station	40% Structures 30% Process Equipment 30% Auxilliary Equipment
b. Lagoon Treatment Facilities	
(1) Aerators & Chlorination Equipment	20 year life
(2) All other Facilities	40 year life

5. Interest During Construction

	½ p.c.i. p=period of construction (years) c=construction cost i=interest rate
a. Mechanical Treatment Facilities	p=2½ years
b. All other facilities	p=1½ years

Based on the factors shown above the total present worths for the regional alternatives were calculated and documented.

In order to present the cost data in form which would be most useable, the capital, operation-maintenance, and present worth results (converted to equivalent annual costs) were allocated to each individual community. This allocation of costs was made based on flow contribution to the system from each community by multiplying the total estimated cost for the system by the cost allocation ratio shown below:

System Component	Cost Allocation Ratio
1. Treatment Facilities	Ratio of the average year 2000 flow contribution from the community to the total average year 2000 flow at the plant.
2. Collection System	Ratio of the peak year 2000 flow contribution from the community to the total peak year 2000 flow to the sysem at every indentifiable tributary flow point along the proposed interceptor sewers reaches.

The cost allocation methodology described above for the treatment facilities is straightforward. However, the collection system methodology was complex and required considerable calculation because the costs of each section of proposed sewer line were prorated to the contributing communities by different ratios at every reach of line where a tributary flow could be identified.

AREAWIDE PLAN SELECTION

Since the components of the four alternatives have been looked at in various levels of detail, the methodology used in the final evaluation was to identify those criteria which would have an influence in the selection of the final plan. The criteria identified by the consultants fell into three main areas:

- o Technical (flexibility and costs),
- o Socio-economic (effluent agreements, water recharges, site availability, agricultural preservation, use of Indian lands),
- o Environmental (water resources, public health, archeological resources, biological resources).

Technical Evaluation

Flexibility: In evaluating the flexibility of each of the four alternatives the primary concern was to identify the alternative offering the most options to the region as a whole for wastewater collection and treatment.

On the westside, the construction of the Reems Road plant would offer considerably more flexibility to the westside communities. Without this plant, the westside communities must develop a pumpback system to the 91st Avenue plant. This type system, by its nature, was less readily expanded than would be a system including collection by gravity and treatment on the westside (Reems Road). Therefore, those alternatives containing the Reems Road plant (2 and 4) were viewed as more flexible than those without it.

Similarly, if the Northeast plant were constructed (alternative 3 and 4) the participating communities would have greater flexibility for the treatment of their wastewater. A small local plant can generally be expanded more readily than a large regional plant, and as such is better able to accommodate future population changes.

A negative consideration regarding the Northeast plant, however, involved the timing of the decision for a Northeast plant and the planning of the Southern Avenue Interceptor (SAI). If a plan were selected which included the Northeast plant, then the size of the SAI would necessarily be reduced. Conversely, should an option without the Northeast plant have been selected, then by necessity the Southern Avenue Interceptor and the 91st Avenue WWTP would have been sized to accommodate flows from the northeast communities.

It became apparent that if a Northeast plant were built, all commitments and agreements must be certain and final prior to a commitment to the SAI.

The option was retained that, should at a future date more flows be generated than were projected,the northeast communities would still have the option of planning a northeast facility.

Costs: Each of the four areawide alternatives involved a combination of the following component parts:

- o collection systems
- o treatment facilities
- o reuse/disposal systems

Additionally, the component parts were developed on a time schedule, being staged for construction at various times between 1980 and the year 2000 according to need.

In order to evaluate the total wastewater systems on an areawide basis, costs were developed for each of the component parts and combined as required for each areawide alternative. Capital and annual operation and maintenance costs were developed. Because all facilities were not scheduled to be constructed at the same time, present worth and equivalent annual costs were developed in order to form an equal basis for comparison between the various alternatives.

Table III-24 summarizes the various costs for each of the four alternatives.

As can be seen from the table, Alternative 1 proved to be the least costly, followed by Alternative 2, then Alternative 3, with Alternative 4 being the most costly. Caution must be exercised, however, when comparing the alternatives on the basis of costs as there is only a 7 percent difference between the least and most costly alternative on the basis of capital cost.

TABLE III-24

SUMMARY OF REGIONAL ALTERNATIVE COSTS

Alternative		Capital Cost	Annual O & M	Total Annual
		(Millions of Dollars)		
1.	91st Avenue	$114.91	$1.87	$14.06
	Tolleson	6.83	0.29	0.89
	Gilbert	9.85	0.26	0.66
	Chandler	10.43	0.46	1.10
	TOTAL	142.02	2.88	16.71
2.	91st Avenue	107.39	1.64	13.54
	Tolleson	6.83	0.29	0.89
	Gilbert	9.85	0.26	0.66
	Chandler	10.43	0.46	1.10
	Reems Road	11.92	0.22	1.29
	TOTAL	146.42	2.87	17.48
3.	91st Avenue	105.59	1.64	12.97
	Tolleson	6.83	0.29	0.89
	Gilbert	9.85	0.26	0.66
	Chandler	10.43	0.46	1.10
	Northeast	15.54	0.52	1.82
	TOTAL	148.24	3.16	17.44
4.	91st Avenue	97.26	1.42	12.59
	Tolleson	6.83	0.29	0.89
	Gilbert	9.85	0.26	0.66
	Chandler	10.43	0.46	1.10
	Reems Road	11.92	0.22	1.29
	Northeast	15.54	0.51	1.82
	TOTAL	$151.82	$3.16	$18.35

CHAPTER IV

FLOOD CONTROL HYDROLOGY

The Phoenix Urban Study, as a part of its comprehensive examination of water resource issues in the Phoenix metropolitan area, examined alternatives for eight flood hazard areas:

- o Glendale-Maryvale
- o Cave Creek Below the Arizona Canal
- o Old Crosscut Canal
- o South Phoenix
- o Upper Indian Bend Wash
- o Gila Floodway
- o Scatter Wash
- o Salt River through Phoenix

A large portion of this analysis involved investigations into the hydrological factors in the study area which affect flooding. The findings of these studies are presented below. Tables containing hydrologic data for Glendale-Maryvale, Cave Creek Below the Arizona Canal, Old Crosscut Canal, South Phoenix, and Upper Indian Bend Wash are found in Appendix A.

RUNOFF MODEL FOR VALLEY AREAS

Sheet Flow Runoff

Most watersheds in the study area do not have steep slopes. Urbanization together with agricultural development has obliterated most original watercourses, and runoff occurs basically as sheetflow. Flat valley areas are not conducive to good runoff measurement hence, sufficient data with which to derive precipitation-runoff relationships from past runoff events does not exist. A theoretical procedure was needed which allowed computation of flood hydrographs using parameters that could be determined from topographic maps or had generally accepted values as opposed to empirically determined coefficients. A review of literature revealed that several studies dealing with sheet flow have been conducted and various models for determining hydrographs have been derived. For this study, some models were rejected because the difficulty of accurately estimating rather sensitive input variables. Purely graphic methods were not used because the large number of hydrograph computations necessitated computerization of the methodology. The runoff model developed in detail in the unpublished Gila Floodway Report (1977) was used for this study. Its derivation is briefly described in the following paragraph.

Derivation of Single Linear Reservoir Model: In this study, a linear storage system was used to determine the time distribution of runoff from an effective rainfall hyetograph. Although nonlinearity of the rainfall-runoff process has long been recognized, the lack of data makes the use of a simple procedure the most reasonable approach. The linear storage system is analogous to a reservoir in which storage is related to outflow by the equation

$$S\text{-}KO \tag{1}$$

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

$$1\text{-}0 = \frac{dS}{dt} \tag{2}$$

equation (1) can be expressed as

$$O_2 = C_1(I_1+I_2) + C_2O_1 \tag{3}$$

where

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

and

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

In studies of urban watersheds, utilizing the linear storage system concept, the coefficient K can be approximated by "lag time" (time between centers of mass of effective precipitation and runoff). An apparent relationship between lag time and measurable physical parameters does not exist; however, there are relationships between sheet flow time of concentration and measurable physical characteristics. Time of concentration is defined as the time from commencement of rainfall excess until flow from the uppermost edge of the basin arrives at the overflow point (analogous to time to equilibrium).

It can be shown that for a constant effective rainfall intensity, time of concentration, tc, using kinematic wave theory, can be expressed as

$$t_c = \left(\frac{L i^{1-m}}{\alpha} \right)^{1/m} \tag{6}$$

where

L = length

α = coefficient

i = effective rainfall intensity

m = exponent

Coefficient values for an altered form of the above equation have been demonstrated:

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

where tc = time of concentration in minutes

L = length in feet

i = effective rainfall intensity in inches per hour

n = Manning's roughness coefficient

s = slope in feet per foot

A typical hydrograph resulting from a constant effective rainfall intensity i of a duration t=tc is shown on (Figure IV-1). If the time to peak is used to approximately the time to the center of mass of the hydrograph and K is approximated by this lag time, then K=tc/2. Equations (4) and (5) then become respectively,

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

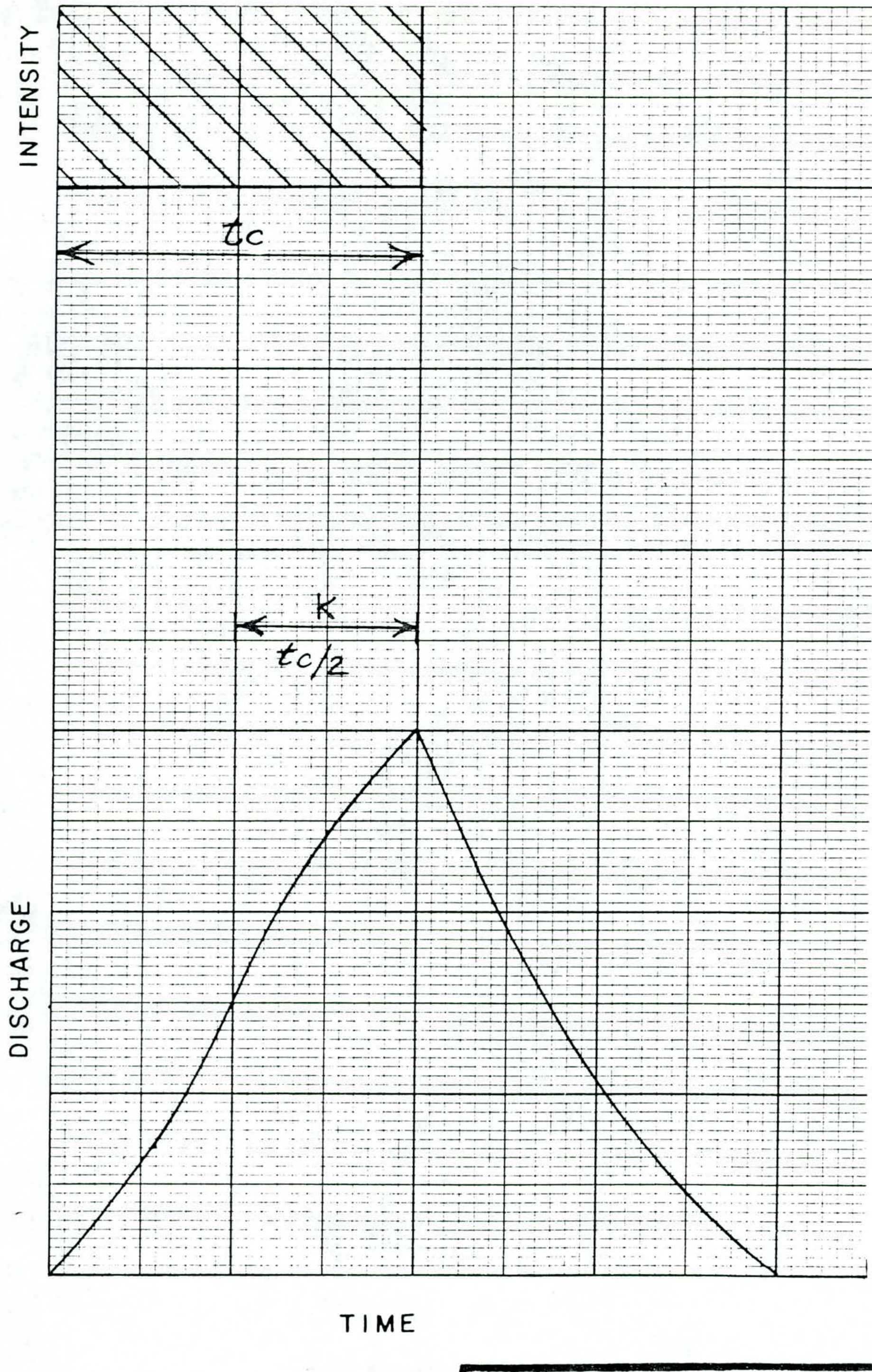
and

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

TABLE III-21

INTERCEPTOR CAPITAL COSTS
(Dollars per Linear Foot)

PIPE DIAMETER	DEPTH OF EXCAVATION (FEET)					
			(With Pavement Placement)		(Without Pavement Replacement)	
(Inches)	10	20	20	30	10	30
12	$28.75	$57.25	$48.25	$104.75	$24.25	$88.50
15	35.75	64.50	54.75	112.00	35.75	95.25
18	44.25	73.75	64.25	121.75	39.00	104.50
21	52.25	81.25	71.50	130.00	46.50	112.50
24	62.50	91.50	80.25	140.50	56.00	122.00
27	72.50	101.50	90.00	151.00	65.50	132.00
30	83.00	113.75	101.75	163.75	75.50	144.50
33	94.00	124.50	112.50	175.00	86.00	155.00
36	105.25	136.50	123.00	187.25	96.25	166.25
39	116.00	148.00	134.15	198.75	106.75	177.50
42	124.75	157.50	143.25	208.50	115.00	187.25
48	140.25	173.00	158.00	224.75	130.00	202.25
51	148.00	180.75	165.50	232.50	137.25	210.00
54	157.00	191.50	175.00	244.25	144.75	220.25
57	166.25	201.75	184.50	256.00	153.75	231.50
60	176.00	211.50	194.00	265.75	163.00	241.00
66	194.00	229.75	210.50	284.75	179.25	258.00
72	216.50	253.00	232.75	308.50	201.00	280.75
78	242.00	278.75	256.50	335.00	224.75	305.50
84	269.50	306.75	284.25	364.00	251.00	334.00



DETERMINATION OF COEFFICIENT K

FIGURE IV-1

Values C_1 and C_2 may be substituted into equation (2), which describes the rainfall-runoff process. This method of analyzing sheet flow conditions is referred to as the "single linear reservoir" model.

SYNTHESIS OF THE STANDARD PROJECT FLOOD

The Standard Project Flood (SPF) represents the flood that would result from the most severe combination of meteorologic and hydrologic conditions considered reasonably characteristic of the region. Normally larger than any past recorded flood in the area, it can be expected to be exceeded only on rare occasions. Preparation of standard project flood estimates in this report were made in accordance with EM1110-2-1411 (Standard Project Flood Determinations).

Standard Project Storm: The August 19, 1954 thunderstorm that was centered generally in the Queen Creek drainage area was determined to be storm with the most severe flood producing rainfall depth area duration relationship and isohyetal pattern that may reasonably be expected to occur over the central portion of Arizona. While the storm lasted about 9 hours, local observations during the storm indicated that nearly all of the precipitation fell during a 7-hour period and that most of the rainfall occurred at many stations within 3 hours or less. Very short durations (5 minutes to 1 hour) of extremely intense rates of precipitation, although not measured in the August 19, 1954 Queen Creek storm because of the lack of properly functioning recording rain gages in the area at the time have been measured on a number of other occasions in central Arizona. They are therefore considered to be reasonably characteristic of the heavier thunderstorms of this part of the state. A standard project storm of 7-hours duration, having large portions of the total precipitation occurring within 1 to 3 hours, was developed from this information.

Rainfall-Runoff Relationships

Mountain and Steep Valley Area: The Phoenix Mountain S-graph was considered applicable to the mountainous portions of the study. This S-graph was used for subareas in the South Mountains and in the upper Cave Creek basin where runoff concentrates S-graphs are used in conjunction with estimated lag times to develop subarea unit hydrographs. Subarea characteristics needed to derive lag times were estimated from topographic maps and field observations. Subarea characteristics are listed in tables 6 through 10 in Appendix A.

Valley Areas: The "single linear reservoir" runoff model was applied to convert rainfall to runoff in the study area. For the subareas that included both valley and mountainous terrain it was found that sheet flow type runoff predominated. The "single linear reservoir" model also was used for these subareas. Subarea characteristics describing sheet-flow areas are listed in tables 6 through 10 in Appendix A.

Precipitation Loss Rates: Because loss rate functions developed for the Gila River Basin, New River and Phoenix City streams, Arizona, Arizona, Design Memorandum No. 2 involved similar soil characteristics to those in the study area, they were used by the Urban Study. Consideration of on-site storage, along with other necessary assumptions, suggested the use of an initial loss and an average constant loss rate. The average loss rate during the fifth and sixth hours, the intense portion of the storm, was determined to be 0.35 inch per hour. This value was used as the constant loss rate for SPF calculations. Summer storms in the study area often occur on dry watersheds. Although the soil may have been wetted by antecedent rainfall, evaporation rates remain high, and depression storage must be satisfied prior to runoff. The effects of depression storage were considered for valley areas but were considered negligible in the mountainous areas.

Flood Routing: The wide range of discharge and channel conditions encountered in this study necessitated the use of several routing techniques to describe adequately the attenuation of a flood wave under each condition. For sheetflow areas where a large amount of channel storage is encountered, the Muskingum flood routing method was chosen. This method also was employed for Indian Bend Wash where overbank storage is significant. The Successive Average Lag method was used for well defined project channels such as those proposed for the Glendale-Maryvale area. Reservoir routing was accomplished by the Modified Puls routing procedure. Each of these flood routing methods is described in detail in "Routing of Floods Through River Channels" EM1102-1408, Corps of Engineers, 1 March 1960.

Several coefficients need to be defined for both methods of channel routing. The Muskingum constant K is approximated by the travel time of a flood wave through a reach. Flood wave travel time in a reach is determined by dividing reach length by average peak flow velocity. Manning's Formula for normal flow and an appropriate channel cross section are used to compute the average peak velocity for the reach. The average peak discharge for a reach is taken as the mean of peak discharges at the upstream and downstream limits of the reach. The Muskingum X values were based on the amount of flow in the stream overbanks. An X value of 0.3 would be used for a well defined natural stream. As overbank flow increases, X decreases until, when X is zero, outflow is strictly a function of storage.

An X value of zero was used to route floods through the sheet flow areas where no well defined channels exist. For Upper Indian Bend Wash, the amount of overbank flow was determined from normal depth calculations and judgement was used to determine an X value of 0.2. An X value of 0.3 was used for the proposed channel in the South Mountains. The Successive Average Lag routing method is based on empirical observations of flood wave attenuation in which the choice of routing coefficients is a function of flood wave travel times (described above) and the unit time used for the routing computation.

Channel Infiltration

Losses due to streambed infiltration in the Phoenix area were determined using observed flood data for the September 3-7, 1970 storm. The record of water behind Cave Creek Dam as well as the observed flood hydrograph for Cave Creek at Phoenix allowed computation of channel infiltration. The recession limb of the observed flood hydrograph for Cave Creek at Phoenix leveled out at a constant flow of 290 cfs between 1800 hours of September 5, and 2400 hours September 7. Normally channel infiltration would diminish rapidly the tail end of recession flow. In this case the steady outflow from Cave Creek Dam was the source of the constant flow. Using the stage recorder chart for the upstream face of the dam, and applying the orifice formula to the four-foot-square ungated outlet, the outflow was found to be 400 cfs for the same period that 290 cfs was recorded downstream. This meant 110 cfs was lost to channel infiltration in the 11.7 mile channel reach. Assuming an average wetted channel width equal to 75 feet, a loss rate of 1.05 cfs per wetted acre was computed. With this observed infiltration rate on Cave Creek, the following infiltration rates were chosen for the Phoenix region: main channel equals: 1.25 cfs per wetted acre, and overbank equals 0.50 cfs per wetted acre. The higher value for main channel infiltration was chosen because the 1.05 cfs per wetted acre figure was for a 290 cfs discharge. Higher discharges would produce higher hydrostatic heads and more bottom turbulence to create higher infiltration rates. The overbank material is less pervious than stream bed deposits, hence the infiltration rate for the overbank area was taken as 0.50 cfs per wetted acre. The effect of channel infiltration on a flood hydrograph is computed by reducing the entire flood hydrograph by the ratio of the discharge lost in channel infiltration to the average peak discharge in the reach. Infiltration was considered negligible for floods routed through sheetflow areas. In these areas well defined channels do not exist and the porous soil conditions associated with channels are not expected to be encountered. The effects of urbanization such as pavement and soil compaction also will limit infiltration in these areas. The discharge lost to infiltration is found by applying the infiltration rates established above to the natural channel cross section when the flow is at the average peak discharge.

Effects on Irrigation Canals

Floods in some of the study areas will inundate and cross major irrigation canals. Cave Creek will inundate the Arizona Canal (capacity of 700 cfs) and the Grand Canal (capacity of 700 cfs). Runoff from Camelback Mountain will inundate the Arizona Canal (capacity 1,125 cfs). Runoff from the South Mountains will inundate a series of smaller canals: the High Line Canal, the Western Canal, and the North and South Branches of the San Francisco Canal. Major floods from thunderstorms are likely to occur during the height of the summer growing season, when the irrigation canals are flowing at near capacity. Upon inundation of a canal, some of the irrigation water it carries will contribute to the flood. All of the canals, however, are at least partially entrenched and irrigation flow in the entrenchment will probably continue to flow down the canal. The part of the irrigation flow confined by levees will probably contribute to the flood. The quantity of irrigation water likely to be expelled from the irrigation canals is relatively small. When compared to the expected flood discharges it is negligible (less than about 2 percent of the standard project flood and less than about 10 pecent of the 25-year flood at Cave Creek and the Grand Canal). In adddition, the Salt River Project's "Supervisory Control" system allows real time monitoring of flood situations. When inundation of the canals is anticipated, inflows to the canals are ceased. Although there is a considerable lag time for the effects of gate changes to be felt, these actions should result in some reduction of canal flows at the flood areas.

Base Flow and Snowmelt

Base flow is considered negligible for much of the study area, because runoff occurs only as a direct response to high intensity rainfall. Allowance for snowmelt is inappropriate in this region for storms occurring in the summer season. Snowmelt, however, is an important factor in determining runoff originating from winter precipitation on the Salt-Verde watershed.

Stream System Analysis

The stream system analysis approach to computation of floods involves division of a study area into subbasins (which are homogeneous with respect to hydrologic and meteorologic factors) and routing and combining the flood hydrographs generated from each subbasin to determine the design flood at a desired concentration point. Subdividing a watershed into subbasins permits more accurate modeling of the runoff process, as variations in topography, urbanization, and rainfall, as well as channel shape and slope may be incorporated into the hydrologic description of the basin.

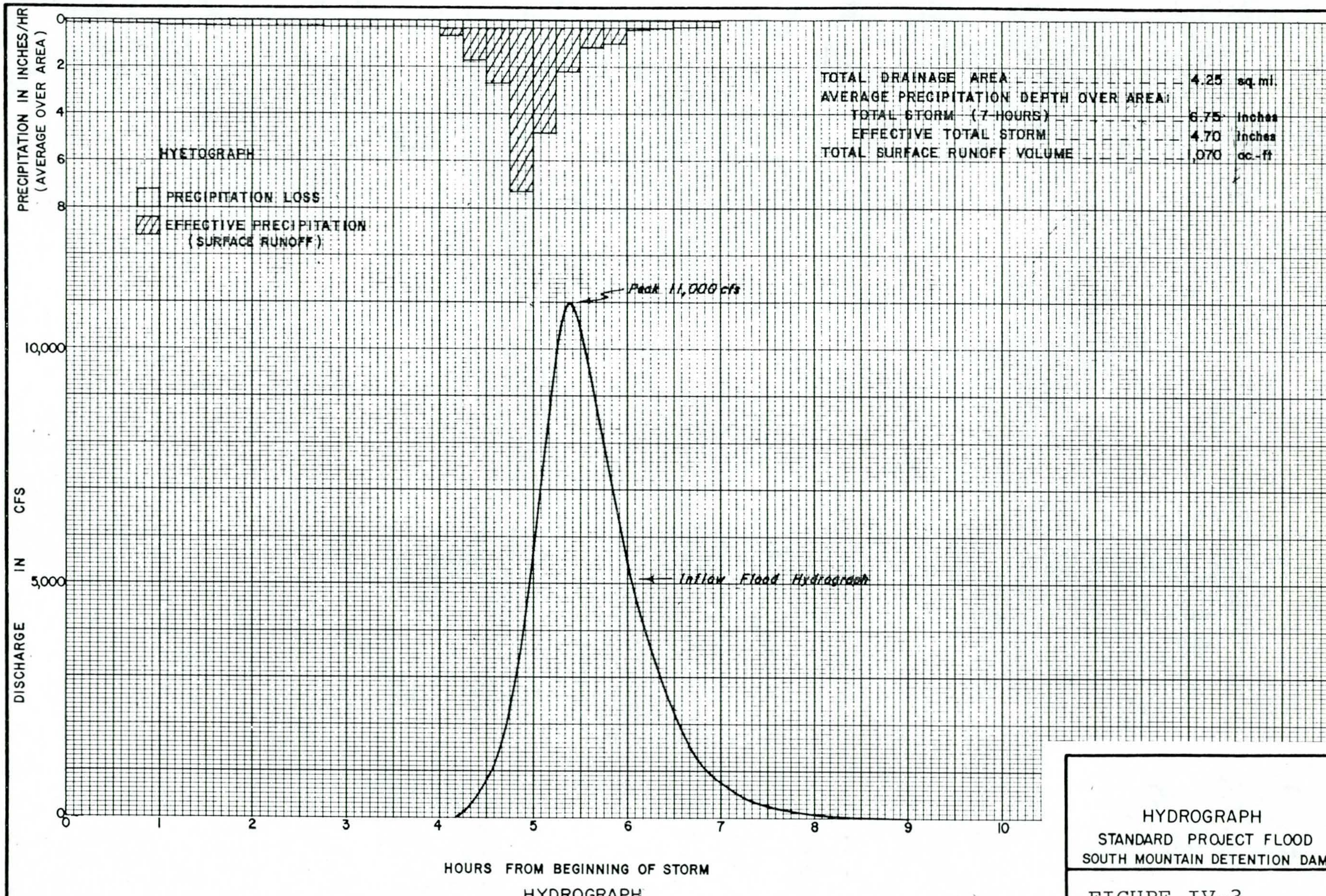
Standard Project Flood

The standard project flood is computed by centering the standard project storm in the most critical flood producing manner. Application of rainfall loss rates, depression storage, and on-site storage policies to standard project precipitation enables determination of the rainfall excess hyetograph. The rainfall excess hyetograph is then applied to the subarea runoff model to produce a subarea hydrograph. Routing (taking into account infiltration loss rates) and combining of all subarea flood hydrographs to the desired concentration point completed the computation of strandard project flood. SPF peak discharges are given in tables 1 through 5. Selected SPF peak discharges, plotted on an enveloping curve of observed peak discharges on streams in the Phoenix area are shown in Figure IV-2. Because sheetflow basins are typically less efficient in producing peak discharges than are basins with concentrated flows, the SPF's for sheetflow areas plot lower than non sheet-flow areas as shown in Figure IV-2. The somewhat high plotting position for the SPF at the South Mountain Detention Dam is a reflection of the steep basin and orographic influence in the South Mountains. The SPF inflow hydrograph to the South Mountain Detention Dam is shown in Figure IV-3.

SEDIMENT ALLOWANCE AND SYNTHESIS OF PROBABLE MAXIMUM FLOOD FOR SOUTH MOUNTAIN DETENTION DAM

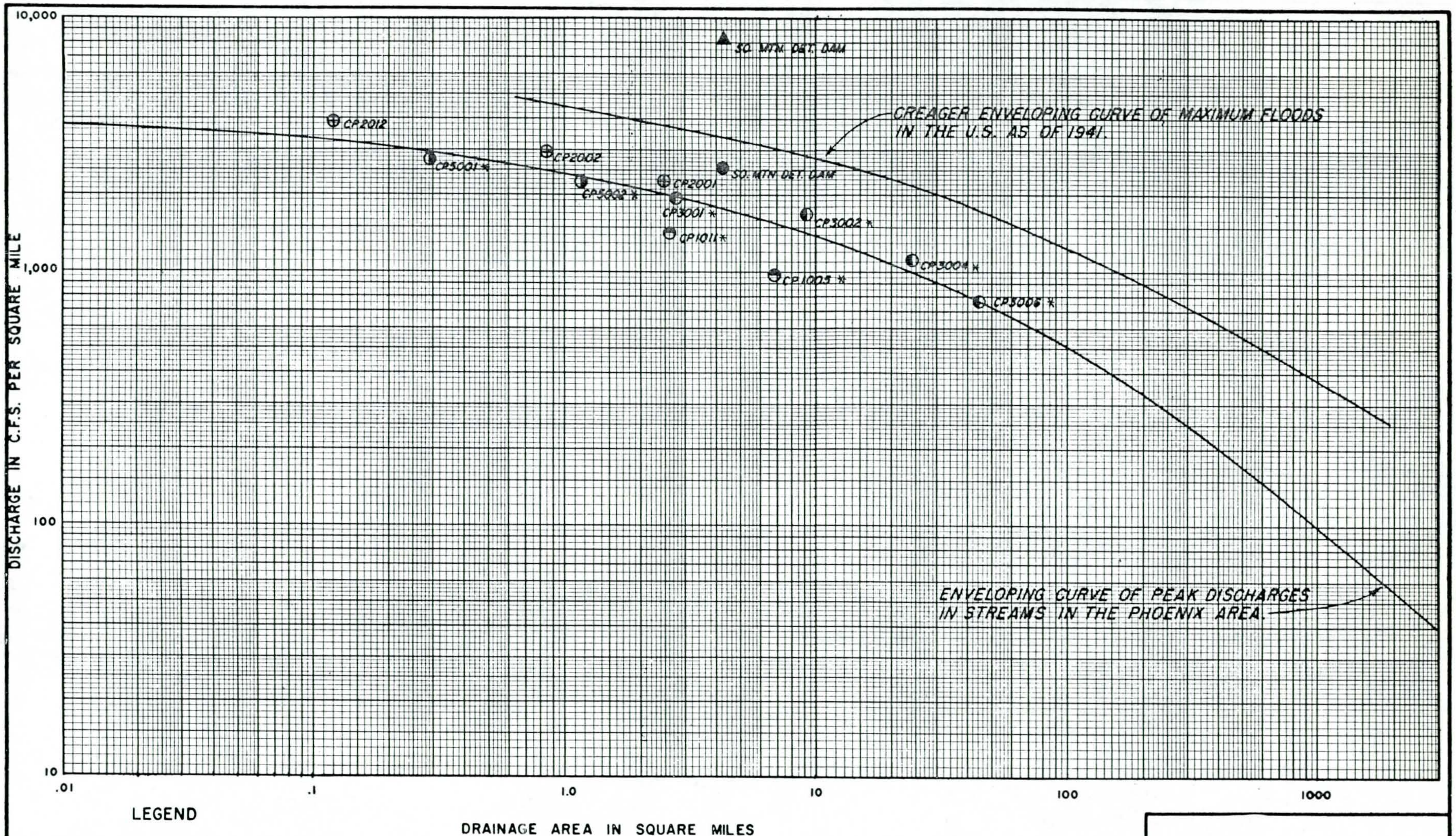
Sediment Deposition

The rate of sediment deposition in the study area was established from a determination of sediment deposited in Cave Creek Dam during a 47-year period, 1923-1970. The sediment deposition rate was found to be 0.24 acre-feet per square mile per year. However, since the survey made in 1970 does not include significant debris produced during the September 1970 and March, 1978 floods, the sediment deposition rate was increased to 0.30 acre-feet per square mile per year to compute sediment storage allowance. Considering a 100-year design period, a sediment storage of about 130 acre-feet should be allocated for the South Mountain Detention Dam.



HYDROGRAPH
STANDARD PROJECT FLOOD
SOUTH MOUNTAIN DETENTION DAM

FIGURE IV-3



LEGEND

STANDARD PROJECT FLOOD PEAK DISCHARGES
IN PRESENT CONDITIONS AT:

- GLENDALE-MARYVALE AREA
- SOUTH PHOENIX AREA
- UPPER INDIAN BEND WASH
- OLD CROSS CUT CANAL AREA
- PROPOSED SOUTH MOUTAIN DETENTION DAM
- ▲ PROBABLE MAXIMUM FLOOD PEAK DISCHARGE AT PROPOSED SOUTH MOUNTAIN DETENTION DAM
- * SHEETFLOW AREA

ENVELOPING CURVE
OF PEAK DISCHARGES

FIGURE IV-2

Definition of the Probable Maximum Flood

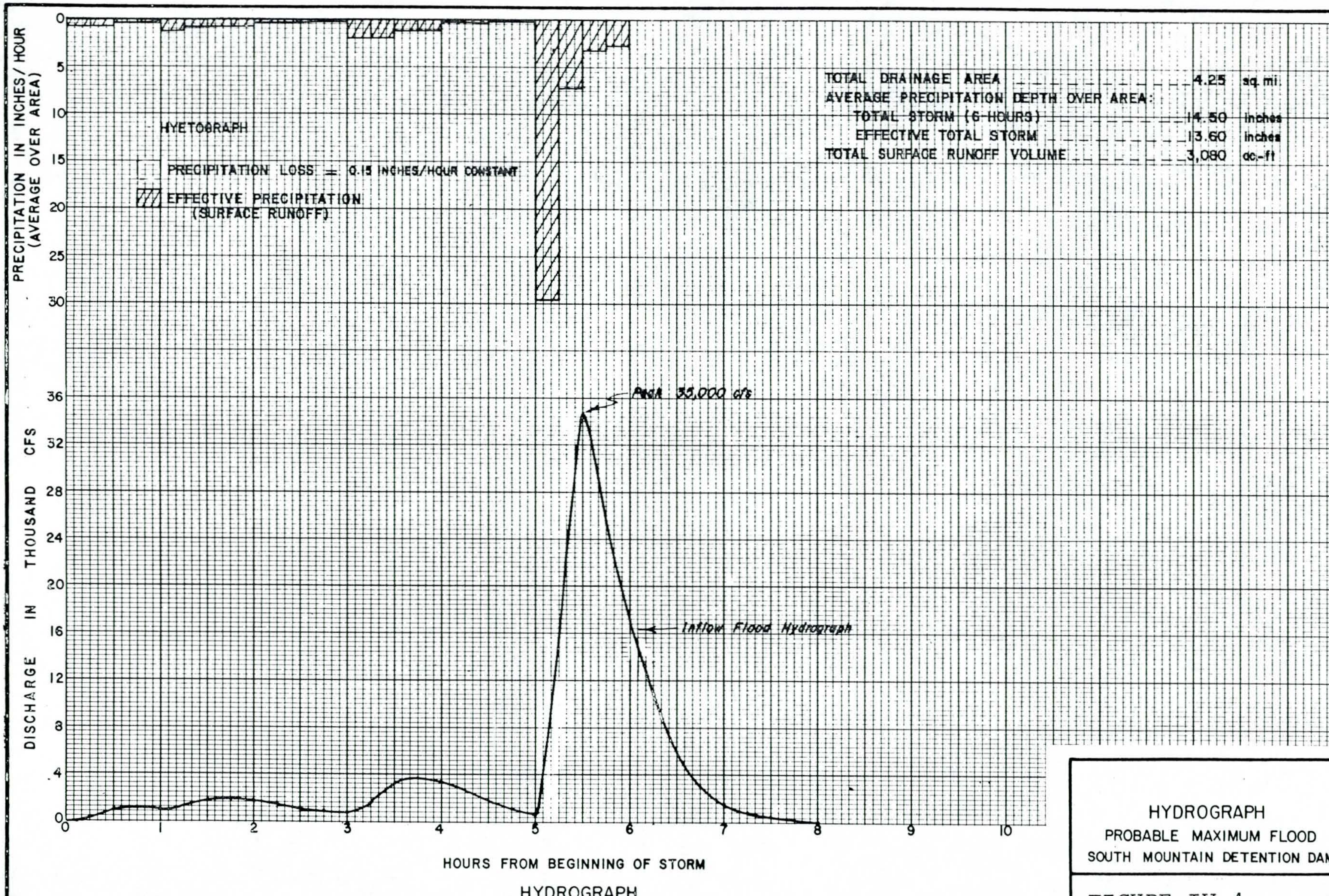
The probable maximum flood (PMF) is defined as the flood that would result if the probable maximum precipitation for the drainage area were to occur at a time when ground conditions were conducive to maximum runoff. Probable maximum flood as its name implies, is an estimate of the upper limit of flood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

Probable Maximum Precipitation

Probable maximum precipitation (PMP) is considered the practical upper limit of availble precipitable water over an area as estimated by the Hydrometeorological Branch of the National Weather Service. The preliminary draft of "Probable Maximum Thunderstorm Precipitation Estimates Southwest States" dated August 1972 prepared by the National Weather Service and amended as per a letter, dated March 29, 1973 ("Probable Maximum Thunderstorm Precipitation Estimates for Southwest States," August 1972), was used to establish local storm PMP estimates for a time period of 15 minutes as well as the incremental PMP time pattern. As with standard project precipitation, the local storm PMP with its high rainfall intensities produced the critical rainfall amounts as compared with general storm PMP for the relatively small drainage areas in this report.

Probable Maximum Flood

Computation of PMF for the thunderstorm event is accomplished in the same manner as SPF with two exceptions. First, basin lag time is reduced by 15 percent to account for the reduction in time of concentration of rainfall excess characteristics of large floods where the hydraulic efficiency of the watershed is increased by high depths of flow. Secondly, the loss rate is taken as a constant equal to 0.15 inches per hour for the entire duration of the storm. This is a minimum loss rate deemed reasonable of a watershed saturated by antecedent rainfall. The PMF hydrograph for the proposed South Phoenix Detention Dam is shown in Figure IV-4. This discharge also is plotted on an eveloping curve of observed peak discharges on streams in the Phoenix area shown in Figure IV-2. The high plotting position of the PMF is a reflection of steep drainage basins and significant orographic effect. It is also a reflection of the configuration of subareas. Three separate washes meet just above the dam, allowing for unusually fast runoff compared to that from a similar sized basin with one main stream. Consequently the high PMP plotted in



HYDROGRAPH

HYDROGRAPH
PROBABLE MAXIMUM FLOOD
SOUTH MOUNTAIN DETENTION DAM

FIGURE IV-4

Figure IV-2 appears to be reasonable and adquate. For spillway design at the damsite the water surface elevation should be assumed at spillway crest at the beginning of PMP.

DISCHARGE FREQUENCY DETERMINATION

Salt River Valley

In the absence of runoff data from comparable basins, flood frequency in the valley areas was based on rainfall frequency. The idea of determining flood frequency from rainfall frequency is not new. The intention is to estimate the flood of a selected frequency from rainfall of the same frequency. The actual relationship between frequency of rainfall and the derived flood is obscure as each part of the computational model introduces some joint probability. For this reason, frequency analysis of obseved runoff data is the preferred procedure. The basic premise adopted by the Urban Study was that, if "average" values of other parameters such as Manning's n value and loss rate are used, the frequency of the derived flood should approximate the frequency of rainfall. The rainfall parameters chosen to preserve the consistency between rainfall and runoff frequency were the maximum 15 minute, 30 minute, and 1 hour precipitation amounts. Because of the nature of summer storms in the area, these parameters are good indicators of storm severity. Summer storms generally last 12 hours or less, with most of the rain falling within 3 hours. The intense portion of the storm often lasts 1 hour or less. A comprehensive analysis of temporal patterns for summer storms in the area, formed the basis for determining the average time distribution of rainfall employed in this study. The maximum 15 minute, 30 minute, and 1 hour precipitation amounts were determined from n-year 6 hour and n-year 24 hour rainfall amounts and regression equations for finding n-year t-hour amounts presented in NOAA Atlas, Vol. VIII. N-year flood peak discharge and total storm volumes were calculated in the same manner as SPF. Constant loss rates, estimated from the "Dry Watershed" loss function, were adopted as follows:

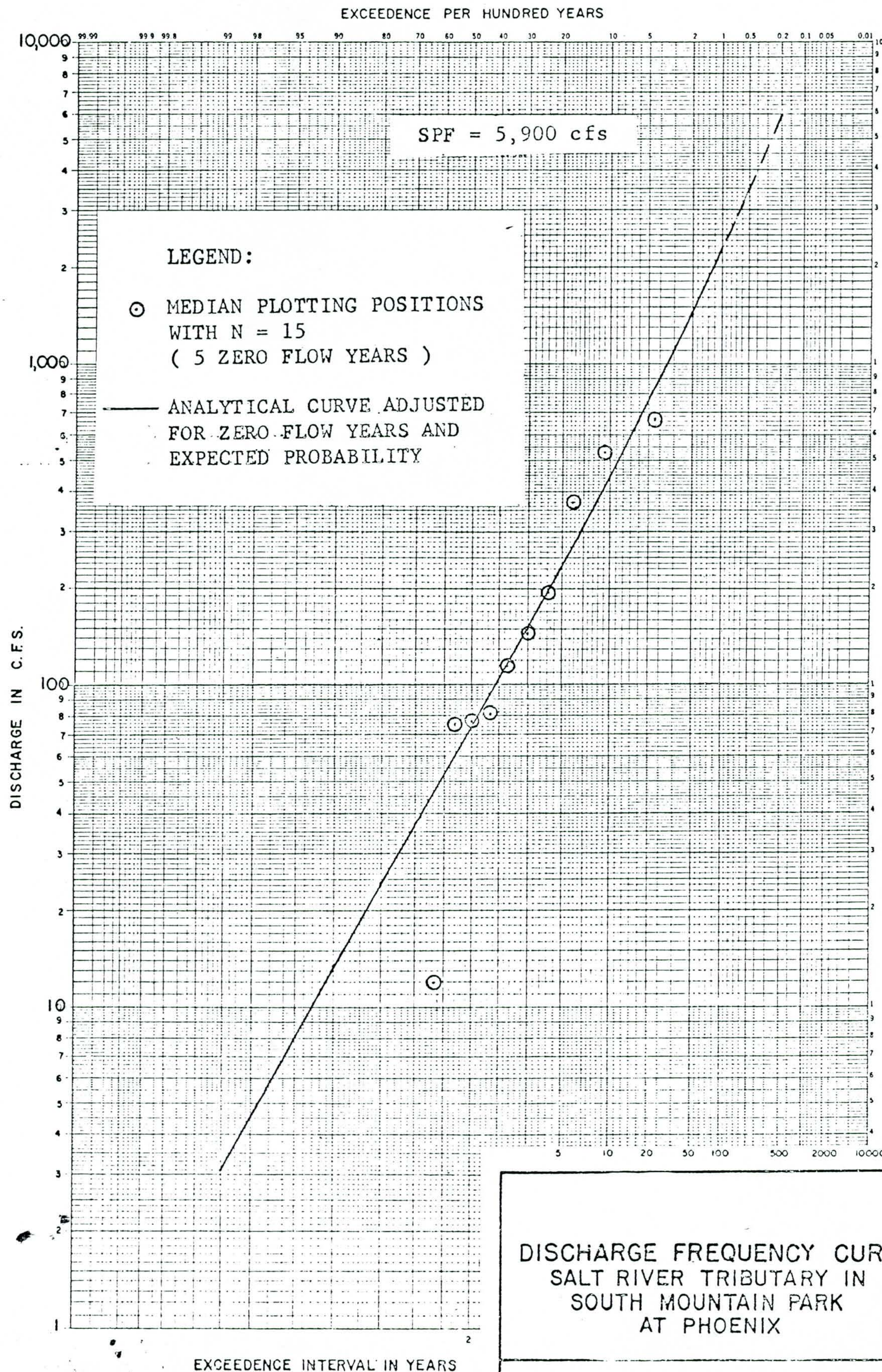
Storm Frequency Years	Constant Loss Rate Inch per hour
100	0.40
50	0.45
25	0.50
10	0.55

Subarea hydrographs were generated by these procedures. Routing and combining and the effects of infiltration were employed as necessary to compute flood hydrographs of concentration points. Discharge frequency values are listed in tables 1 through 5 in Appendix A.

South Mountain

The U.S. Geological Survey stream gage "Salt River Tributary in South Mountain Park" was used to develop flood frequencies for the steep basins in the South Mountains. The record for this gage extends from 1963 to 1976 and includes 5 years with no runoff. Criteria from the U.S. Water Resources Council's "Guideline For Determining Flood Flow Frequency, Bulletin no. 17 of the Hydrology Committee," dated March 1976 were used to establish a discharge-frequency relationship for the gage site. A generalized skew coefficient and adjustments for zero-flow years and expected probability were employed to derive the discharge-frequency curve shown in Figure IV-5. Based on this curve the SPF at the site had a return frequency of 500 years. From the frequency-curve ratios of n-year frequency floods to SPF were computed. The standard project flood hydrograph for each subarea was multipled by the respective ratio, to determine n-year peak discharges. Because additional development is not expected in these mountains the ratios for present conditions are applicable to future conditions.

For concentration points with more than one subarea routing and combining were employed as necessary. For concentration points where runoff comes from both valley and mountainous areas. Subarea hydrographs were generated by the appropriate procedure either described in this paragraph for the mountainous subarea or described in the previous paragraph for the valley subarea. Discharge frequency values are presented in table 2 in Appendix A.



DISCHARGE FREQUENCY CURVE
SALT RIVER TRIBUTARY IN
SOUTH MOUNTAIN PARK
AT PHOENIX

FIGURE IV-5

CHAPTER V

FLOOD-WARNING SYSTEMS*

OBJECTIVES OF FLOOD-WARNING SYSTEMS

Although many definitions can be applied to the concept of the word flood, a flood in an urban area may be thought of as a significant flow of water through an area in which such flow is both infrequent and undesirable. In an urban area, therefore, prevention of as many floods as possible is desired. In the absence of structural flood prevention, the warning of such floods is highly desirable. The specific objectives of flood prevention and flood warning are many, but can be considered as falling into two basic categories: 1) the prevention of damages, and 2) the saving of lives and prevention of injuries. Those affected by floods can be divided roughly into four classifications: travelers and commuters, residents, those engaged inbusiness and other indoor activities, and outdoor recreationists.

Specific objectives

Prevention of damages: This category includes, in the broad sense, not only direct property damage but also economic losses and human suffering resulting from the effects of floods. Virtually all of these problems listed as "damages" can be minimized by structural flood prevention (where such prevention is feasible). Only a limited amount of damages can be avoided by the warning of impending floods.

Saving of lives and prevention of injuries: The loss of life and the occurrrence of injury as the result of floods can be minimized either by a total structural flood prevention program or by a comprehensive and adequate flood warning system (both of which are easy to define but very difficult and costly, and in many cases impossible, to implement).

Groups Affected By Floods

Travelers and commuters: Travelers and commuters are likely to be affected by just about any type of flooding, including overland sheet flow and the inundation of any dip crossing. Damages for this classification would range from minor inconveniences (with the least amounts of runoff) up to major disruption of services. There is a possibility that commuters could become stranded for days from their places of business during moderate or severe floods.

*Note: Data for this chapter were developed during the period 1976-1977.

Damage to vehicles could result to those parked on streets or to those caught in excessively deep or fast flowing water. Loss of life and injuries would be limited primarily to persons stranded or swept away by high water in dip crossings or other places with fast moving flow. A good flood warning system would go a long way toward the prevention of vehicular damages and toward the saving of lives (except for those ignoring the warning system). Inconveniences and temporary job disruptions could be avoided only by the introduction of structures, such as bridges over dip crossings. These problems would be mitigated only partially by a flood warning system.

Residents: Flood damages to residences can be minimized, or entirely avoided in some cases, by adequate flood control structures. Such damages can also be alleviated by preplanned permanent floodproofing of individual buildings. The minimization of flood damages by flood warning systems would be limited to certain local temporary floodproofing measures and the temporary relocation or removal of portable household items. The extent of such actions would be dependent upon the amount of warning time available. Measures for saving lives and reducing injuries, although perhaps somewhat less time consuming than most of those required for the prevention of damages, would also depend upon the timeliness of the warning. Life-saving measures, such as evacuation, may become necessary in the face of a flood threat for several reasons. First there is the obvious danger of drowning. There is also the danger of serious--perhaps fatal--injury that could result from the partial or total collapse of an occupied structure caused by the force of the floodwaters or debris carried by such floodwaters. There is also the possibility of fire or explosion which can result from shorted electrical wires or broken gaslines. There are, on the other hand, numerous special problems encountered with the evacuation of residents because of a flood threat:

Inconveniences and possible losses: Evacuation from one's home always causes considerable inconvenience and hardship to the one who must leave his or her place of residence. There is even the possibility of economic losses and suffering (beyond those caused by the flooding) which could result from looting that might take place while a resident is away (despite the efforts of law enforcement officials to prevent such actions).

Additional danger to life. Even if a flood warning is received and the evacuation order is given well in advance of any flooding or local adverse weather, there are still some dangers inherent in the evacuation prodedure itself. The shock of the announcement and the moving of sick, infirm, or elderly persons can on occasion result in serious medical complications. The panic which may set in amont some individuals or crowds may endanger the lives and well-being of themselves and others. One example of this might be the recklessness with which a panicked individual might drive his vehicle in trying to escape the impending flood. If the area being evacuated is also experiencing part of the intense storm which is responsible for the flood, there is additional peril caused by possible lightning strikes, blown-down trees and powerlines, and conceivably (although rarely) large hail. If the floodwaters have already reached the area being evacuated, the risks of drowning or injury from floating debris may conceivably be greater outdoors at the time of attempted evacuation trhan they would become inside one's house later in the flood. All of these hazards must be weighed against the dangers of remaining home. It should be pointed out that an inquiry of Federal, state, county, and local officials has turned up no known incident within the Phoenix Urban Study area in which any resident has lost his or her life because of the flooding of homes, although it is likely that the potential for some fatalities could have certain floods resulted from had evacuation procedures not been implemented.

Those engaged in business and other indoor activites: This classification includes anyone in industrial or commerci l ··ildings. The fact that persons engaged in business or other activities away from their homes are usually awake, dressed, and alert tends to lessen the time necessary for evacuation. The fact that most of these individuals are mobile and healthy also reduces some of the evacuation problems and dangers. A major exception to this would be the patients on the lower floors of flood prone hospitals, where a serious problem could exist in some cases. There can also be a problem in certain relatively crowded buildings, such as large restaurants and especially theaters, where mass panic could compound the problems of evacuation.

Outdoor recreationists: Recent concepts in flood plain zoning and multiple-use of flood zones have spurred the creation of recreational "green belts" in many flood plains and even in the bottom of some of the ephemeral streams, such as Indian Bend Wash. As a consequence, people are frequently attracted to flood hazard areas. Persons engaged in outdoor recreation activities in these green belts, plus those who work in these facilities, must be warned of impending floods in time to allow for proper evacuation. In a few cases this even involves persons camping or otherwise sleeping overnight (authorized or unauthorized) in certain of these gren belt areas. Normally the time required for the evacuation of persons in recreation areas, and the problems involved with such evacuation, would not be as great as those involved with residential evacuations. There is also the relatively minor problem in these green belts of securing loose tables, trash cans, and other movable items prior to the arrival of significant running water--not only to save the items themselves, but also to prevent downstream damages or injuries.

TYPES OF FLOODS OCCURRING IN THE STUDY AREA

General floods

A general flood can be considered as any flood in which the water rises fairly gradually, the high water normally lasts for a considerable time, and the flood subsides relatively slowly. Large areas may be covered by the floodwaters.

Sizes of rivers and drainage basins: General floods are significant mostly on larger rivers, having drainage areas of 300-500 square miles or greater. In the Phoenix area these include the Gila, Salt, Verde, and Agua Fria Rivers, and to some extent, the New River. Floodflows on these rivers can equal many tens of thousands of cubic feed per second (cfs), and in some cases exceed 100,000 cfs. During a prolonged sorm, however, some of the smallest washes may experience in low runoff of considerable duration. For certain dip crossings, prolonged low flow can be a problem, causing a serious disruption of general traffic movement. Therefore, such a flow may fall into the classification of a general flood.

Causes: General floods can result from several different causes:

General storms. Most general floods in Arizona are caused by the general winter type of storms. These storms move into the region from off the Pacific Ocean between mid-October and late April, and may last from 1 day to several days, with light to moderate intensity rainfall over large areas. A few of the classic flood producing general winter storms include those of February 1891, November 1905, January 1916, February-March 1938, and Deember 1967. Some general summer storms, usually consisting of light to moderate general rain with some localized heavy thunderstorms superimposed, have been large enough to produce general flooding. Many of these types of storms are associated with tropical cyclones which move up the west coast of Mexico or through the Sea of Cortez (Gulf of California) between late July and mid-October and which may on occasion enter southern California or Arizona. Some of the better examples of this type of storm that have affected central and southern Arizona include those of September 1926, September 1939, August 1951, September 1961, September 1970, and October 1972.

Snowmelt. Some general floods in parts of Arizona can result from snow melt or, more likely, a combination of rainfall and snowmelt. They occur during winter or spring on only those rivers having very large drainage areas because of the limitations of the speed with which snow can melt. The high flows on the Salt and Verde Rivers in 1965-1966 and 1973 resulted largely from snowmelt. Some of the earlier floods, such as those of 1891, 1916, and 1938 were undoubtedly augmented by snowmelt.

Upstream releases. On streams having upstream reservoirs, such as the Gila, Salt, and Agua Fria Rivers, flow in the channel can occur as the result of releases or spillage from the upstream reservoirs. In the cases of San Carlos Reservoir (Coolidge Dam) on the Gila River, Lake Pleasant (Waddell Dam) on the Agua Fria River, and the Salt River Project reservoir system on the Salt and Verde Rivers, water is not normally released down the rivers in large quantities from these reservoirs but is stored for conservation and recreation uses. Large releases have been made only during floods after and reservoirs have filled. Such a condition occurred on the Salt River briefly in December 1965 - January 1966 and again in the spring of 1973, and would likely have occurred during certain very wet earlier years, such as 1891 and 1916, had the reservoirs existed then. It should be pointed out that San Carlos Reservoir, Lake Pleasant, and the Salt River Project reservoir system are all water-conservation reservoirs and are not designed to operate for flood control. During much of the time these reservoirs are not full, however, and the storage of incoming water serves to act as a de facto type of flood control. When the reservoirs become full, though, one cannot depend upon them for flood control purposes. In 1973, in order to minimizew inconveniences and other problems downstream on the Salt and Gila Rivers (including the Phoenix Urban Study area), the Salt River Project did begin to release water gradually from their reservoir system even before their reservoirs had filled. This became possible because of the knowledge of the quantities of water still available in the upstream snowpack--knowledge gained from the existence of an expanded early warning meteorological system and from an improved system of monitoring snowpacks. Knowing that the yet-to-melt snow would eventually fill their reservoirs, the Salt River Project engineers were able to release water down the Salt River more gradually, and over a longer period of time, than would have occurred had their reservoirs spilled out of control. There could be times, however, when such gradual releases may not be possible throughout an entire season, and some flooding may become unavoidable.

Travel times: Because of the longer stream lengths characteristic of larger rivers, the travel times associated with general floods are normally in excess of 2 to 4 hours. Sometimes several days are involved. There is normally considerable warning time for any general floods on the major streams.

Flash floods: A flash flood is one in which there is a sudden rise of water--sometimes in the form of a wall--moving rapidly down a normally dry or low-flowing stream. The waters of flash floods are often laden with boulders, debris, and other solid material. Such a flood, although frequently very destructive, is usually only of short duration. A flash flood may occur as an isolated phenomenon or during general flooding.

Drainage basin sizes: Most flash floods occur on streams having drainage areas of less than approximately 400-600 square miles, or they can occur as overland flow. Flash floods are notoriously dangerous in confined canyons. It is possible, although rare, for a flash flood to occur on a larger river, where conveyance capacity is usually large.

Causes: Nearly all flash floods in the southwestern United States are the direct result of intense local rainstorms, sometimes referred to as cloudbursts. These very heavy convective showers, which are usually in the form of thunderstorms, may occur as isolated phenomena in a general field of moist, unstable air flowing into Arizona from the south and east, or may be embedded within general storms, such as tropical storms. Nearly all such intense local storms in Arizona occur during the summer or early fall months. A few of the more intense flood producing local storms of this type in and near the Phoenix area include those of: 19 August 1954 in the Queen Creek drainage area between the Superstition Mountains and Superior, 16 August 1963 in the Glendale-Maryvale Area, 14 September 1969 in Tempe, 5 September 1970 in the upper Tonto Creek area (occurring in conjunction with a very heavy general storm), and 22 June 1972 in northeast Phoenix.

Travel times: Because of the short distances over which water travels in the drainage basins and channels of the smaller streams, and because the steeper terrain in some of these smaller drainage basins will cause high velocities, the travel times for flash floods are usually less than 2 hours, and in some cases, much less than 1/2 hour.

PREDICTABILITY, DETECTABILITY, AND WARNING TIME

General floods

The detection of rising water upstream on the larger rivers by human observations and by stream gages, coupled with the larger travel times associated with these larger streams and the gradual rises of water levels in a general flood, helps cases to provide considerable warning time prior to the arrival of a general flood peak. Advanced prediction and early detection of the possible causes of general floods can provide additional warning time.

General storms

Since general storms normally move rather gradually into Arizona from the west or northwest in winter and from the south in summer, their progress and development over ocean and land areas can nearly always be monitored by satellite, radar, surface and upper air observations, and other meteorological methods. Computer forecasts of these storsm are normally available and reasonably accurate. Some advance notice of the arrival of a general storm in the region can be provided by the National Weather Service. Once the storm begins, it can virtually always be detected and reported by human observations, rain gages, and radar. Although precise rainfall intensities throughout each drainage basin are never known, sufficient overall information is usually available about the magnitudes of general storms that judgements about the size of the resulting general floods can be made.

Snowmelt

Even more gradual than the floods created by general rain storms are the floods generated by snowmelt. Improved methods of observation and monitoring of snowpack conditions and of atmospheric conditions whch may be conducive to snow melt, along with increased efforts in the monitoring of these conditions by the Salt River Project, with the cooperation of the Soil Conservation Service, the National Weather Service, the US Geological Survey, and other agencies, has helped in the detection and prediction of snowmelt runoff.

Upstream releases: The existence of upstream reservoirs on the major rivers generally reduces and delays flood peaks. In addition, the constant, careful monitoring of reservoir levels, inflows, outflows, and upstream conditions, provides early warning to downstream areas.

Flash floods

Most of the flood warning problems facing the Phoenix Urban Study area come from locally generated flash floods.

Intense local storm rainfall: The general meteorological conditions favoring the occurrence of intense local thunderstorms are frequently predictable several hours or longer in advance. The exact location and time of individual thunderstorm cells, however, is normally impossible to predict with any reasonable accuracy. As for the detection of such local storms and their precipitation, there are several methods, some more timely than others:

Radar: The use of radar is often helpful in the detection and location of intense local storms. The heights of the cloud tops and the areas of heaviest precipitation can be measured with a fair degree of accuracy by radar. Some information is also available as to the approximate intensities of the rainfall. Radar, however, may miss some local storm cells and may underestimate the intensities of certain others. The National Weather Service, including the forecast office at Phoenix Sky Harbor Airport, currently monitors all available radar reports of precipitation phenomena and relays pertinent storm information to the Salt River Project and to various county and local agencies.

Ordinary recording and nonrecording rain gages: These types of gages are useful in the post storm analyses, and quite useful--if read daily--in the detection of general storm magnitudes for the purpose of general flood prediction. They are, however, of little or no use in supplying real-time information about intense local storms for the purpose of flash flood prediction. Such gages would be of some use if they were reliably and almost continuously monitored by a human observer during heavy rainfall (an almost impossible task, especially for gages installed in remote, unpopulated upstream areas). Since the intense portions of many local storms are frequently missed by conventional ground based rain gage networks, such networks cannot be completely relied upon to provide early warning of impending flash floods.

Telemark rain gages: Rainfall measuring devices which relay (via radio or telephone) rainfall amounts from a remote site upon interrogation area of considerable use in general storms. They are of use in intense local storms only if the existence of the storm is known (from radar or other means) and the gage is interrogated. More useful are telemark gages of this type with automatic alarms. These alarms will sound whenever the rainfall intensity at a gfage exceeds a predetermined level. This predetermined intensity should be one at which flodoing could be produced if their intensity should continue or be exceeded. The problem of missing intense local cells with a conventional network also exists with the telemark-alarm rain gages unless hundreds of such gages were to be installed throughout pertinent basins in the Phoenix area.

Rain gages with automatic telemetry. Rainfall measuring devices which automatically and remotely report significant changes in accumulated rainfall are very useful in detecting local storms. As with the previous two types of rain gages, a drainage basin must be representatively instrumented by such gages in order that sufficient information can be provided for the prediction of impending flash floods.

Sudden rises in upstream water levels: Any sudden and significant rise in the water level upstream in a channel creates the potential for a serious disaster downstream. Since a large upstream rise of water level usually foretells of a downstream flood, there is normally a better accuracy of prediction by the monitiring of upstream water levels than there is from rainfall monitoring. The trade-off here, however, is that the lead time of any flood warning for downstream areas is signifantly less when based strictly upon reports of upstream water levels than when based upon reports of upstream rainfall intensity.

Visual observations: The qualitative visual observation of water levels in an upstream portion of a channel is naturally of some use to downstream interests. This is of use for flash flood prediction only if such observation is continuous (including at night), or if there is a reliable and timely method by which the observer would be dispatched immediately after the detection of heavy rainfall or of a significant rise in stream level. Some sort of rainfall or stream level sensing alarm could be used to alert an observer.

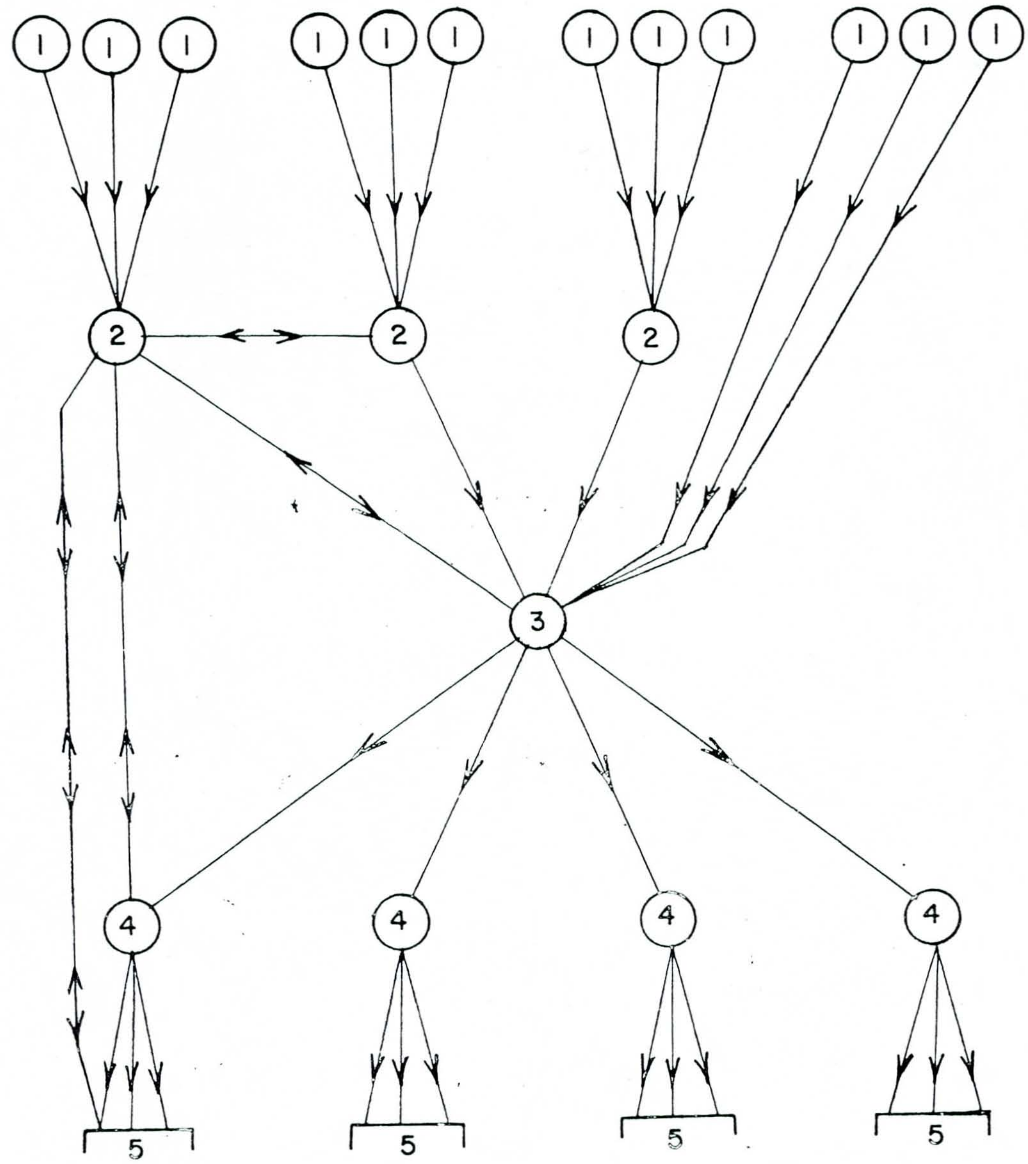
Ordinary, telemark, and telemetry stream gages: Virtually the same arguments can be made about the various type of stream gages that have been made about the corresponding types of rain gages. The number of automatically reporting stream gages required upstream from the prediction target area would not be nearly as great as would be the number of automatic rain gages, but several stream gages (or at least simple stram level sensing alarms) would be necessary in order to provide both accuracy and timeliness of flood warnings for downstream areas.

COMMUNICATION TO THE PUBLIC

No matter what measuring or sensing devices and other equipment are used in the detection of upstream occurrences of heavy rainfall and/or high stream levels, there remains a major task of translating such information into flood warnings and of communicating such warnings to the public in a timely manner. An effective communications system is probably the most important part of an effective flood warning system. There are several different types of communications systems which can be utilized. Some of these systems are more reliable; other are less reliable but more timely.

Components in the communicating systems

The communications of flood warnings can be viewed as consisting of a system with components and links. Figure V-1 is a schematic diagram of an idealized network of communications systems which could be used in the greater Phoenix metropolitan area. A complete system would consist of five basic components. The simpler, more direct, and faster systems would consist of only two or three components. These are diagrammed in Figures V-2 and V-3. The various components of the flood warning communications network are listed below.



LEGEND

(1) Data Sensors, Observers

(2) Meteorological-Hydrologic Agencies

(3) Central Emergency Control

(4) State and Local Public Service and Special Agencies

[5] Public

PHOENIX URBAN STUDY
FLOOD-WARNING SYSTEMS

COMPREHENSIVE FLOOD WARNING COMMUNICATIONS NETWORK

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

FIGURE V-1

Component No. 1: Sensor/observer: This component of the system is the human observer or mechanical device which measures the rainfall intensity or stream level. There may be one, a few, or many sensing stations in any one system. Such tools as weather radar and other meteorological and hydrologic information also would fit into this general category.

Component No. 2: Meteorological/hydrological agency: This component is an applied scientific and/or engineering agency. The National Weather Service is the most prominent agency comprising this compoent. Other agencies which would fit this category include the US Geological Survey, the Soil Conservation Service, the Bureau of Reclamation, the Corps of Engineers, the Salt River Project, and certain state and local agencies. The role of this type of agency in a flood warning system or network is to collect, process, and interpret data from the sensing devices and other sources. In some cases this agency issues flood watches or warnings or some other statement of potential flood conditions.

Component No. 3: Central Emergency Control: This agency would act as the "nerve center" of a flood warning system or network of systems. It would collect all warnings issued by the meteorological/ hydrologic agency (Component No. 2) and certain data transmitted directly from the sensors/observers (Component No. 1), and would relay the warnings or other information to the various agencies of Component No. 4. For ths agency to be effective in the warning of general floods, some official of this agency must be on standby 24 hours. In order to be effective in the warning of flash floods, this agency must maintain a crew on full operation 24 hours. At the present time there is no such agency which serves as a "nerve center" for all general and flash flood warnings or other messages affecting the Phoenix Urban Study area. The National Weather Service presently serves this function in part through its issuance and dissemination of flood (including flash flood) watches and warnings plus other weather messages (see paragraph entitled, "NAWAS, NWWS, NOAA Weather Radio," and the four subsequent paragraphs). The National Weather Service Forecast Office at Phoenix Airport, however, serves the entire State of Arizona and cannot operate a major flood warning network for only the Phoenix Urban Study area. Furthermore the National Weather Service is not staffed and budgeted to serve as a communications nerve center for the direct dissemination of flood warning information to many other different agencies. The Salt River Project has its own Supervisory Control center for the operation of its canal system (see paragraph entitled, "Salt River Project"). This control center also notifies other agencies in the event of any emergency or other unusual conditions. The prime concern of the Salt River Project, however, is its own canal system and water supply, and not the issuance and dissemination of flood warnings for the entire Phoenix Urban Study area. The agency which would

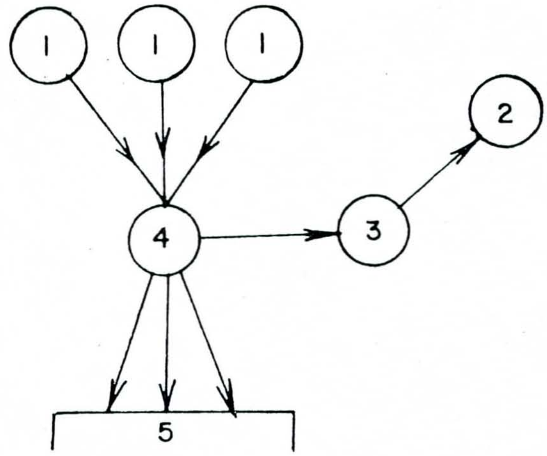


Figure 2 FROM SENSORS TO PUBLIC SERVICE AGENCY TO PUBLIC

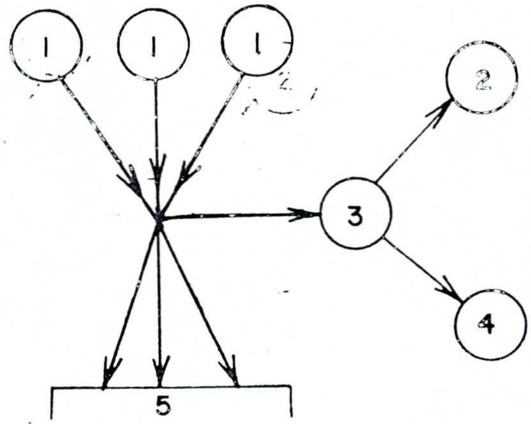


Figure 3 DIRECT FROM SENSORS TO PUBLIC

LEGEND

1 Data Sensors, Observers

2 Meteorological – Hydrologic Agencies

3 Central Emergency Control

4 State and Local Public Service and Special Agencies

5 Public

PHOENIX URBAN STUDY
FLOOD-WARNING SYSTEMS

LOCAL FLOOD WARNING COMMUNICATIONS SYSTEMS

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

serve as the Central Emergency Control (Component No. 3) for a flood warning network in the Phoenix area would most likely need to be a local agency, perhaps a Maricopa County agency, operating 24 hours every day--at least during all periods in which there was any chance whatever of heavy rain or flooding. The only such agency presently fitting this description is the Maricopa County Sheriff's Office. The primary duties of this office, however, concern law enforcement and not flood control or flood warning. The Maricopa County Department of Civil Defense and Emergency Services does not maintain regular 24-hour office operation, although its director or some other official can be reached at all times. It would appear that finding a suitable agency willing to serve on 24-hour duty as the Central Emergency Control for a greater Phoenix network of flood warning systems may be difficult.

Component No. 4: State and local public service and special agencies: Warnings and other information moving through the communications systems in the flood warning network (see Fig. V-1) are relayed to each State, county, and local government, law enforcement, highway, traffic, and other concerned public service agency in the Phoenix Urban Study area, along with the area's radio and television stations, and certain special agencies and interests. The law enforcement agencies are all on duty 24 hours. The county, city, and special agencies are required to have crews on standby. Actions taken by highway and traffic crews consist mostly of the placing of barricades, detour signs, etc., at the numerous dip drossings and other low areas that are flooded or flood locations to enforce traffic control, and if necessary, to order evacuations. Various special agencies take the proper actions to secure their specific interests from the effects of the flood (such as, e.g., the removal of picnic tables and other loose items from green belt recreational areas along Indian Bend Wash, etc.).

Component No.5: Public: The final component of any such communications system or network is the public itself. Each segment of the public, depending upon its location, the activity in which it is engaged, and the flood conditions receives a specific flood warning message and takes actions, as appropriate.

Links in the communications systems: The rapid and reliable conveyance of data or warnings through a communications system from the sensing devices all the way to the public is absolutely vital to any flood warning system. It is no coincidence that many communications links tend to fail during times of stress and emergency. Telephone lines frequently go out during storms or floods. Some radio transmitters or receivers and their antennas fail when they become wet. They also can be destroyed by lightning strikes. An overloading of human links in the system frequently results in errors or loss of timeliness. It is essential that a viable communications system be preplanned and tested so that the system will work during severe storms and floods, and it is best that ordinary telephone lines be used as little as possible in such a communications system. Radio telemetry from the sensing deices and two-way radio voice or data communications between components in the system would be a satisfactory type of communications links in most cases. Some sort of radio system as a back-up for telephone or teletype communcatios, and vice versa, would be ideal. A reliable minicomputer, high-priority time sharing computer terminal, or other data processing device which could save valuable time in the interpreting of data or in the performance of necessary calculations would be a necessary investment for such a system. This minicomputer would logically be located at the Central Emergency Control (Component No. 3) of the flood warning system or network. Some of the tasks which such a minicomputer could perform include: 1) the processing of raw data signals and the translation of the signals into a digitized form for readout and/or printout; 2) the sounding of an alarm based upon different predetermined threshold rainfall intensities or stram levels at the various gages, or upon certain combinations or different rainfall intensities and/or stream levels; 3) some simple predictions of flood hydrographs, based upon a hydrologic model for the watershed, using input of rainfall and/or upstream runoff data. An electronic or manual back-up system to such a computer/processor should be provided so that the raw data signals could not become "locked up" in a "down" computer in such a manner that they could not be retrieved or interpreted.

Possible types of communications systems

It can be seen in Figures V-1-3 that there are a number of possible communications systems within a flood warning network. These range from complete systems involving all five components (fig. V-1) to the simple and direct systems involving only two or three components (figs. V-2 and 3) in the transmission of information from the sensors to the public. The more complete systems would be used when conditions provide substantial lead time between the detection of a flood potential and the likely occurrence of the flood. The more direct systems would be required when advanced notice is at a minimum and very rapid dissemination of the flood warning is essential. The latter would be more likely to occur with floods on smaller drainage basins, such as those of small creeks and washes. For the smaller basins an alarm, triggered directly by high rainfall intensities or stream levels, would ring in the office of a local 24-hour public service agency (see fig. V-2). This agency would then take actions to close dip crossings or other flooded streets, and order evacuations if the conditions warranted. Urgent radio and television broadcasts would be issued immediately for the potentially more serious flood situations, and evacuation warnings would be blared over bullhorns from police cars and heliocopters where possible. Other components in the flood warning network would also be notified of the potential event, but the first priorty would be the immediate protection of the public. For the very smallest basins a direct link between a sensing device and a warning device could be implemented (fig. V-3). An example of this would be a prominent flashing red light or a flashing sign reading "FLOODED" at a dip crossing--a light or sign that would be directly hooked up to an upstream sensing device. One could also conceive of an upstream sensor that is linked directly to a siren, horn, or loud buzzer in a residential neighborhood, which could be used to alert the residents to evacuate. A prerecorded voice message could even be activated to sound over loudspeakers. It should be cautioned, however, that there are many pitfalls with this type of system. Not only must such a system be free from mechanical failures and vandalism in order to be credible (a near impossibility), but any sudden evacuation of residents (especially, e.g., in the middle of the night) would be subject to the many additional dangers discussed in the subparagraph entitled, "Additional danger to life."

AUTOMATIC DETECTION/WARNING SYSTEMS IN OPERATION;
PROTOTYPE SYSTEM: EQUIPMENT AND COSTS

NAWAS, NWWS, NOAA Weather Radio

Currently in operation at National Weather Service Forecast Offices throughout the nation are several communications systems for weather messages and warnings. Each system has telephone or teletype circuits or radio transmission designed to cover a certain regional area.

National Warning System (NAWAS): This hotline telephone system is used for all weather warnings and other urgent messages. In Arizona one NAWAS system covers the entire state. In addition to the National Weather Service offices throughout Arizona, the state's Department of Public Safgety and all county sheriff's offices are on the circuit. The state and county offices of Emergency Services are also on the circuit but receive messages only during their weekday business operations or when, during emergencies already called, they are on special 24-hour duty.

NOAA Weather Wire Service (NWWS): This state-wide teletype circuit carries, among other weather information, all special weather statements, all watches, and all warnings. Included on this circuit are all National Weather Service offices in the state, many Arizona radio and television stations, the Department of Public Safety, the state's Office of Emergency Services, the Salt River Project, Arizona Public Service utilties, Arizona State University, and others. A bell on each teletype terminal can be activated whenever an important message is treansmitted.

NOAA Weather Radio: This is a special VHF FM radio channel for the exclusive broadcast of continuous weather information within limited regions. In the Maricopa County area the frequency is 162.55 megahertz. All weather warnings, in addition to watches and more routine weather information, are broadcasted on this radio channel. Anyone who wishes may monitor this circuit. Radio receivers having the capability for this channel are readily available. A special Tone Alert (1000 hertz) is also sounded just prior to the issuance of urgent weather warnings. This turns on every radio receiver equipped with a special device that is activated by this tone.

Special radio links: The National Weather Service Forecast Office in Phoenix also maintains a special radio link with both the state and the Maricopa County Offices of Emergency Services. This circuit is for coordination and feedback of significant weather information. It is in effect only during the regular business hours of the emergency services offices, except during emergency periods in which these offices are in operation 24 hours. If any sudden notification of the chiefs of these offices is required during nights or weekends, they are telephoned at home by the National Weather Service.

Existing National Weather Service Automatic Flash Flood Warning Systems

The first automatic alarm system of this type was installed by the National Weather Service in 1972, and many more have been installed and operated with success across the United States.

Elements: This simple system has three main elements, linked by electrical circuitry:

Sensor. An automatic water level sensor consists of an enclosed float device which activates an electric current when lifted to a critical height by rising water. This device may be mounted on a permanent structure such as a bridge support.

Intermediate station. This station is the power supply for the system and is normally located several miles downstream where both electric power and telephone service are available.

Alarm station. From the intermediate station the signal is sent to the alarm station, located in a facility operated 24 hours every day, such as a police or sheriff's station or firehouse. The alarm consists of a flashing lamp and an audible signal. When the alarm sounds, the local public safety officials assume the responsibility of taking appropriate action to protect lives and property of the public.

Cost. Cost of equipment and installation vary, depending upon the location and accessibility. Costs of the system are shared, by signed contracts of agreement, between the National Weather Service and local communities.

Examples. Several examples of National Weather Service flash flood warning systems are described:

Gallup, New Mexico. An automatic flood warning device was installed in 1976 on the South Fork of the Puerco River at the western boundary of Fort Wingate, east of Gallup, New Mexico. This system is designed to provide approximately an hour of advanced flood warning to Gallup and vicinity by sensing high water levels on the river about 7 miles upstream from Gallup. The readout device and alarm are located in the local sheriff's office.

Willow Beach, Arizona. Another such automatic device installed at Willow Beach on Lake Mohave of the Colorado River serves as a flash flood warning for a camping area. A stream level sensing device upstream on a dry wash will give about 10 to 15 minutes of advanced warning time for the campground. An evacuation procedure plan must be carefully worked out ahead of time, and all campers must be made familiar with the warning system and the evacuation plan before they are allowed to camp at the grounds.

Other warning systems. There are several other National Weather Service warning systems in use in the southwestern United States. These include three Automatic Digital Recorder Binary Decimal Transmitter (ADRBDT) devices installed on the Gila River near Redrock, New Mexico, upstream from Duncan, Arizona; on the Little Colorado River near Woodruff, Arizona, just upstream from Holbrook; and on the Puerco River at Chambers, Arizona, some distance upstream from Holbrook. The Little Colorado River device has been bullet-proofed. A telemark stream gage has also been installed on the San Francisco River near Glenwood, New Mexico, upstream from Clifton, Arizona. All four of these flood warning systems can be interrogated by telephone, but none have automatic alarm capability.

More elaborate hydrologic computation systems in existence: A number of complete river and reservoir level computation systems, some with forecasting capability, are in existence in various regions of the United States. New systems are being installed all the time. The requirements for some multipurpose hydrologic computation systems ae highly complex, such as those for the vast Columbia River reservoir system or for the numerous rivers, reservoirs, canals, and aqueducts of the Tennessee Valley Authority (TVA). Other requirements are much simpler, such as those for reservoir systems of certain Corps of Engineers districts which are operated for flood control purposes only. Three examples of hydrologic computation systems in the western

United States are very briefly examined. It should be noted that each of these systems is designed for water control operations and not for the specific purpose of issuing flood warnings to the public.

Columbia River reservoir system: The cooperative Columbia River Forecasting Unit is a multimillion dollar river and reservoir prediction and regulations organizations operated jointly by the North Pacific Division of the Corps of Engineers, the Portland River Forecast Center of the National Weather Service, and the Bonneville Power Administration. The heart of this operations center is the Streamflow Synthesis and Reservoir Regulation (SSARR) computer model and several auxiliary computer programs. This SSARR system is able to monitor the highly complex aspects of precipitation, snowmelt, evaporation, infiltration, soil moisture, base flow, channel percolation, reservoir inflow and outflow, and other elements throughout the more than 250,000 square miles of highly variable terain comprising the Columbia River drainage, and to integrate these many variables into a series of surprisingly accurate real-time predictions of runoff and reservoir levels within the basin. This SSARR model has performed with a great deal of success, even during periods of severe drought and flood, and it has achieved international recognition as an example of a very useful and worthwhile hydrologic computation system.

Salt River Project: The Salt River Project operates a rather sophisticated hydrologic system, known as Supervisory Control, for monitoring and controlling water distribution from the Salt River through its 138-mile canal network to more than 238,000 acres of water users in the greater Phoenix area. With the aid of a small process computer and a complex system of electronic links, a single operator, sitting in front of a large console, can control the entire Salt River Project water distribution system by remotely operating the many gates and pumps located throughout the Salt River Valley. The total cost of the entire Supervisory Control and electronic telemetry system was approximately $3.3 million.

Corps of Engineers, Los Angeles District hydrometeorological system: Recently installed, and still in the testing phase, is a moderately sophisticated hydrometeorological operations system for analyzing rainfall, river flow, and reservoir levels and inflow for the purposes of reservoir regulation. A data processor is preprogrammed to sense critical rainfall intensities or river levels and to set off an alarm. During off duty hours this alarm sounds in homes of key personnel. The cost of the entire system, including installation, is estimated to be around $700,000 dollars.

Other existing automatic systems: A number of counties, cities, and other local communities in the southwestern United States Operate various types of meteorological and/or hydrologic data measuring systems, some with an appreciable network of automatically or semiautomatically reporting sensors, and others with just the simplest of equipment. Most of the county flood control districts in southern California maintain a fairly dense network of precipitation and stream gages, many of which are recording and some of which have telemark or automatic telemetry capability. The City of Phoenix, Arizona has a rather simple system of seven automatically reporting rain gages, located at fire stations scattered throughout the city. This system is to be expanded to nine such gages. Each gage in the system is linked via telephone lines to a recorder in the City Engineer's office. Increments of precipitation measured at a gage sets off a loud click at the recorder site, but there is no actual alarm in the system and no method of 24-hour monitoring. The cost of each rain gage is approximately $400, and about $200 per month is required for rental and maintenance of the communications lines.

Self-Help Program and other methods of human reporting: Under the guidance of the National Weather Service, programs for self-help procedures are being set up in cities and other communities in the southwestern United States. Such flood warning systems have recently gone into effect in Kingman and Lake Havasu City, Arizona, and a similar self-help program is being implemented in Clark County (including Las Vegas), Nevada. In this program, fence post type rain gages, each costing around $2.00, are issued to individuals willing to cooperate in the self-help operations. During any significant precipitation these individuals are expected to telephone changes in the accumulated rainfall in to the National Weather Service. This helps in the detection and location of possible flood producing precipitation. There are some problems in generating sufficient enthusiasm for such self-help programs, but the enthusiasm can be encouraged by the announcement of participants' names on television weather programs, etc. In the greater Phoenix area there are about 100 rain gages at the homes of Salt River Project employees. About 50 to 60 percent of these will report in to the Salt River Project during any storm of consequence.

Some flood warning procedures in operation in the Phoenix area: Once the danger of potential flooding has been detected and interpreted, and a flood warning of other advisory has been issued, the various county, city, and other agencies take certain actions. The Maricopa County Highway Department, the City of Phoenix, and other communities have programs set up by which crews immediately begin to monitor traffic conditions in potential problem areas and to erect barricades at troublesome road-stream intersections. City of Scottsdale crews monitor Indian Bend Wash green belt and other areas of potential flooding. All city and county offices keep in close touch with the National Weather Service through NAWAS, NWWs, NOAA Weather Radio, telephone, or other means of communications (see paragraph entitled, "NAWAS, NWWS, NOAA Weather Radio," and the four subsequent paragraphs).

Prototype Systems

River Forecast Center, Sacramento, California: Several different prototypes of automatic flood warning systems are currently being examined by the joint Federal-State River Forecast Center in Sacramento, California. A number of different engineering systems design and manufacturing companies are under consideration for possible contract. What is desired most by the River Forecast Center is a warning system consisting of three to five tipping-bucket rain gages, each located in a representative portion of a watershed, a power-supply system and antennas for radio transmission, and a continuous readout (e.g., a teletype machine) at a base station or central office. A minicomputer at this base station or central office could be added for tie-in purposes, and perhaps to analyze incoming precipitation data and evaluate the flood threat in a particular basin through the use of a simple hydrologic model. The minicomputer could be programmed to sound and alarm by activating the bell on the teletype receiver(s).

RECOMMENDATIONS FOR FLOOD-WARNING SYSTEMS IN THE URBAN STUDY AREA

Because of the many different types of flood problems encountered in the greater Phoenix area, the different types of detection and warning systems available, different community needs, no simple recommendations for a single comprehensive flood warning system or network of systems can be made here. Each individual drainage basin and each community has its own requirements, and these should be considered separately.

Public Education

Basic to any flood warning system is the education of the public about the many dangers of flooding and about the types and functions of available flood warning systems, particularly the one being installed in the local community. Public apathy and deliberate disregard are probably the greatest contributors to fatalities and serious injuries that result from floods. A step-by-step education program must be undertaken by local counties and communities in order to minimize flood-related deaths and injuries. First, the public must be interested in the problems of flooding and must be made aware of the many hazards of floods. Second, the public must be informed as to the proper steps to take--whether at home, at work or school, on the road, or engaged in other activities--in order to avoid dangers of flooding.

Stages of Flood Alerts

It is recommended that some sort of graduated scale of flood alert levels be set up for the Phoenix Urban Study area. A suggested numbering system for the various stages of flood readiness is listed as follows:

Stage 0...No significant flood threat to any part of the Phoenix Urban Study area withinthe next 3 to 4 days.

Stage 1...A slight chance of significant precipitation in the greater Phoenix area and/or significant releases from upstream reservoirs within the next 3 or 4 days. General notification of concerned agencies recommended.

Stage 2...Flood potential moderate or high (including use of the description "Flash Flood Potential High" by the National Weather Service) for at least some portion of the Phoenix Urban Study area. Precipitation capable of generating flood producing runoff having a high probability of occurring within the next 24 hours, or significant releases from upstream reservoirs likely within the next 24 hours. Key personnel at concerned agencies available for duty. Cursory monitoring of flood warning systems in effect.

Stage 3...General flooding likely in the Phoenix Urban Study area within the next 12 hours, or Flash Flood Watch in effect for all of part of the study area (significant flash flooding is possible in some portion of the region), or minor urban flooding (including some flooded dip crossings) already in progress in some portion of the study area. Key personnel at concerned agencies on duty or standing by for immediate call. Close monitoring of flood warning systems in effect.

Stage 4...Flood Warning or Flash Flood Warning in effect (flooding is imminent) for some portion of the Phoenix Urban Study area, or reports from sensors of flood warning systems indicate that flooding is imminent, or moderate flooding already in progress in some portion of the study area. Key personel at concerned agencies on duty; other personnel standing by for possible mobilization.

Stage 5...Severe or extensive flooding confirmed to be in progress in one or more portions of the Phoenix Urban Study area. Concerned agencies fully mobilized for flood duty.

Four Alternative Types of Flood Warning Systems

For any flood warning system or network of systems there is a whole range of possible alternatives which could be considered, depending upon what the Federal Government and the individual communities are willing to spend. For the Phoenix Urban Study area four alternatives are examined. These range in complexity from the continuation of the existing flood warning programs to the implementation of a complete, elaborate flood warning network covering the entire urban study area. Any alternative flood warning system should be designed and installed in such a manner that it can later be upgraded to the next level of complexity with a minimum of wasted expenditures. In other words, the total cost of a more complex system upgraded from a simple system should not be significantly greater (except for inflation) than would be the cost of the complex system built all at one time.

No addition to existing system: This alternative would involve no installation of new equipment or significant modification of existing flood warning programs in the Phoenix Urban Study Area. Perhaps items like the stages of flood alerts (see paragraph entitled, "Stages of flood alerts") could be adopted under this alternative. The existing flood warning programs could be continued throughout the entire Phoenix Urban Study area, or they could be continued for certain drainage basins or communities, while other alternatives are adopted for other portions of the study area.

Minimum, least expensive warning systems: Either for the entire urban study area or for certain specific flood prone areas, a minimum warning system might be the desired alternative. Such a system in each basin would likely consist of one or a few elementary automatic rain or stream gages with an alarm that would sound in a central office or directly at a potential trouble spot, such as a dip crossing. An example of a minimum automatic system (without even an alarm) is the group of rain gages operated by the City of Phoenix (see paragraph entitled, "Other existing automatic systems"). Some of the Self-Help programs sponsored by the National Weather Service and most other programs of direct human observation and reporting would also fall into this minimum and least expensive category.

Systems of moderate extent and complexity: A major step up from a minimum system would be one in which a drainage basin or group of basins is covered by a number of automatic rain and stream gages. Some of these gages might have alarm-only capability, while others could be capable of remote quantitative precipitation or stream height readouts, along with an alarm. On a basin of intermediate size, perhaps the upstream portions could be instrumented with one or more automatically reporting alarm rain and/or stream gages for the detection of floods or flood producing storms. In the downstream portions of the basin a self-help program of human observations could be set up, mostly for the purpose of verifying, and reporting any changes in upstream conditions as they move downstream. One or more detection/warning systems of the type described here could perhaps be electronically tied together through a centralized minicomputer. Systems of moderate comnplexity could perhaps be installed in certain highly flood prone portions of the Phoenix Urban Study area, while only minimum flood warning systems or no flood warning systems at all are implemented in other portions of the study area.

Complete, elaborate network: The ultimate flood warning program for the Phoenix Urban Study area would be a complete network of flood warning systems, all linked together through a computer and sophisticated console located in a Central Emergency Control officer manned 24 hours every day. The network of detectors would consist of amy fully automatic rain and stram gages, with coverage on every river, creek, and wash of significance within the Urban Study area. Each gage would automatically report quantitative precipi-tation or stream height data to the central computer. This computer would process all incoming data and print out the result, not only in the Central Emergency Control office but also at a number of remote terminals located in such offices as the National Weather Service, the state and county emergency services, local sheriff and police offices, the Salt River Project, etc. A complete, computerized hydrologic model with flood prediction capability could be run for each affected basin in the study area on a real time basis. A sophisticated alarm system, possibly with more than one alarm stage, would be activated by the computer, based upon certain combinations of rainfall intensities and/or stream levels at the various gages within each drainage basin or upon certain predicted downstream water levels computed by the hydrologic model. The alarm in each case would automatically sound in the Central Emergency Control office and in all Federal, state, and county offices on teh circuit of remote terminals, as well as at the Supervisory Control center of the Salt River Project. It would also sound in the office of each city or local community in which the flood alert is effective.

Distribution of costs: The decision as to what type and complexity of a flood warning system (if any) is to be installed in a particular basin or community would be up to the local communities themselves. It is likely that some agreements and contracts between the Federal Government and local interests for sharing of equipment and installation costs of such systems could be made. The operation and maintenance of such systems would likely be entirely up to the local communities.

Tentative Recommendations for Specific Watersheds

Some of the feassible flood warning system in the specific watersheds of the Phoenix Urban Study area are discussed in the following paragraphs, and a few preliminary recommendations for systems of various levels of complexity are made.

Gila River, main stem: The only flooding of significance on the Gila River through the southwestern corner of the Phoenix Urban Study area would be general flooding, with considerable advanced warning time available in virtually every case. The only possibilities for improvement of flood warnings on this river might be the installation of a telemark with alarm or automatic telemetry on each of the two US Geological Survey recording stream gages (nos. 09479500 and 09489000) presently located on the Gila and Santa Cruz Rivers respectively, each just above the confluence of these two rivers.

Salt River: Grantie Reef Dam to Gila River; Verde River: from below Bartlett Dam to Salt River: Any important floodflows on these rivers will be governed by releases or spillage from the upstream reservoirs operated by the Salt River Project, or from Orme Dam, if built. Any conditions leading to the necessary release of water from these reservoirs will be monitored carefully over a long period of time, and considerable advanced notice should always be given prior to any such releases.

Verde River below Bartlett: There is, however, the possibility of significant flow into the Verde River below Bartlett Dam (the lowest Salt River Project dam on the Verde River). This water would come mostly from Camp Creek and/or Sycamore Creek as the result of thunderstorm produced flash floods. The potential for flooding along the edge of Fort McDowell Indian Reservation exists. If flooding in this area is considered to be a problem in the near or distant future, then perhaps some sort of automatically reporting rain and/or stream gages (with alarm) in the Camp Creek and Sycamore Creek watersheds might be desirable. On Sycamore Creek a good location for an automatic stream gage would be the site of the present US Geological Survey stream gage number 09510200. Farther downstream on the Verde, at the Beeline Highway (State Route 87) overcrossing, the US Geological Survey presently operates a telemark stream gage with alarm (no. 09511300). This could also be tied into the Phoenix Urban Study area flood warning network.

Salt River below Granite Reef Dam: Downstream from Grantie Reef Dams several of the Salt River road crossings are dip crossings, and a number of others consist of low-flow culverts. Here almost any flow down the Salt River could cause traffic disruptions and possibly pose the potential for vehicular damage. It would appear, though, that any significant flows on the Salt River through this area would have joined the river above Granite Reef Dam, located approximately 10 river miles upstream from the closest significant crossing on the Salt River: that of North Country Club Drive (Route 87) leading northward out of Mesa. In such a case, the rise in water would likely be detected by Salt River Project personnel as it passed Granite Reef Dam, and approximately an hour warning time could be given before the water reached North Country Club Drive and other nearby crossings. Any flow in the Salt River that should be generated over drainages between Granite Reef Dam and the mouth of Indian Bend Wash would likely be relatively small because of the limited drainage areas involved. Floods generated by local thunderstorms would probably dissipate as they travel downstream on the Salt River toward southwest Phoenix. Thus no additional automatic rain or stream gages are recommended for flood warning purposes along the Salt River through Phoenix other than those on Indian Bend Wash (see paragraph entitled, "Indian Bend Wash").

Agua Fria River, New River, and Skunk Creek below dams: The Agua Fria River is subject to possible general flooding resulting from spillage from the upstream Waddell Dam. Plenty of advanced notification should be available prior to any flooding of this type. The same argument will apply to the New River Dam and to Adobe Dam on Skunk Creek, once these projects have been completed. Significant runoff generated on any or all of these streams below the dams is likely to come from local thunderstorms. As discussed in the subparagraph entitled, "Future Projects," additional runoff will be diverted by the Arizona Canal Diversion Channel into the Skunk Creek - New River -Agua Fria River system. Most of the land along the floodways of these streams is agricultural or undeveloped, with only scattered urbanization. Under the new flood plain zoning regulations, future development in these floodways will be limited primarily to agricultural and other nonurban types of land usage. Nevertheless, some warning is desirable prir to the arrival of floodwaters in these areas, especially flash flood waters. Damages to equipment for farming and other operations can be prevented or minimized by an hour or more of advanced flood warning. There are also numerous dip crossings of roads along these streams, and an advanced warning of any flows in these streams would be very beneficial to the convenience and the safety of travelers. Thus the installation of automatically reporting rain and/or stram gages in the middle and upper portions of the drainage basins of these three streams (below the dams) is worthy of consideration. Several National Weather Service rain gages and US Geological Survey stream gages already exist in the drainage basins of these three streams below the dams. It is recommended that automatically reporting equipment be installed at each of these gages, and that certain additional gages be installed.

Cave Creek: The flood warning problems on Cave Creek can be divided into two reaches of the stream:

Cave Buttes Dam to Arizona Canal Diversion Channel. This area is essentially undeveloped at this time, and flood plain zoning will control future development in the floodway. Nevertheless, some warning for motorists traversing Cave Creek via its dip crossings, would be desireable. Since the entire length of Cave Creek between the dam and the Arizona Canal is only about 10-1/2 miles, travel times down this portion of Cave Creek are relatively short. Travel times down the primary tributaries to this segment of Cave Creek, such as Moon Valley Wash, are even shorter. Therefore an automatically reporting stream gage on Cave Creek about 3 or 4 miles below Cave

Buttes Dam, along with a representatively located rain gage in the drainage basin of each primary tributary, would appear to be adequate as a minimum detection system for this portion of Cave Creek. Additional rain and stream gages would provide for a better and more complete flood detection system. Because of the relatively short warning times available, a communications system involving only Components 1, 4, and 5 would likely be best suited to flood warnings in this area.

Arizona Canal to Black Canyon Highway. As discussed in the paragraph entitled, "Urbanized washes," this portion of Cave Creek presents significant problems. Most of the historic creek bed is highly urbanized, so that flooding occurs frequently. The impoundment of water by the Black Canyon Highway embankment can cause extensive ponding over a large area, with depths of several feet in places. A flood warning system for this lower Cave Creek area is badly needed, according to engineers of the City of Phoenix, the county of Maricopa, and other agencies. Since the area is so well urbanized, the detection of runoff by stream gage would be difficult. The placement of one or more automatically reporting alarm rain gages over the drainage area is probably the best solution. A communications system consisting of Components 1, 3, 4, and 5, or even a complete 1-2-3-4-5 system (fig. V-1), would probably suffice for the warning of any build-up of water to significant depths in the impoundment area north of Black Canyon Highway. On the other hand, any warning of flowing water through the urbanized portions of the Cave Creek channel would likely require a system of Components 1, 4, and 5 (e.g., fig. V-2). An additional warning would be needed for the very infrequent occasions in which the flow down upper Cave Creek cannot be handled by the Arizona Canal Diversion Channel and ends up spilling over the diversion channel and traveling down lower Cave Creek. This could be accomplished by the automation of US Geological Survey stream gage number 09512400, located on Cave Creek less than 1 mile upstream of the Arizona Canal.

Indian Bend Wash: Most of the channel and flood plain of Indian Bend Wash is being developed as green belt recreational areas. Persons using these areas, plus those traveling across the wash (many of them through dip crossings), require advanced warning of impending high water through the channel. Flow down Indian Bend Wash is monitored by the Salt River Project at the intersection of the wash and the Arizona Canal. Aiding in this monitoring is the US Geological Survey recording stream gage (no. 09512100) on the Wash at Indian Bend Road, a fraction of a mile upstream of the canal. Above and below the canal some additional monitoring of rainfall and runoff conditions is desired. Since inflow into the wash is primarily overland flow and not from well defined tributary washes, the selection of rain gages (automatically reporting, with alarm) representatively located within the drainage areas instead of stream gages in the channel might be the wiser choice in this case, at least for a minimum flood warning detection system. Perhaps one stream gage between Indian School Road and Thomas Road could help to "fine tune" any warning already under consideration. A communications system consisting of Components 1, 3, 4, and 5, or perhaps only 1, 4, and 5, would likely be required in order to provide adequate warnings to persons in the Indian Bend Wash flood plain.

Drainage north of dams and CAP Aqueduct: North of the Central Arizona Project aqueduct and the proposed New River, Adobe, and Cave Buttes Dams, there is a considerable drainage area for the headwaters of the various rivers, creeks, and washes from Agua Fria River on the west to Indian Bend Wash on the east. In this area the communities of New River, Cave Creek, and Carefree are subject to occasional flooding. The remainder of the area is largely undeveloped at the present, and the development of new permanent residences in the floodways of these streams will not be permitted in the future. There is, however, already significant rangeland use and increasing agricultural development in this area. Most roads in the region have numerous dip crossings. Some development of recreational facilities is also underway, with more planned. In addition there are also a number of trailer and mobile home parks in the flood plains of these streams and in areas of potential flooding from overland flow. An especially prominent example of this threat to trailers and mobile homes exists along portions of Scatter Wash, a tributary to Skunk Creek just below the site of Adobe Dam. Warning of impending floods is, therefore, also desired for these areas, especially in the intermediate and more distant future. These areas are almost totally ungaged for either rainfall or streamflow, and the headwaters of the streams are almost completely unmonitored. Therefore, several automatically reporting alarm type rain gages and stream gages, properly located in the drainage areas of these various streams, would be of considerable help to present and especially future use of these northern portions of the study area. A communications system of components 1, 3,4, and 5 would probably be adequate for most of this region except for the smaller washes, where a 1-4-5 system might be required.

Waterman Wash and its tributaries: Waterman Wash, which is located just south of the far southwestern corner of the Phoenix Urban Study area, runs generally northward through the eastern portion of Rainbow Valley between the Sierra Estrella and Maricopa Mountains and flows into the Gila River just southeast of Buckeye. The land usage in this region, including the community of Rainbow Valley, is largely agricultural at the presnt time, and is increasing in development. The drainage area for Waterman Wash and its numerous tributaries is quite large, and thus the main stem of the wash is subject to occasional large scale general flooding. It would also appear conceivable that at times of quite heavy general runoff (such as that resulting from a tropical storm), a considerable amount of flow from Vekol Wash (which is normally a tributary to the Santa Cruz River) could, through the build-up of sand and/or debris, possibly become diverted as sheet flow across several miles of relatively flat desert terain between and south of the Booth Hills and the Haley Hills (near latitude $32^{0}58$'N, longitude $112^{0}13$'W) and end up flowing into the Waterman Wash drainage. Such a diversion could possibly increase the general floodflow in Waterman Wash by an appreciable amount. On the other hand, many of the tributaries to Waterman Wash originate on the steep slopes of the mountains surrounding the valley and have quite small drainage areas. Hence they are capable of producing significant flash floods from intense local thunderstorms. For this valley, therefore, a flood warning system would be very useful. One or more automatically reporting stream gages on the main stem of Waterman Wash a number of miles upstream from the community of Rainbow Valley, plus several automatically reporting rain gages in the upper portions of several key tributary drainages, would likely be the bestr combination of detectors to warn against both general floods and flash floods in the valley. A communi-cations system consissting of Component 1, 3, 4, and 5 would probably be the best for the general floods on Waterman Wash, while a system with only components 1, 4 and 5 would likely be required for the smaller tributary washes.

Drainaes west of Agua Fria River: The portion of the Phoenix Urban Study area west of the Agua Fria River and north of the Gila River, extending from north of Beardsley to west of Buckeye, is subject to occasional flooding from washes originating in the mountains northwest of the study area. Except where protected by McMicken Dam, the Buckeye Dams, and the White Tanks Retarding Structures, a series of flood warning systems may be desired to warn persons in those areas of possible high runoff from these washes, as well as from floods locally generated within the study area. A series of automatic alarm type rain gages is recommended within, and in the drainage above, all ares for which flood warning is deemed desirable. Communi-cations systems consisting of Components 1, 3, 4, and 5 are recommended for the washes having longer watercourses, while systems of only Components 1, 4, and 5 would be recommended for the shorter washes.

Smaller urban and suburban washes: There are many small and very small ephemeral washes in the Phoenix Urban Study area. One important group of these is in the region just north of the Arizona Canal between Cave Creek and Indian Bend Wash. Another is in the large region north of the Central Arizona Project aqueduct.

Still another significant area of this type is the south Phoenix region from Tempe to Laveen, which is subject to periodic flooding from many very small washes originating in the South Mountains and flowing northward toward the Salt River. In addition, the numerous small tributaries to Waterman Wash (paragraph entitled, "Waterman Wash and its tributaries") would also fall into this category. For all of these smaller washes a very rapid response warning system would be required. With travel times often less than 30 minutes and sometimes less than 15 minutes, the most direct warning systems possible are necessary.

Dip crossings. For some of the dip crossings, a communcations system consisting of Components 1, 4, and 5 with a single rain gage (automatic, with alarm), or perhaps a direct 1-5 communications system with a single stream gage, would probably be required. In the latter case, an automatic alarm-type water level sensor, whch is located several hundred yards upstream of a dip crossing, and which triggers a flashing red light or other warning device at the crossing, may in some cases be about the only way to provide adequate warning of high water.

Recreational areas. Whenever hevy rain falls in a portion of the basin of a small urban or suburban wash, at least some rain should fall over the other portions of the basin. Therefore, in the various green belt and other recreational areas located within the floodways of these smaller washes, the same rain storm responsible for the potential flooding will likely discourage the continuation of most recreational activities and encourage people to take shelter or leave the areas. (In such areas, structures which could be used as shelters from rain should not be built in the floodways of any washes.) The intensity of the rain at such a recreational facility could probably also be used as a signal to officials and employees at the facility to move loose objects such as trash cans out of the flood plain, to the extent that time permits. Some sort of alarm device would still be desired, however--even one with only 5 to 10 minutes of advanced warning time. This could come either from an automatic rain gage representatively located in the drainage, reporting directly (a 1-5 communications system) or through a local police or fire office (a 1-4-5 system), or else from a directly reporting stream gage (1-5 system) located upstream on the wash.

Evacuation of residents. The evacuation of residents from the flood plains of these small washes would usually not be possible within the time between the first signs of heavy rainfall over the drainage and the occurrence of flooding. Therefore, systems to warn or evacuate the residential public are not recommended for the smaller washes. It would still be desirable if structural flood protection alternatives are not adopted, to have some sort of flood detection system in the basin of each small urban or suburban wash where the flood problem is considered critical, so that agencies providing relief to the flooded areas would be notified of the problems as soon as possible.

Overland flow: The problem of overland flow must be treated in much the same way as is runoff on the smaller washes. The lead times between the onset of heavy rainfall and the occurrence of flooding are usually short. Detection of the flood potential would be made primarily by automatic rain gages. It is conceivable, though, that a water level sensor with a downstream alarm could be implemented at one or more key locations on the side of a certain street or highway where deepwater quite frequently flows. Evacuation of persons engaged in recreation and other activities may be possible in most cases (except where large crowds are gathered, such as in a theater) if a reliable 1-4-5 or 1-5 rain gage detection/communications system can be set up. The evacuation of residents from the threat of moving overland floodwaters would be nearly impossible, but on the other hand, should almost never be necessary. Evacuation of residents from the threat of deep ponding of water caused by accumulated overland flooding may become necessary and may be possible to carry out in some areas. It should be cautioned, however, that by the time that such an evaucation procedure can be initiated some shallow ponding of water has likely already occurred, thus rendering the evacuation itself more difficult and perhaps dangerous. It is recommended that plans for possible evacuations from the threat of flooding be considered only in areas where there will be sufficient warning time for the successful completion of such evacuations. If adequate warning time cannot be provided, then structural flood control alternatives should be considered.

CONCLUSIONS

For the Phoenix Urban Study area many different types of potential flood problems exist, and each must be treated separately. For general flooding on most of the large rivers, adequate warning of impending floods should quite esily be provided by agencies operating the upstream reservoirs. Therefore, rather few, if any, additions to the existing flood warning systems on these rivers are likely to be required. Such additions, for the refinement of flood monitoring on the larger rivers, might be desirable in a complete and elaborate flood warning network, but they should not be considered as high-priority for minimum flood warning systems. At the other extreme of the spectrum, there may be a danger to residents from the flooding of the smaller urban and suburban washes, but here the available warning times would be so short that evacuation of these residents would likely be impossible in most cases. Thus other alternatives for the protection of residents in the flood plains of small washes are recommended. For most of the flood problems in the study area, however, some sort of automatic warning system would appear to be both feasible and desirable, and should be considered. Perhaps a few simple systems, such as those currently being installed by the National Weather Service, should be tried in the Phoenix Urban

Study area. These could later be joined by other simple systems around the region, wherever the flood problem is considered to be greatest. They should be constructed and installed so that they can still later be combined into a system of moderate complexity, and perhaps eventually into a single comprehensive flood warning network with a fully opeational Central Emergency Control. Each community within the Phoenix Urban Study area should consider its flood problems and should recommend what types of flood warning systems it would like to see installed in the area of its jurisdiction. Frequent consultations and conferences among Federal, state, county, and local agencies concerned with the Phoenix area should take place, and the recommended plans for flood warning systems in the various portions of the study area should be periodically updated.

APPENDIX A

Hydrologic Data

TABLE 1-A

Peak Discharges & Total Storm Runoff Volumes

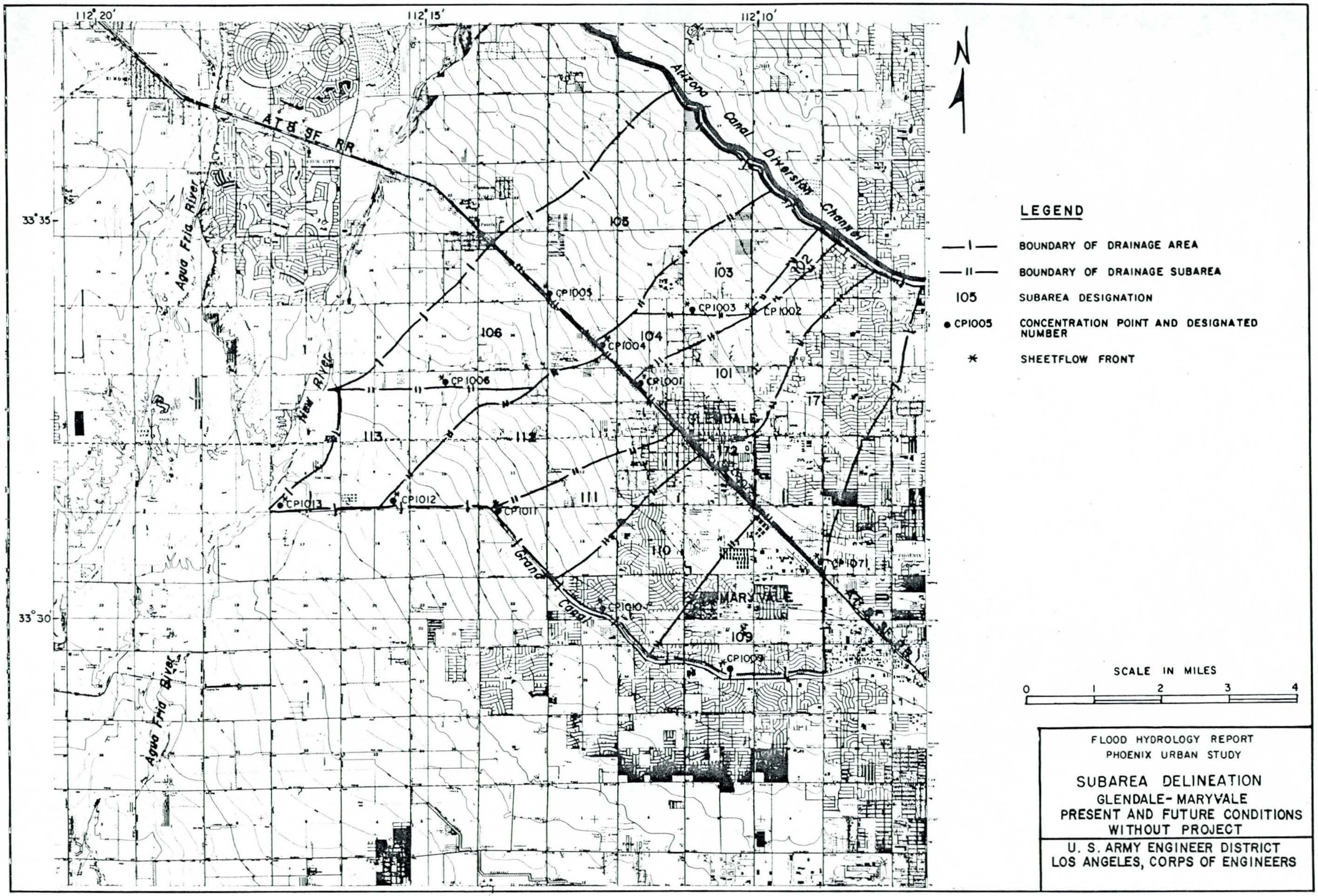
Glendale-Maryvale, Present Conditions Without Project

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
1001*	NE side of AT&SF RR near Northern Ave.	4.07	6,300 (1,030)	3,000 (455)	2,300 (360)	1,700 (280)
1002*	South of Olive Ave. & 51st Ave. intersection	0.36	710 (85)	280 (30)	180 (25)	100 (16)
1003*	Near Olive Ave. & 59th Ave. intersection	2.50	3,200 (490)	680 (140)	320 (70)	140 (40)
1004*+	NE side of AT&SF RR near 67th Ave.	7.42	4,000 (550)	0 (0)	0 (0)	0 (0)
1005*+	NE side of AT&SF RR near 75th Ave. & Olive Ave. intersection	13.97	6,100 (1,565)	800 (190)	170 (55)	0 (0)
1006*	South of Northern Ave. between 91st Ave. & 83rd Ave.	4.29	3,500 (685)	600 (125)	120 (45)	55 (25)
1071*	NE side of AT&SF RR near S. 7th Ave. & East Ave. intersection	0.50	1,300 (130)	750 (60)	600 (50)	480 (45)
1072*	NE side of AT&SF RR & Camelback Rd. junction	5.40	7,300 (1,320)	3,500 (625)	2,800 (515)	2,200 (415)
1009*	(N. Side) above Grand Canal between 50th Ave. & 51st Ave.	3.85	7,500 (1,045)	3,600 (445)	2,900 (370)	2,300 (300)
1010*	Above Grand Canal between 75th Ave. & 67th Ave.	4.53	6,800 (1,115)	3,500 (550)	2,700 (435)	2,100 (335)
1011*	Above Grand Canal between 83rd Ave. & 75th Ave.	2.61	3,700 (550)	1,100 (200)	570 (120)	280 (60)
1012*	Above Grand Canal between 91st Ave. & 83rd Ave.	11.60	4,300 (1,780)	2,000 (645)	1,500 (455)	1,100 (250)
1013*	Above Grand Canal between 107th Ave. & 99th Ave.	7.17	2,700 (1,095)	400 (165)	100 (25)	50

Values in parentheses, (1020), indicates total storm runoff volume in acre-feet.

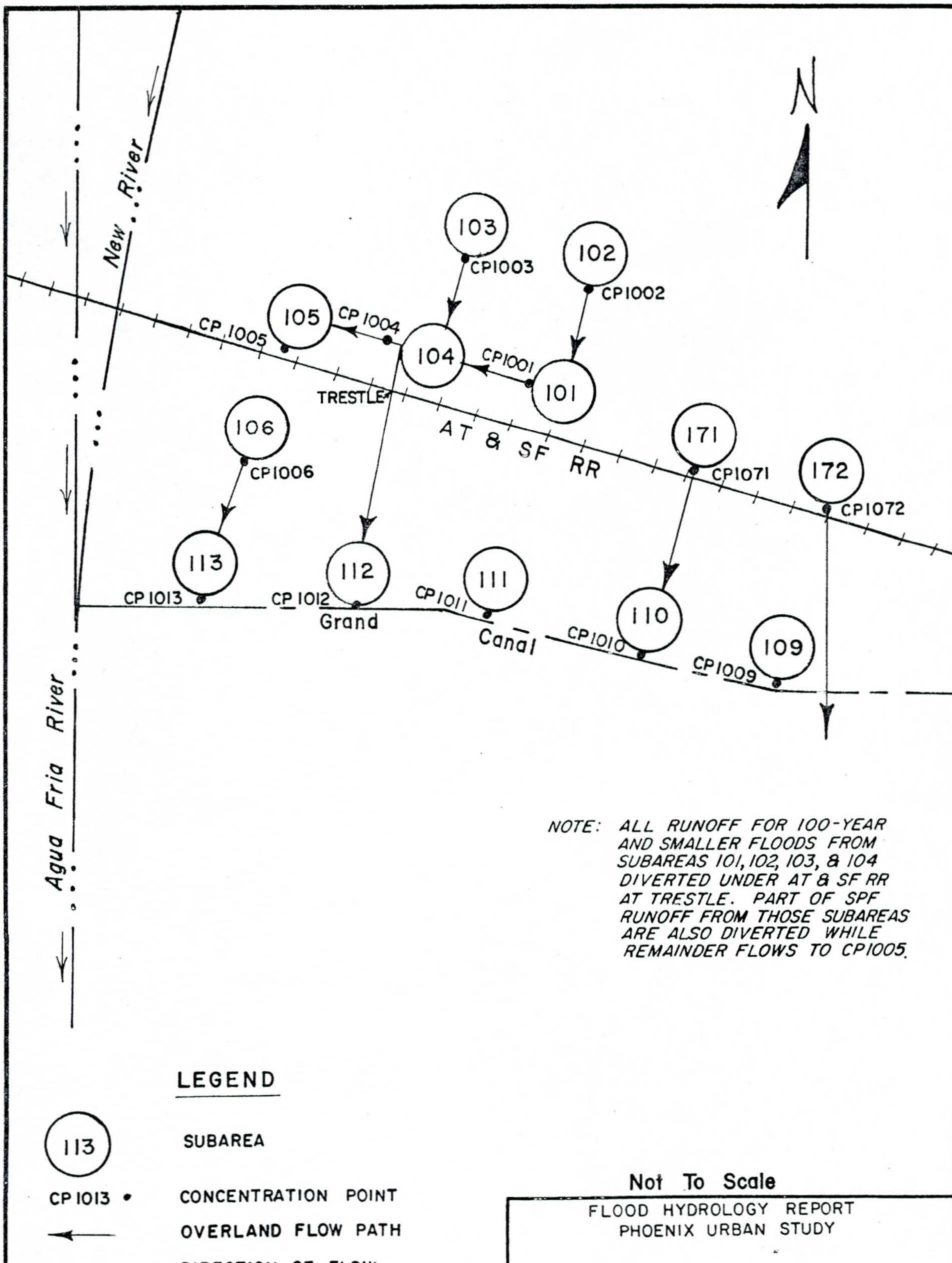
* Sheet-flow front

+ All runoff for 100-year and smaller floods from subareas 101, 102, 103, and 104 (7.42 sq. mi.) diverted under AT&SF RR at trestle. Part of SPF runoff from those subareas are also diverted while remainder flows to CP 1005.



112° 20'
112° 15'
112° 10'
33° 35'
33° 30'
N
AT & SF RR
Arizona Canal Diversion Channel
Agua Fria River
New River
Grand Canal
GLENDALE
MARYVALE
CP1001
CP1002
CP1003
CP1004
CP1005
CP1006
CP1009
CP1010
CP1011
CP1012
CP1013
CP1071
LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
105
SUBAREA DESIGNATION
CP1005
CONCENTRATION POINT AND DESIGNATED NUMBER
SHEETFLOW FRONT
SCALE IN MILES
0
1
2
3
4
FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY
SUBAREA DELINEATION
GLENDALE- MARYVALE
PRESENT AND FUTURE CONDITIONS
WITHOUT PROJECT
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

FIGURE A-1



FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY

SCHEMATIC FLOW DIAGRAM
GLENDALE-MARYVALE
PRESENT CONDITIONS
WITHOUT PROJECT

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

FIGURE A-2

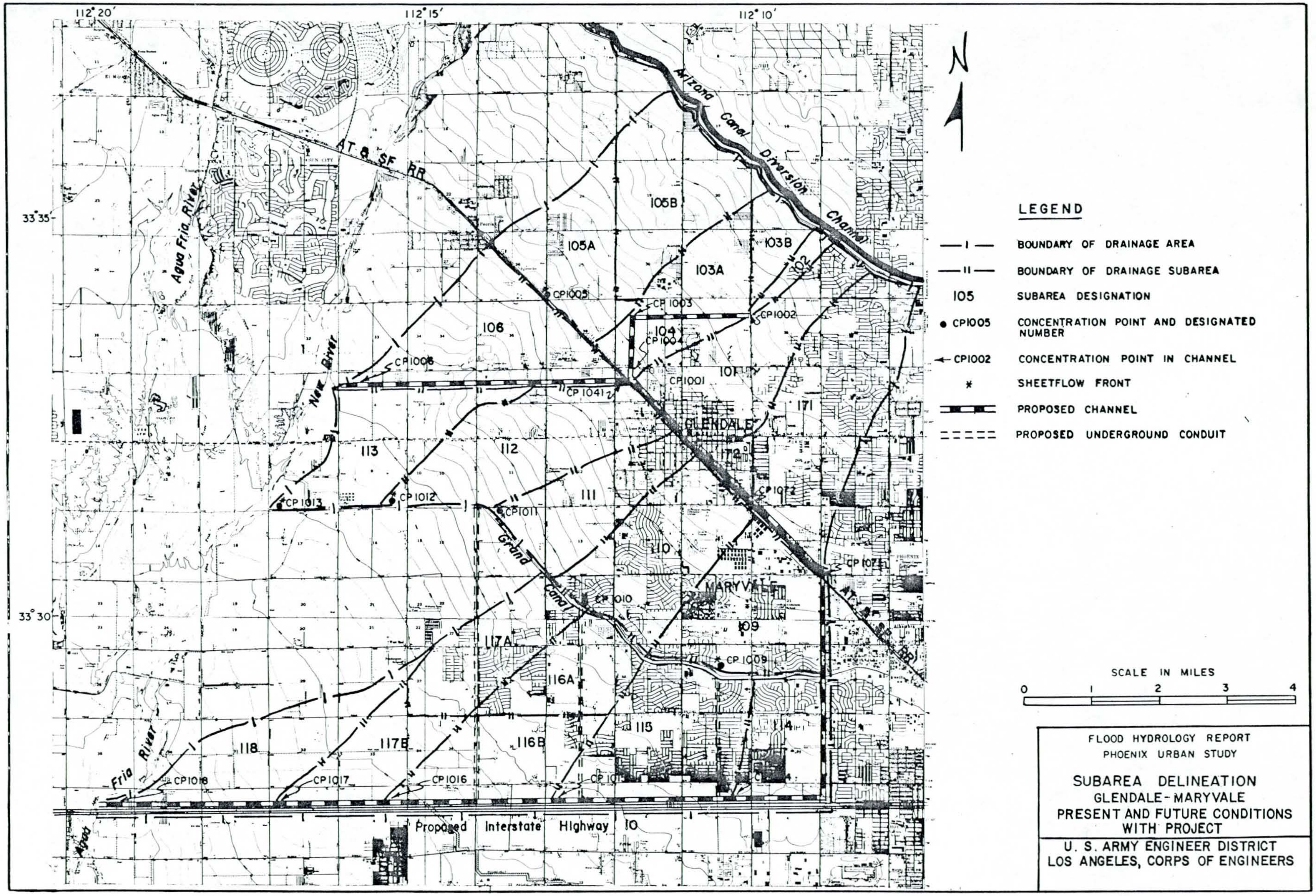
TABLE 1-B

Peak Discharges & Total Storm Runoff Volumes

Glendale-Maryvale, Present Conditions With Project

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
1001	In north unit channel at NE side of AT&SF RR immediately above the channel confl.	3.70	6,300 (950)	3,200 (450)	2,500 (360)	1,800 (275)
1002	In upper end of north unit channel at 15th Ave.	0.36	710 (85)	280 (30)	180 (25)	100 (13)
1003	In north unit channel SE of Olive Ave. & 67th Ave. intersection	2.85	3,800 (570)	960 (175)	480 (95)	230 (55)
1004	In north unit channel at NE side of AT&SF RR immediately above the channel confl.	3.70	5,300	1,700	1,100	680
1041	In north unit channel immediately below the channel confluence near AT&SF RR	6.90	10,900 (1,650)	4,400 (655)	3,400 (470)	2,400 (370)
1005	NE side of AT&SF RR near 75th Ave. & Olive Ave. intersection	6.75	6,700 (1,150)	820 (185)	240 (80)	50 (15)
1006	In lower end of north unit channel immediately above confluence with New River	11.71	12,200 (2,230)	3,900 (705)	2,700 (500)	2,000 (375)
1071	**			**		
1072	**			**		
1009	**			**		
1010	**			**		
1011	**			**		
1012*	Above Grand Canal between 91st Ave. & 83rd Ave.	4.40	4,800 (825)	800 (190)	300 (75)	100 (35)
1013*	Above Grand Canal between 107th Ave. & 99th Ave.	2.90	2,900 (510)	350 (85)	90 (30)	20 (5)

Values in parentheses, (950), indicates total storm runoff volume in acre-feet.
* Sheet-flow front
** Same as present without project, table 1-A



112° 20′
112° 15′
112° 10′
33° 35′
33° 30′
N
AT & SF RR
Agua Fria River
New River
Arizona Canal Diversion Channel
Grand Canal
GLENDALE
MARYVALE
Fria River
Proposed Interstate Highway 10
105B
105A
103A
103B
102
106
104
101
171
113
112
111
110
109
117A
116A
115
114
118
117B
116B
CP1005
CP1003
CP1002
CP1004
CP1006
CP1001
CP1041
CP1072
CP1013
CP1012
CP1011
CP1071
CP1010
CP1009
CP1018
CP1017
CP1016
LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
105 SUBAREA DESIGNATION
CP1005 CONCENTRATION POINT AND DESIGNATED NUMBER
CP1002 CONCENTRATION POINT IN CHANNEL
SHEETFLOW FRONT
PROPOSED CHANNEL
PROPOSED UNDERGROUND CONDUIT
SCALE IN MILES
0 1 2 3 4
FLOOD HYDROLOGY REPORT
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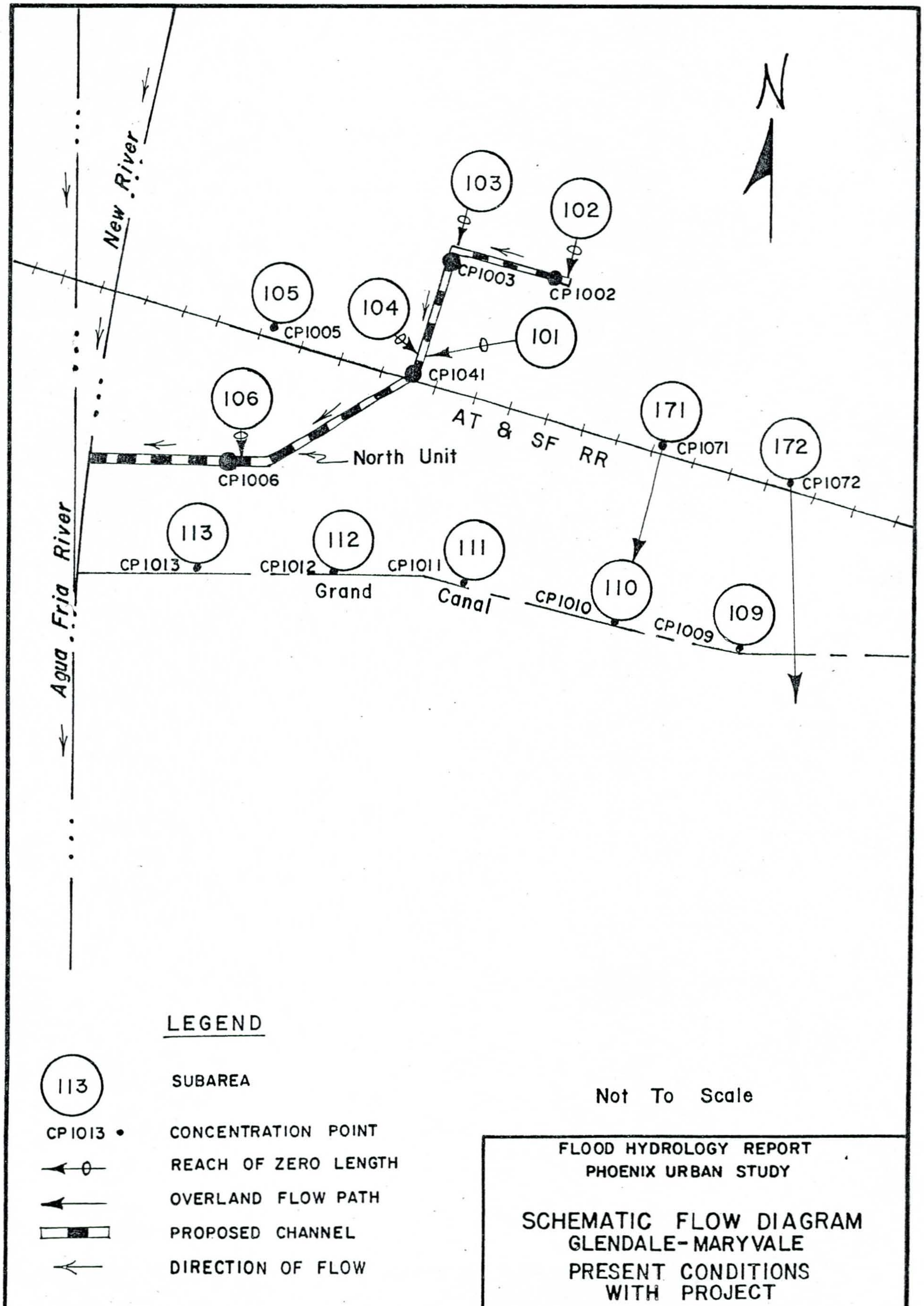
SUBAREA DELINEATION
GLENDALE-MARYVALE
PRESENT AND FUTURE CONDITIONS
WITH PROJECT
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

FIGURE A-3



FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY

SCHEMATIC FLOW DIAGRAM
GLENDALE-MARYVALE
PRESENT CONDITIONS
WITH PROJECT

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

FIGURE A-4

TABLE 1-C

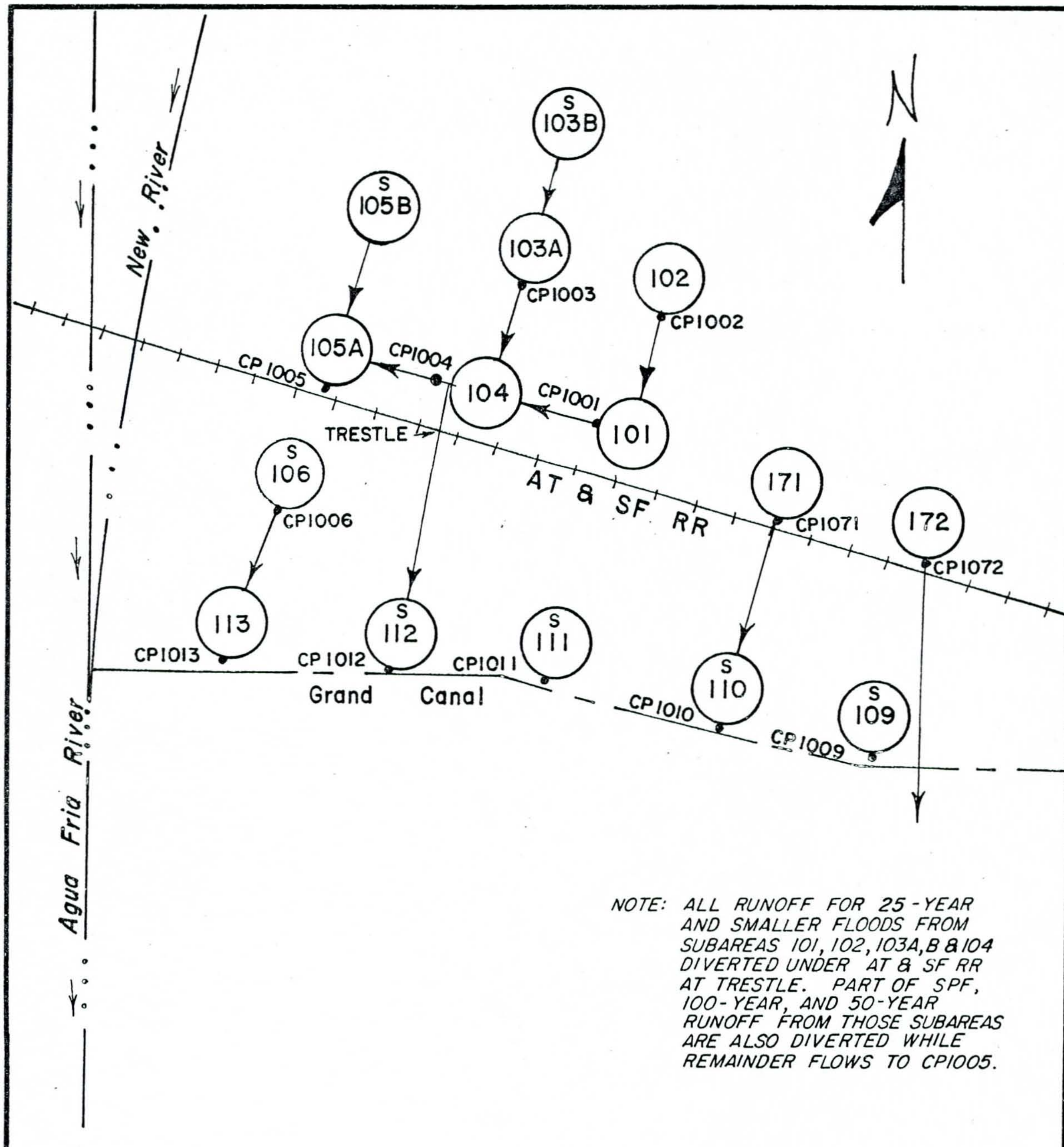
Peak Discharges & Total Storm Runoff Volumes

Glendale-Maryvale, Future Conditions Without Project

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
1001*	**	4.07	6,300 (1,035)	3,200 (495)	2,600 (410)	2,000 (335)
1002*	**	0.36	800 (95)	440 (45)	360 (35)	290 (30)
1003*	**	2.50	3,300 (585)	1,800 (255)	1,400 (210)	1,200 (170)
1004*+	**	7.42	4,500 (640)	690 (50)	15 (1)	0 (0)
1005*+	**	13.97	6,600 (1,965)	3,500 (600)	2,800 (480)	2,200 (390)
1006*	**	4.29	5,900 (1,010)	3,000 (475)	2,400 (400)	1,900 (340)
1071*	**	0.5	1,300 (135)	750 (65)	600 (55)	480 (45)
1072*	**	5.40	7,700 (1,350)	3,900	3,000	2,400
1009*	**	3.85	7,900 (1,080)	3,900 (470)	3,200 (395)	2,600 (335)
1010*	**	4.53	7,500 (1,175)	4,000 (565)	3,200 (460)	2,600 (390)
1011*	**	2.61	4,000 (610)	1,600 (265)	970 (170)	540 (110)
1012*	**	11.60	4,600 (1,885)	2,400 (905)	2,000 (720)	1,600 (555)
1013*	**	7.17	4,100 (1,415)	1,800 (515)	1,400 (400)	1,200 (330)

Values in parentheses, (1035), indicates total storm runoff volume in acre-feet.

* Sheet-flow front
** Same as present without project, table 1-A
\+ All runoff for 25-year and smaller floods from subareas 101, 102, 103A, 103B, and 104 (7.42 sq. mi.) diverted under AT&SF RR at trestle, Part of SPF, 100-year, and 50-year runoff from those subareas are also diverted while remainder flows to CP 1005.



LEGEND

113	SUBAREA
CP1013 ∘	CONCENTRATION POINT
⟵	OVERLAND FLOW PATH
←	DIRECTION OF FLOW
S	ON-SITE STORAGE ACCOUNTED

Not To Scale

FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY

SCHEMATIC FLOW DIAGRAM
GLENDALE-MARYVALE
FUTURE CONDITIONS
WITHOUT PROJECT

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

FIGURE A-5

TABLE 1-D

Peak Discharges & Total Storm Runoff Volumes

Glendale-Maryvale, Future Conditions With Project

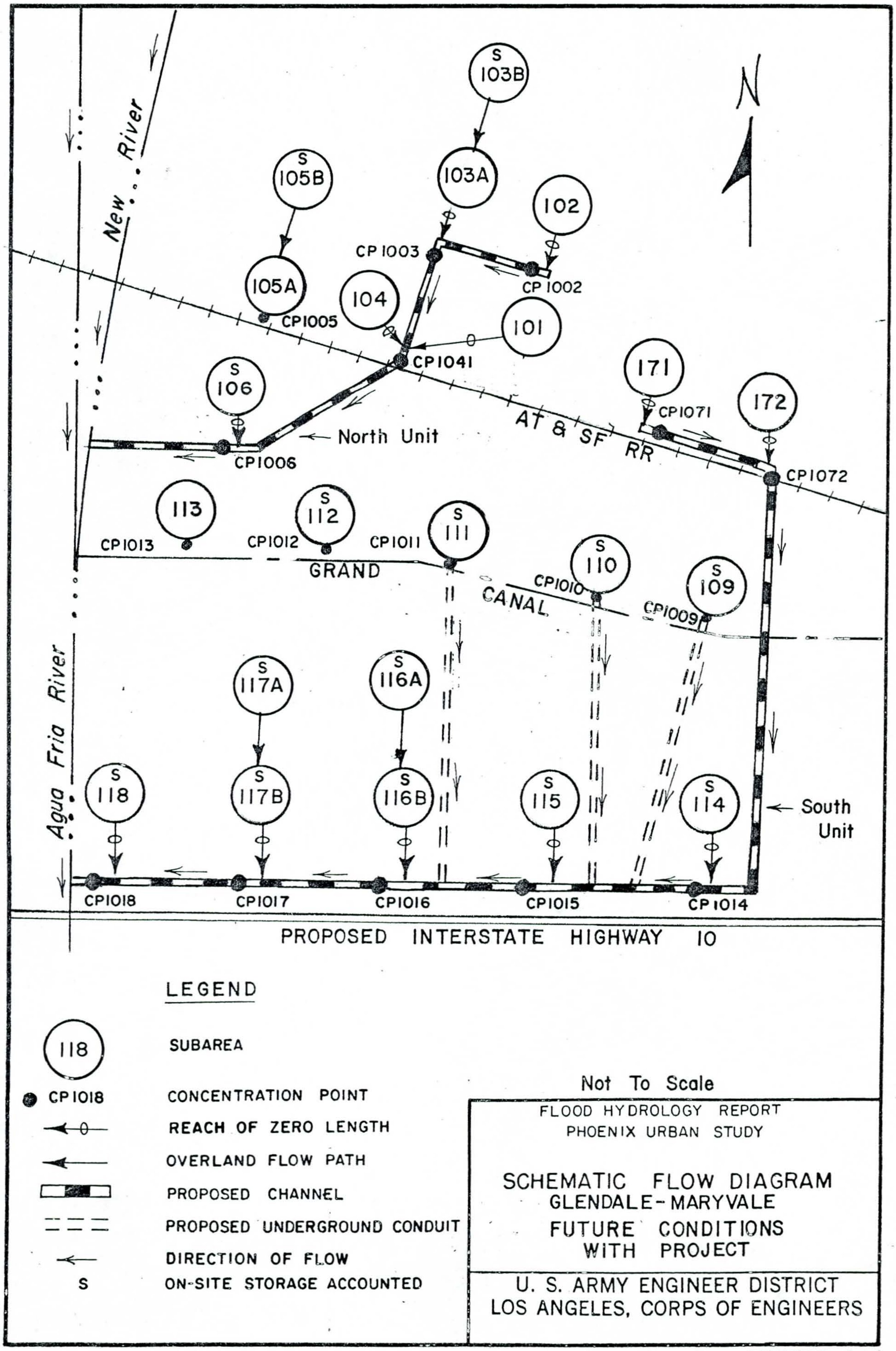
Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
1001	***	3.79	6,300 (950)	3,200 (455)	2,600 (380)	2,000 (305)
1002	***	0.36	800 (95)	440 (45)	360 (35)	290 (30)
1003	***	2.85	4,000 (675)	2,100 (295)	1,700 (245)	1,400 (200)
1004	***	3.70	5,800 (885)	3,000 (395)	2,400 (325)	1,900 (260)
1041	***	6.90	11,300 (1,755)	5,700 (805)	4,600 (670)	3,600 (565)
1005	***	6.75	7,100 (1,475)	3,800 (605)	3,100 (505)	2,500 (415)
1006	***	11.71	15,700 (3,190)	7,700 (1,265)	6,200 (1,050)	5,100 (890)
1071	In south Unit channel at junction of AT&SF RR and Bethany Rd.	0.50	1,300 (135)	730 (65)	600 (55)	490 (45)
1072	In south Unit channel at junction of AT&SF RR and Camelback Rd.	6.35	8,400 (1,455)	4,100 (695)	3,300 (575)	2,600 (470)
1009*	**	3.85	7,900 (1,080)	3,900 (470)	3,200 (395)	2,600 (335)
1010*	**	4.03	7,500 (1,045)	4,000 (505)	3,200 (425)	2,700 (360)
1011*	**	2.61	4,000 (610)	1,600 (265)	970 (170)	540 (110)
1012*	**	4.40	5,000 (835)	900 (235)	400 (100)	160 (50)
1013*	**	2.90	2,900 (510)	350 (85)	90 (30)	20 (5)

TABLE 1-D (Continued)

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
1014	In south unit channel along the freeway west of 51st Ave.	11.90	8,800 (1,860)	4,200 (860)	3,300 (710)	2,600 (570)
1015	In south unit channel along the freeway near 75th Ave.	20.15	19,500 (4,500)	10,000 (2,075)	8,000 (1,705)	6,300 (1,385)
1016	In south unit channel along the freeway west of 91st Ave.	27.65	23,300 (5,775)	10,500 (2,525)	8,300 (2,055)	6,400 (1,625)
1017	In south unit channel along the freeway near 107th Ave.	32.30	24,300 (6,525)	10,500 (2,810)	8,300 (2,285)	6,500 (1,830)
1018	In lower end of south unit channel, along the freeway, immediately above the conluence with Agua Fria River	35.10	24,900 (6,965)	10,200 (2,935)	8,000 (2,385)	6,300 (1,875)

Values in parentheses, (950), indicates total storm runoff volume in acre-feet.

* Sheet-flow front
** Same as present without project, table 1-A
*** Same as present with project, table 1-B


N
New River
Agua Fria River
S 103B
103A
102
S 105B
105A
104
101
CP 1003
CP 1002
CP1005
CP1041
S 106
North Unit
CP1006
171
CP1071
172
CP1072
AT & SF RR
113
S 112
S 111
CP1013
CP1012
CP1011
GRAND
CANAL
S 110
CP1010
S 109
CP1009
S 117A
S 116A
S 118
S 117B
S 116B
S 115
S 114
South Unit
CP1018
CP1017
CP1016
CP1015
CP1014
PROPOSED INTERSTATE HIGHWAY 10
LEGEND
118 SUBAREA
CP1018 CONCENTRATION POINT
REACH OF ZERO LENGTH
OVERLAND FLOW PATH
PROPOSED CHANNEL
PROPOSED UNDERGROUND CONDUIT
DIRECTION OF FLOW
S ON-SITE STORAGE ACCOUNTED
Not To Scale
FLOOD HYDROLOGY REPORT
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SCHEMATIC FLOW DIAGRAM
GLENDALE-MARYVALE
FUTURE CONDITIONS
WITH PROJECT
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

FIGURE A-6

TABLE 2-A

Peak Discharges & Total Storm Runoff Volumes

South Phoenix - Present Without Project

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
2001	Central Ave. near South Mtn. Speedway in South Mtn.	2.45	5,600 (630)	2,200 (250)	1,400 (160)	880 (100)
2002	West of 7th St. in South Mtn.	0.85	2,500 (220)	990 (90)	630 (60)	400 (30)
2003	East of 7th St. in South Mtn.	0.76	2,300 (200)	890 (80)	570 (50)	350 (30)
2004	Between 7th St. & 16th St. in South Mtn.	0.27	810 (70)	320 (30)	200 (18)	130 (12)
2005	Upper end of 16th St. in South Mtn.	0.28	1,000 (70)	400 (30)	260 (18)	160 (12)
2006	West of 16th St. & Dobbins Rd. intersection in South Mtn.	0.65	1,900 (170)	760 (70)	490 (40)	300 (30)
2008	South of 24th St. & Dobbins Rd. intersection in South Mtn.	0.50	1,600 (130)	640 (50)	410 (30)	260 (20)
2009	Between 24th St. & 32nd St. in South Mtn.	0.15	560 (40)	220 (15)	140 (10)	90 (6)
2010	Above North Branch Highline Canal, between 24th & 32nd St.	0.18	640 (50)	250 (18)	160 (12)	100 (7)
2011	Above North Branch Highline Canal near 32nd St.	0.18	670 (50)	260 (18)	170 (12)	100 (7)
2012	Above North Branch Highline Canal east of 32nd St.	0.12	460 (30)	180 (12)	110 (8)	70 (5)
2013	Above North Branch Highline Canal between 32nd & 40th St.	0.18	690 (50)	270 (18)	170 (12)	110 (7)
2014	Above North Branch Highline Canal west of 40th St.	0.14	540	210 (40)	50 (14)	30 (9)
2015	Above North Branch Highline Canal east of 40th St.	0.38	1,100 (100)	440 (40)	290 (20)	180 (15)

TABLE 2-A

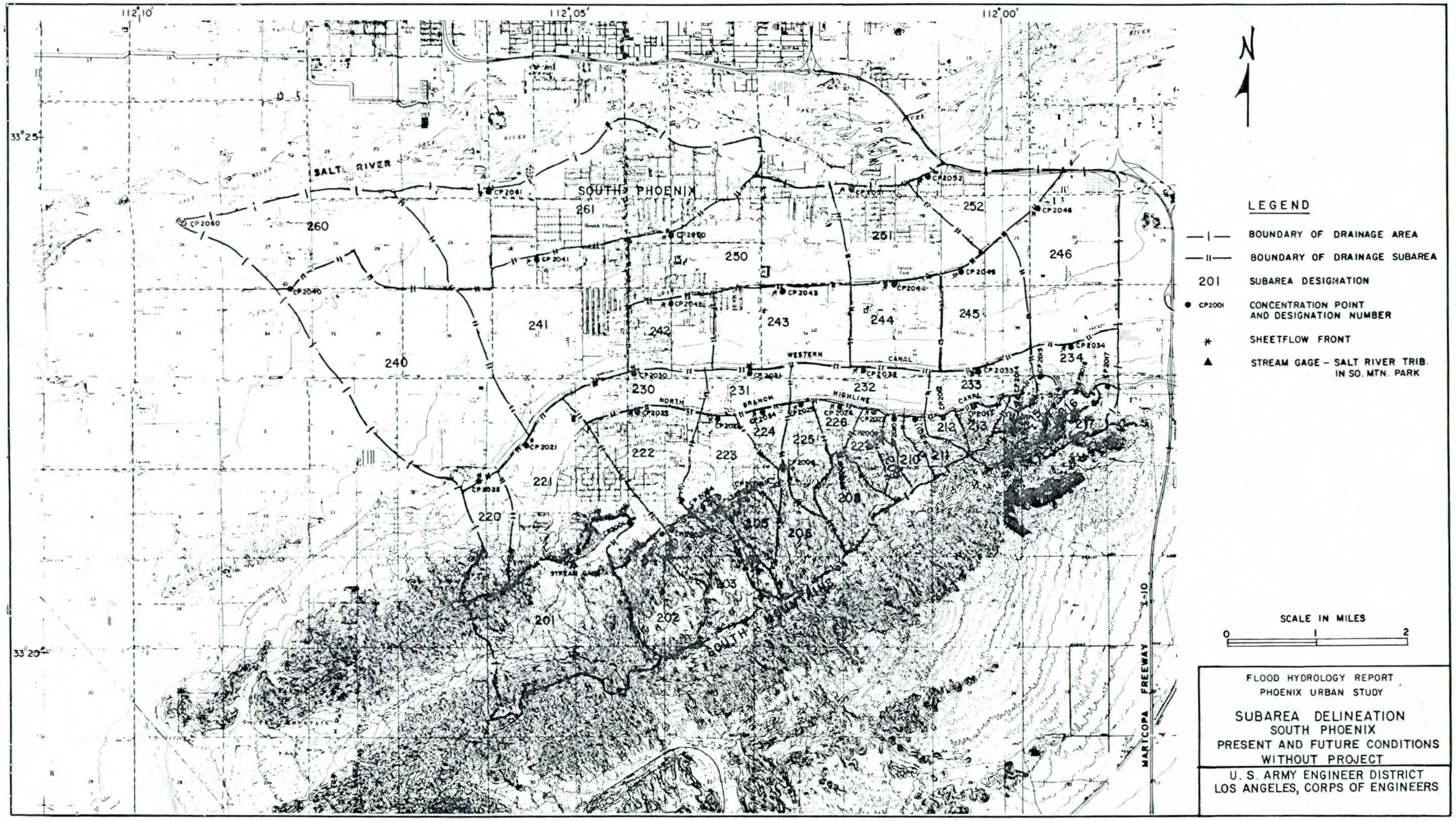
Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
2016	Above No Branch Highline Canal between 40th & 48th St.	0.14	530 (40)	210 (14)	130 (9)	80 (6)
2017	Above No Branch Highline Canal near 489th St.	0.30	900 (80)	350 (30)	230 (20)	140 (12)
2020*	Above Western Canal near 19th Ave.	0.57	1,800 (140)	1,100 (70)	950 (60)	750 (50)
2021*	Above Western Canal between 19th Ave. & 15th Ave.	5.22	5,300 (1300)	2,900 (550)	2,300 (390)	1,900 (280)
2022*	Above No Branch Highline Canal near Central Ave.	1.82	3,000 (480)	1,800 (220)	1,500 (170)	1,200 (130)
2023*	Above No Branch Highline Canal between 7th St. & 16th St.	1.34	3,200 (350)	1,900 (160)	1,600 (120)	1,200 (90)
2024*	Above No Branch Highline Canal near 16th St.	0.30	810 (80)	500 (40)	420 (35)	340 (30)
2025*	Above No Branch Highline Canal between 16th St. & 24th St.	1.09	2,200 (280)	1,200 (120)	950 (90-)	750 (70)
2026*	Above No Branch Highline Canal near 24th St.	0.70	1,600 (190)	690 (80)	510 (60)	370 (40)
2027*	Above No Branch Highline Canal between 24th St. & 32nd St.	0.43	1,200 (110)	740 (50)	610 (40)	490 (30)
2030*	Above Western Canal near Central Ave.	2.49	3,600 (640)	1,700 (280)	1,400 (210)	1,100 (160)
2031*	Above Western Canal near 16th St.	2.13	3,900 (550)	1,900 (240)	1,500 (180)	1,200 (140)
2032*	Above Western Canal near 24th St.	3.33	6,100 (810)	2,500 (320)	1,900 (230)	1,300 (160)
2033*	Above Western Canal near 36th St.	0.94	2,300 (220	780 (80)	510 (50)	310 (30)

TABLE 2-A (Continued)

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
2034*	Above Western Canal between 40th St. & 48th St.	1.33	3,100 (330)	1,300 (130)	870 (90)	550 (60)
2040*	West of Southern Ave. & 35th Ave. intersection	9.63	4,900 (2,100)	2,100 (770)	1,600 (530)	1,200 (360)
2041*	North of Southern Ave. & 15th Ave. intersection	4.07	6,200 (1,100)	3,400 (480)	2,800 (390)	2,200 (310)
2042*	South of Southern Ave. & 7th St. intersection	1.79	2,700 (460)	1,200 (190)	1,900 (150)	1,500 (110)
2043*	West of Southern Ave. & 16th St. intersection	5.11	5,800 (1,200)	2,500 (510)	1,900 (380)	1,500 (290)
2044*	Southern Ave. between 24th St. & 32nd St.	2.62	2,600 (610)	1,100 (220)	780 (160)	550 (100)
2045*	NE of Southern Ave. & 32nd St. intersection	2.05	2,300 (460)	690 (150)	450 (100)	280 (60)
2046*	South of Broadway & 40th St. intersection	3.20	4,600 (840)	2,500 (360)	2,000 (280)	1,400 (210)
2050*	North of Roeser Rd. & 7th St. intersection	10.89	6,600 (2,200)	3,600 (920)	2,900 (710)	2,400 (540)
2051*	North of Broadway & 24th St. intersection	5.91	4,300 (1,300)	1,500 (490)	1,000 (350)	710 (240)
2052*	NW of Broadway & 32nd St. intersection	4.05	3,500 (1,000)	1,700 (430)	1,400 (330)	1,000 (240)
2060*	Salt River between 52st Ave. & 43rd Ave.	11.65	4,400 (2,500)	2,000 (910)	1,500 (640)	1,100 (430)
2061*	North of Broadway & 19th Ave. intersection	16.93	7,100 (3,900)	3,300 (1,800)	2,700 (1,300)	2,200 (1,100)

Values in parentheses, (330), indicates total storm runoff volume in acre-feet.

* Sheet-flow front


LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
201 SUBAREA DESIGNATION
CP2001 CONCENTRATION POINT AND DESIGNATION NUMBER
SHEETFLOW FRONT
STREAM GAGE - SALT RIVER TRIB. IN SO. MTN. PARK
SCALE IN MILES
0 1 2
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SOUTH PHOENIX
PRESENT AND FUTURE CONDITIONS
WITHOUT PROJECT
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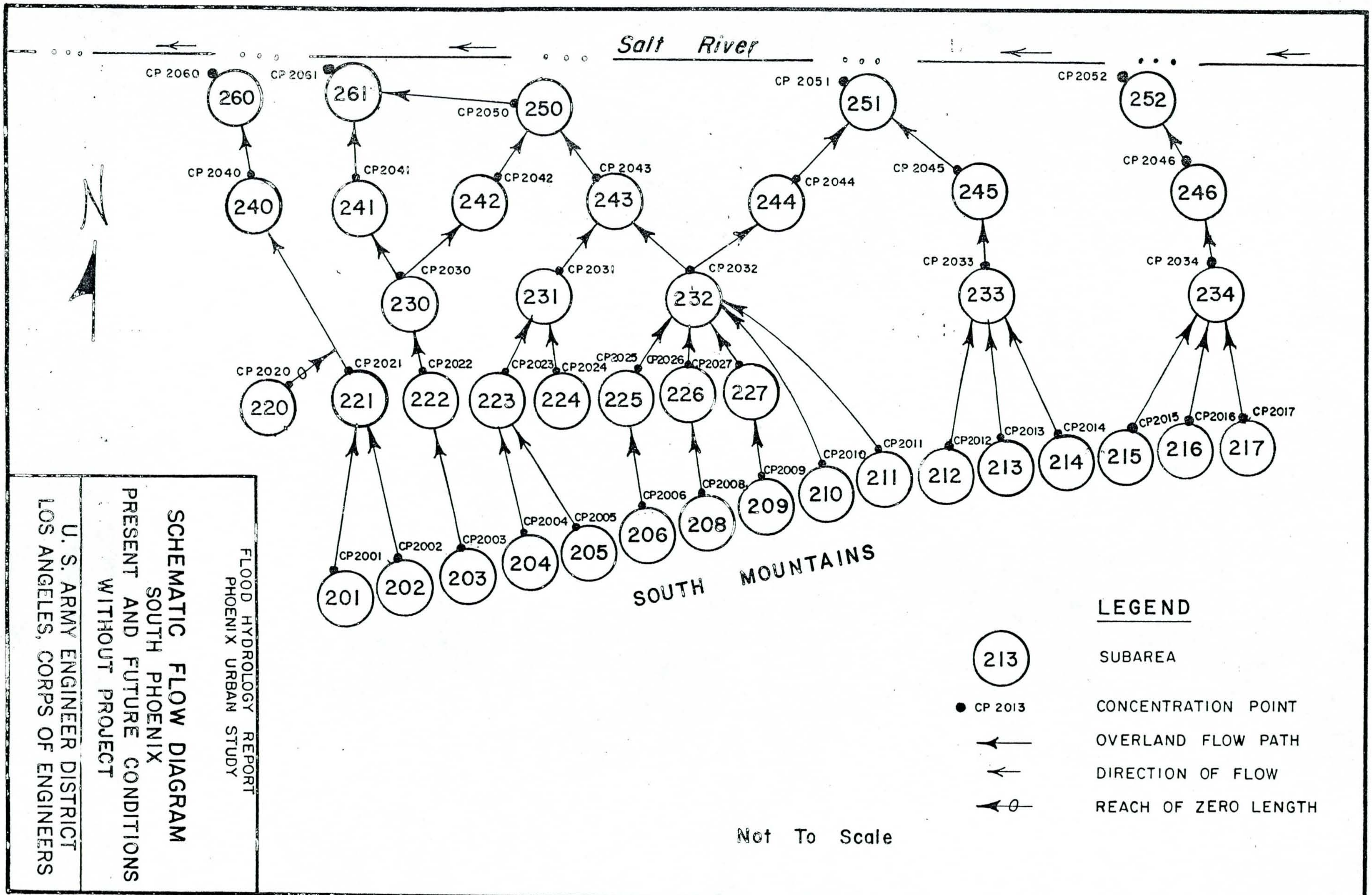
LOS ANGELES, CORPS OF ENGINEERS
SALT RIVER
SOUTH PHOENIX
WESTERN CANAL
HIGHLINE CANAL
MARICOPA FREEWAY
112°10'
112°05'
112°00'
33°25'
33°20'

FIGURE A-7



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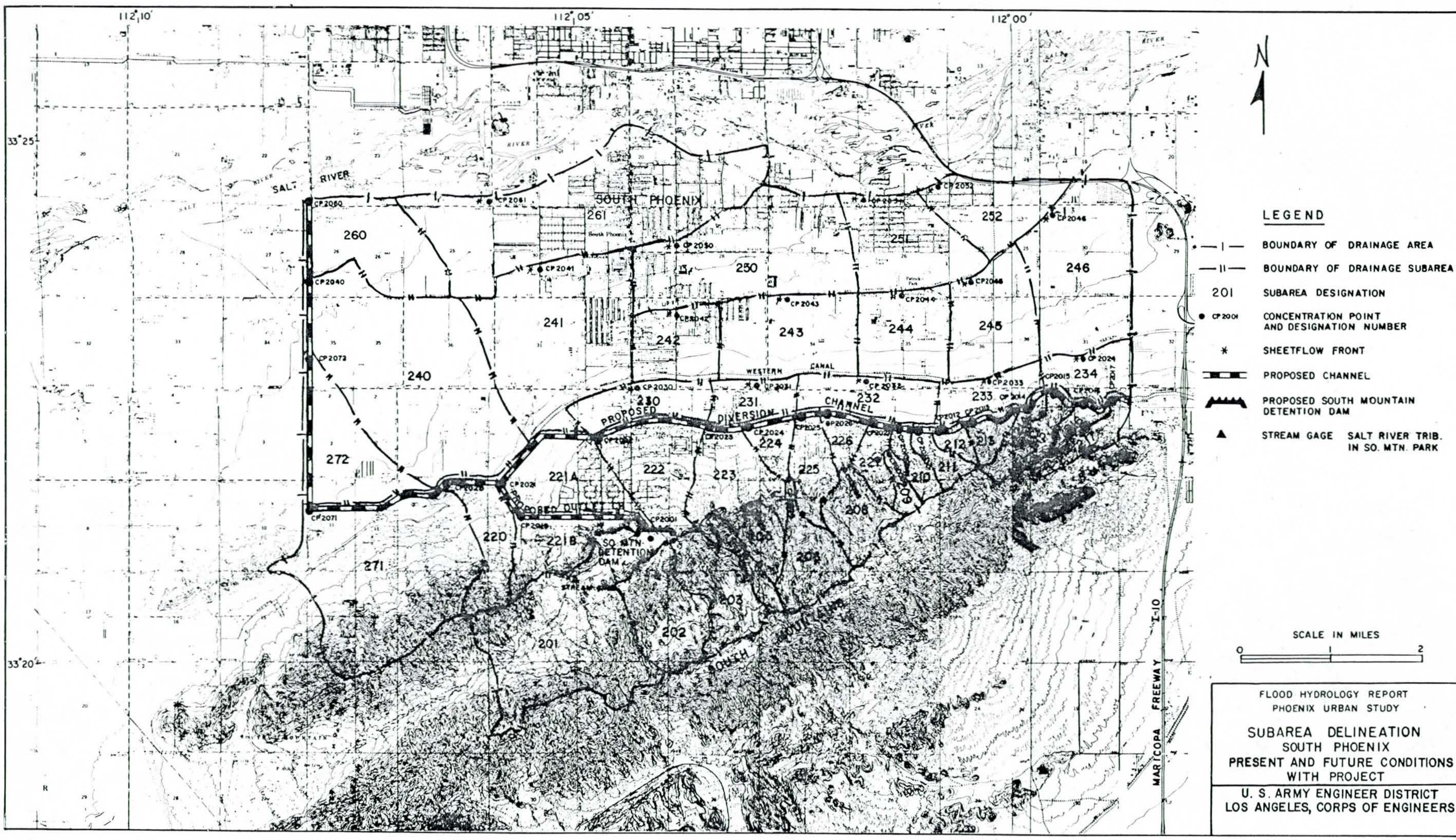
PHOENIX URBAN STUDY

SCHEMATIC FLOW DIAGRAM
SOUTH PHOENIX
PRESENT AND FUTURE CONDITIONS
WITHOUT PROJECT

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

FIGURE A-8



LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
201 SUBAREA DESIGNATION
CP2001 CONCENTRATION POINT AND DESIGNATION NUMBER
SHEETFLOW FRONT
PROPOSED CHANNEL
PROPOSED SOUTH MOUNTAIN DETENTION DAM
STREAM GAGE SALT RIVER TRIB. IN SO. MTN. PARK
SCALE IN MILES
0
1
2
N
SALT RIVER
SOUTH PHOENIX
PROPOSED DIVERSION CHANNEL
WESTERN CANAL
PROPOSED OUTLET
SO. MTN. DETENTION DAM
SOUTH MOUNTAINS
MARICOPA FREEWAY
I-10
112°10'
112°05'
112°00'
33°25'
33°20'
FLOOD HYDROLOGY REPORT
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SUBAREA DELINEATION
SOUTH PHOENIX
PRESENT AND FUTURE CONDITIONS
WITH PROJECT
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

FIGURE A-9

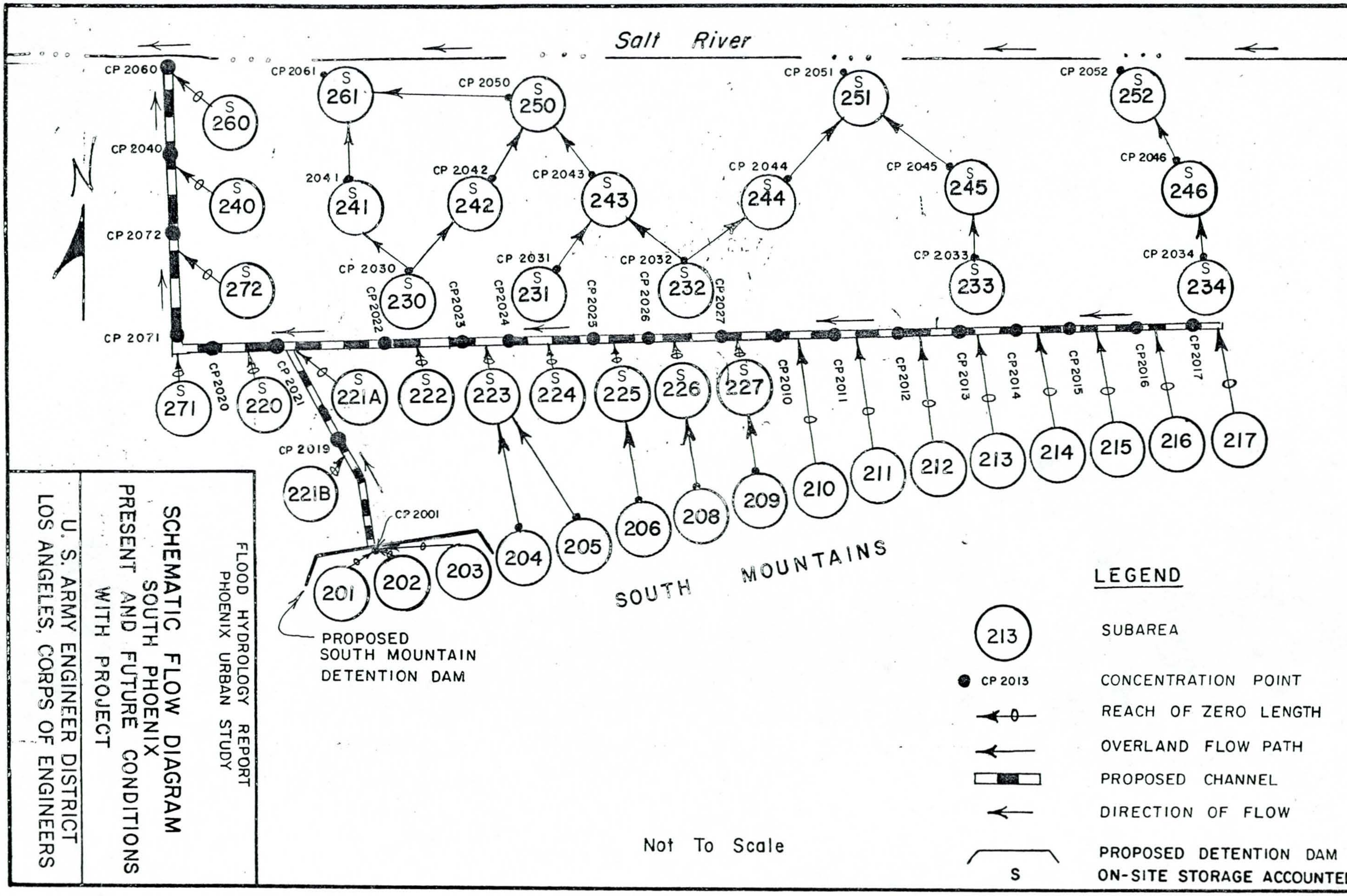
TABLE 2-B

Peak Discharges and Total Storm Runoff Volumes

South Phoenix, Present Conditions With Project

Concentration Point Number	Location	Storm Ctr.DA sq. mi.	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
2030	**	2.49	0.67	1,900	640 (160)	430 (60)	270 (40)
2031*	**	2.13	0.49	1,200 (120)	320 (40)	220 (30)	190 (25)
2032*	**	3.33	0.72	1,300	290	170	80
2033*	**	0.94	0.50	1,400 (110)	360 (30)	240 (20)	140 (11)
2034*	**	1.33	0.51	1,600 (120)	650 (50)	450 (40)	270 (25)
2041*	**	5.07	2.98	6,200 (780)	3,400 (350)	2,800 (290)	2,200 (240)
2042*	**	3.28	1.06	2,600 (270)	1,200 (110)	840 (80)	550 (60)
2043*	**	6.77	2.16	3,900 (510)	1,700 (190)	1,300 (140)	850 (110)
2044*	**	4.28	1.31	2,400 (280)	660 (80)	430 (60)	230 (30)
2045*	**	2.05	1.61	2,300 (350)	560 (100)	360 (70)	210 (40)
2046*	**	3.20	2.38	4,500 (630)	2,500 (280)	2,100 (230)	1,400 (180)
2050*	**	12.38	5.54	6,300 (1,300)	3,600 (540)	2,900 (420)	2,400 (330)
2051*	**	7.57	4.16	3,100 (920)	1,300 (310)	940 (220)	670 (150)
2052*	**	4.05	3.23	3,400 (803)	1,600 (350)	1,300 (280)	990 (210)
2061*	**	18.6	16.93	6,800 (2,900)	3,200 (1,300)	2,600 (1,100)	2,100 (830)

Values in parentheses, (160), indicates total storm runoff volumes in acre-feet
* Sheet-flow front
** Same as present without project, table 2-A.



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SCHEMATIC FLOW DIAGRAM
SOUTH PHOENIX
PRESENT AND FUTURE CONDITIONS
WITH PROJECT

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

FIGURE A-10

TABLE 2-C

Peak Discharges and Total Storm Runoff Volumes

South Phoenix, Future Conditions Without Project

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
2001 thru 2017	**					
2020*	**	0.57	1,900 (150)	1,100 (70)	890 (60)	660 (50)
2021*	**	5.22	5,300 (1,300)	2,700 (540)	2,300 (400)	1,700 (280)
2022*	**	1.82	3,100 (490)	1,900 (220)	1,600 (170)	1,200 (130)
2023*	**	1.34	3,200 (360)	1,900 (160)	1,500 (120)	1,100 (90)
2024*	**	0.30	840 (80)	500 (40)	420 (35)	340 (30)
2025*	**	1.09	2,200 (290)	1,200 (120)	940 (90)	740 (70)
2026*	**	0.70	1,600 (190)	700 (80)	510 (60)	370 (40)
2027*	**	0.43	1,200 (110)	750 (50)	620 (40)	500 (30)
2030*	**	2.49	3,700 (670)	2,000 (310)	1,600 (240)	1,300 (190)
2031*	**	2.13	4,000 (580)	2,000 (260)	1,600 (200)	1,200 (150)
2032*	**	3.33	6,300 (860)	2,900 (370)	2,200 (260)	1,700 (200)
2033*	**	0.94	2,400 (250)	1,300 (110)	930 (80)	650 (60)
2034*	**	1.33	3,100 (350)	1,600 (150)	1,200 (110)	860 (80)

TABLE 2-C (Continued)

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
2040*	**	9.63	6,800 (2,400)	3,200 (1,000)	2,400 (780)	1,700 (580)
2041*	**	4.07	6,800 (1,100)	4,000 (530)	3,400 (450)	2,700 (360)
2042**	**	1.79	3,000 (490)	1,700 (230)	1,500 (190)	1,200 (150)
2043*	**	5.11	5,900 (1,300)	2,800 (590)	2,300 (460)	1,900 (360)
2044*	**	2.62	3,100 (680)	1,600 (300)	1,200 (230)	880 (170)
2045*	**	2.05	3,000 (550)	1,500 (240)	1,200 (190)	830 (140)
2046*	**	3.20	4,800 (850)	2,700 (390)	2,200 (310)	1,800 (245)
2050*	**	10.89	7,000 (2,300)	3,800 (1,100)	3,200 (850)	2,600 (680)
2051*	**	5.91	4,800 (1,500)	2,200 (680)	1,700 (540)	1,300 (410)
2052*	**	4.05	3,600 (1,100)	1,800 (520)	1,500 (420)	1,200 (340)
2060*	**	11.65	5,700 (2,800)	2,500 (1,200)	2,000 (960)	1,400 (720)
2061*	**	16.93	7,400 (4,100)	3,600 (1,900)	3,100 (1,600)	2,500 (1,300)

Values in parentheses, (2,400), indicates total storm runoff volume in acre-feet

* Sheet-flow front

** Same as present without project, table 2A

TABLE 2-D

Peak Discharges and Total Storm Runoff Volumes

South Phoenix, Future Conditions With Project

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
2017	In diversion channel west of 48th St.	.30	900 (80)	350 (30)	230 (20)	140 (12)
2016	In diversion channel between 40th St. & 48th St.	.44	1,200 (110)	470 (40)	300 (30)	190 (18)
2015	In diversion channel east near 40th St.	.82	2,100 (210)	820 (80)	530 (50)	330 (30)
2014	In diversion channel west of 40th St.	.96	2,400 (240)	950 (100)	610 (60)	380 (40)
2013	In diversion channel east of 32nd St.	1.14	2,500 (290)	1,000 (120)	640 (70)	400 (50)
2012	In diversion channel near 32nd St.	1.26	2,900 (360)	1,100 (130)	710 (80)	440 (50)
2011	In diversion channel between 28th St. & 32nd ST.	1.44	3,300 (360)	1,300 (140)	830 (90)	520 (60)
2010	In diversion channel near 28th St.	1.62	3,400 (410)	1,400 (160)	870 (110)	540 (70)
2027	In diversion channel east of 24th St.	2.05	4,400 (520)	1,800 (210)	1,200 (140)	860 (100)
2026	In diversion channel between 16th St. & 24th St.	2.78	5,900 (710)	2,400 (290)	1,700 (200)	1,200 (130)
2025	In diversion channel between 16th St.	3.87	7,700 (980)	3,200 (410)	2,300 (280)	1,700 (200)
2024	In diversion channel west of 16th St.	4.17	7,800 (1,100)	3,300 (450)	2,400 (320)	1,800 (230)
2023	In diversion channel between 7th St. & 16th St.	5.51	9,300 (1,400)	4,100 (590)	3,100 (420)	2,300 (310)
2022	In diversion channel east of 7th Ave.	6.57	10,000 (1,700)	4,700 (720)	3,700 (540)	2,800 (400)

TABLE 2-D (Continued)

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
2001	At South Mountain Detention Dam	4.25	11,000 (1,000)	3,900 (390)	2,500 (250)	1,600 (160)
2019	In outlet channel north of Phoenix Police Academy	4.88	2,200 (150)	1,400 (70)	1,200 (60)	960 (50)
2021	Below the confluence of diversion channel & outlet channel	12.55	10,000 (2,000)	4,900 (870)	4,000 (670)	3,100 (510)
2020	In diversion channel west of 19th Ave.	13.12	10.00 (2,100)	5,100 (930)	4,200 (720)	3,300 (550)
2071	In diversion channel south of Dobbins Rd. & 35th Ave. intersection	16.22	15,500 (2,800)	7,900 (1,200)	6,500 (960)	5,100 (740)
2072	In diversion channel north of Baseline Rd. & 35th Ave. intersection	17.29	16,500 (3,000)	8,300 (1,300)	6,800 (1,040)	5,200 (790)
2040	In diversion channel north of Southern Ave. & 35th Ave. intersection	21.13	21,000 (3,900)	10,500 (1,700)	8,400 (1,300)	6,300 (1,000)
2060	In diversion channel north of Broadway & 35th Ave.	22.29	22,500 (4,100)	11,000 (1,800)	8,800 (1,400)	6,600 (1,100)

Values in parentheses, (1,000), are total storm runoff volumes in acre-feet.

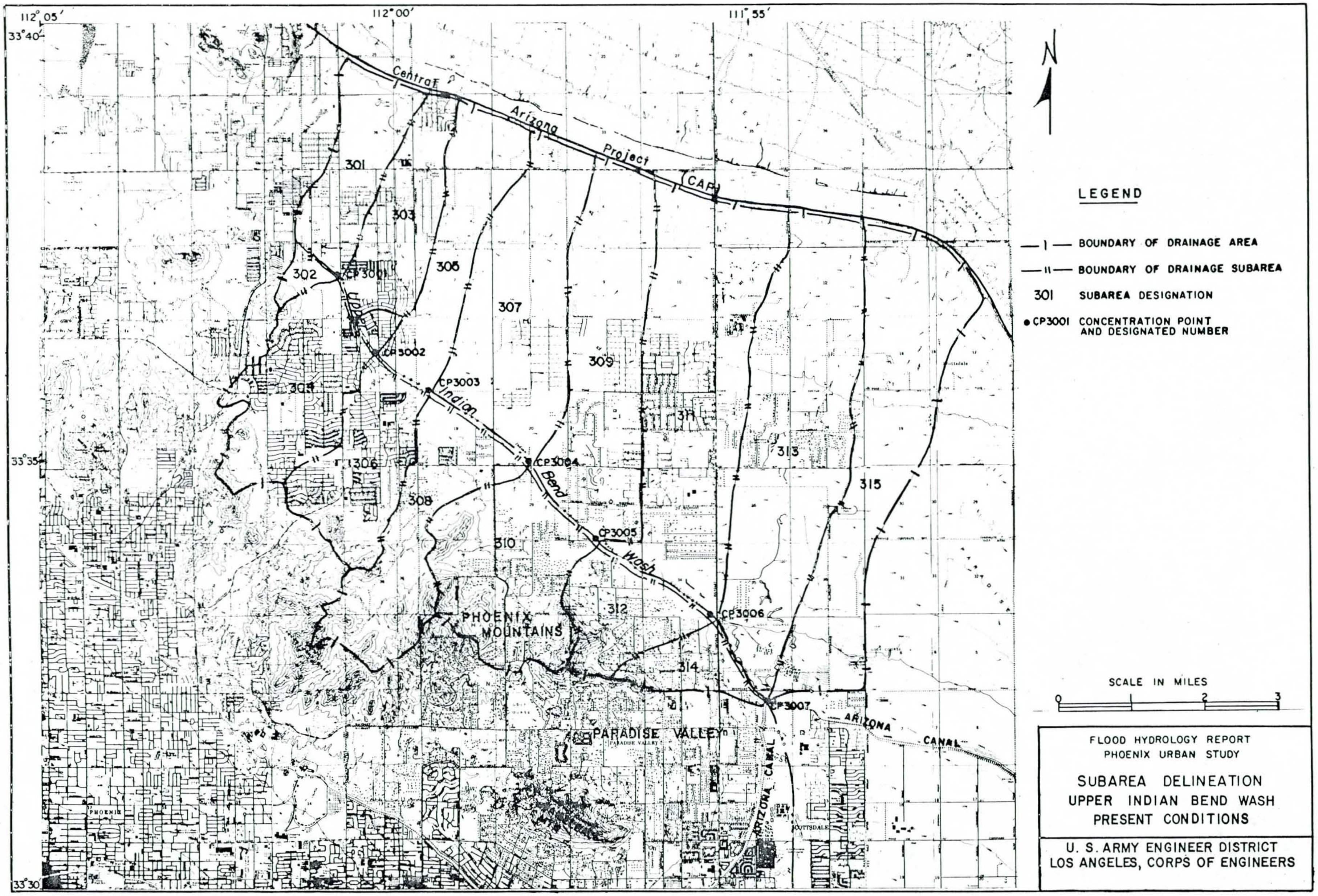
TABLE 3-A

Peak Discharges & Total Storm Runoff Volumes

Upper Indian Bend Wash, Present Conditions

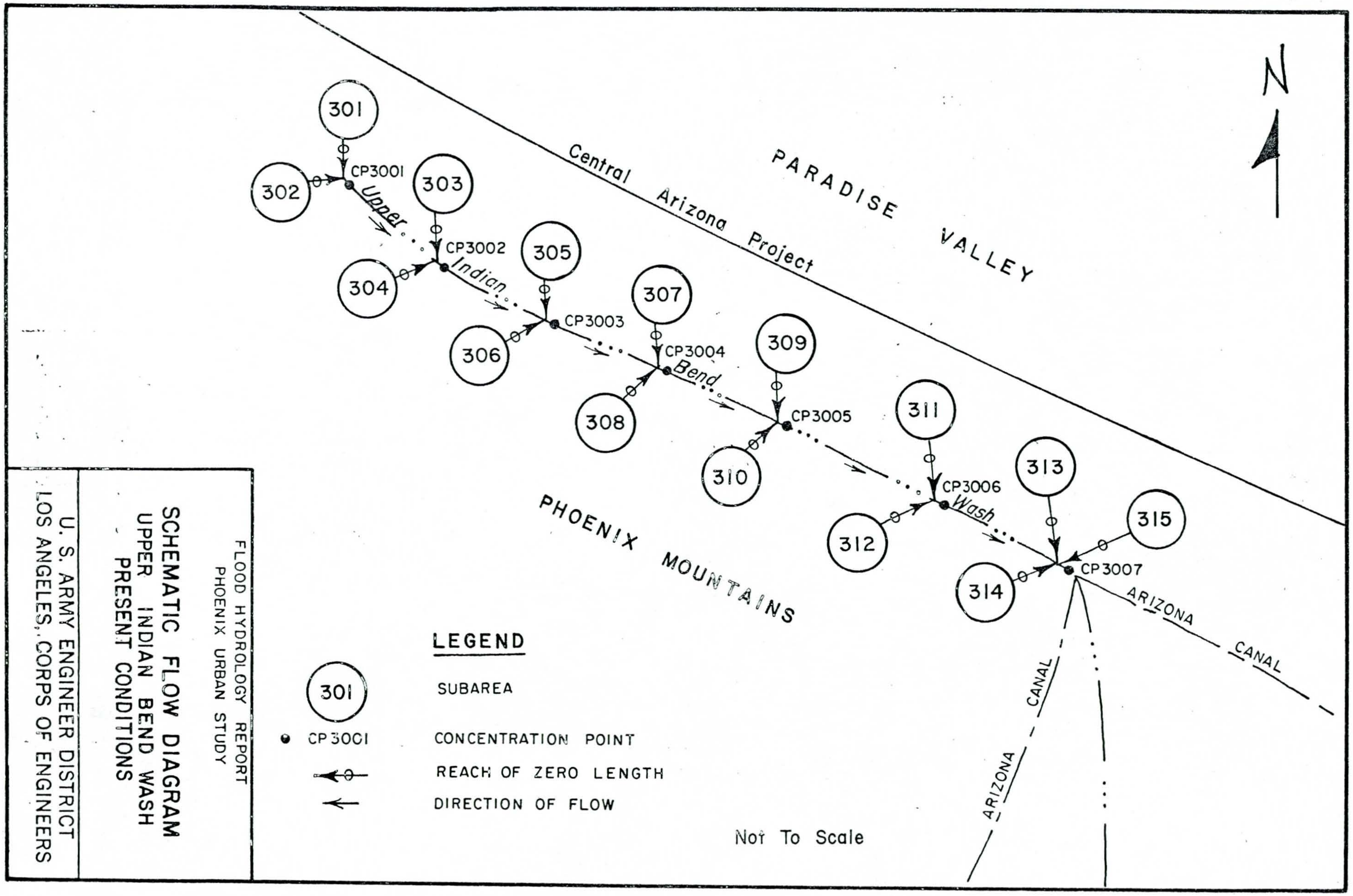
Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
3001	Indian Bend Wash at 32nd St.	2.77	5,500 (670)	2,400 (305)	1,400 (230)	1,200 (185)
3002	Indian Bend Wash at 36th St.	9.17	15,500 (2,090)	6,000 (940)	3,500 (735)	2,000 (585)
3003	Indian Bend Wash at Cactus Rd.	14.25	21,000 (3,200)	9,000 (1,425)	5,600 (1,090)	3,200 (855)
3004	Indian Bend Wash at Shea Blvd.	23.92	27,000 (4,800)	13,000 (2,060)	8,500 (1,545)	5,400 (1,185)
3005	Indian Bend Wash at Double Tree Branch Rd.	34.24	31,000 (6,455)	14,500 (2,735)	10,000 (2,070)	6,500 (1,545)
3006	Indian Bend Wash at Scottsdale Rd.	44.26	34,000 (7,995)	16,000 (3,375)	11,000 (2,585)	7,000 (1,050)
3007	Indian Bend Wash immediately above Arizona Canal	59.6	39,000 (10,165)	17,000 (3,095)	12,000 (2,345)	7,200

Values in parentheses, (670), are total storm runoff volumes in acre-feet.



112°05′
112°00′
111°55′
33°40′
33°35′
33°30′
N
Central Arizona Project (CAP)
Indian Bend Wash
301
302
303
304
305
306
307
308
309
310
311
312
313
314
315
CP3001
CP3002
CP3003
CP3004
CP3005
CP3006
CP3007
PHOENIX MOUNTAINS
PARADISE VALLEY
ARIZONA CANAL
LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
301 SUBAREA DESIGNATION
CP3001 CONCENTRATION POINT AND DESIGNATED NUMBER
SCALE IN MILES
0 1 2 3
FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY
SUBAREA DELINEATION
UPPER INDIAN BEND WASH
PRESENT CONDITIONS
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

FIGURE A-11



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PHOENIX URBAN STUDY

SCHEMATIC FLOW DIAGRAM
UPPER INDIAN BEND WASH
PRESENT CONDITIONS

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

FIGURE A-12

TABLE 3-B

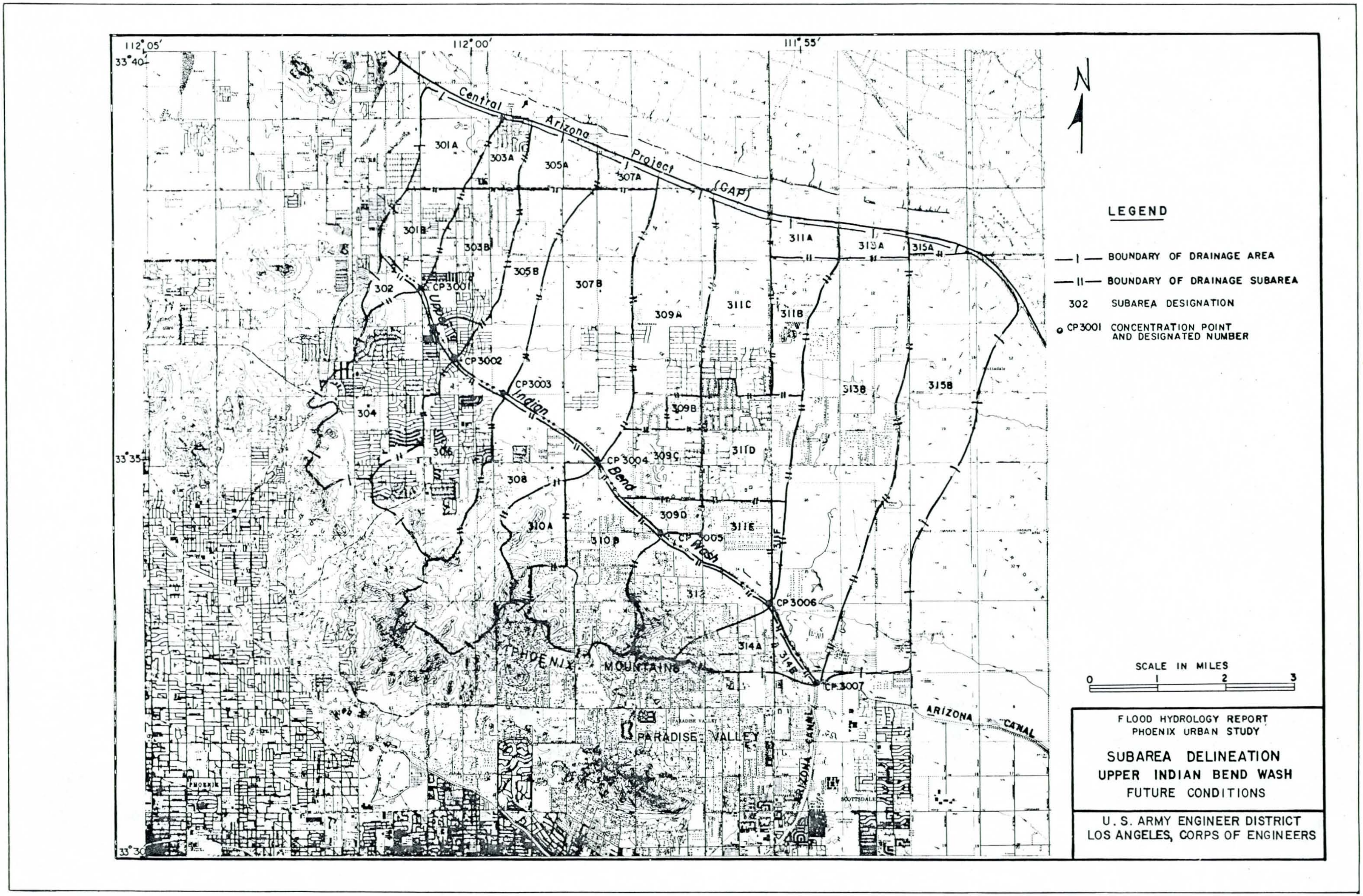
Peak Discharges & Total Storm Runoff Volumes

Upper Indian Bend Wash, Future Conditions

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
3001	*	2.77	5,500 (810)	2,400 (280)	1,400 (210)	1,200 (155)
3002	*	9.17	15,500 (2,480)	6,000 (730)	3,500 (580)	2,000 (440)
3003	*	14.25	21,000 (3,890)	9,000 (1,150)	5,600 (880)	3,200 (665)
3004	*	23.92	27,000 (6,120)	13,000 (1,920)	8,500 (1,420)	5,400 (1,075)
3005	*	34.24	31,000 (8,250)	14,500 (2,540)	10,000 (1,980)	6,500 (1,480)
3006	*	44.26	34,000 (10,000)	16,000 (3,095)	11,000 (2,325)	7,000 (1,785)
3007	*	59.6	39,000 (10,550)	17,000 (3,810)	12,000 (2,890)	7,200 (2,140)

Values in parentheses, (810), are total storm runoff volumes in acre-feet.

* Same as present conditions, table 5A


112°05′
112°00′
111°55′
33°40′
33°35′
33°30′
N
Central Arizona Project (CAP)
Upper Indian Bend Wash
PHOENIX MOUNTAINS
PARADISE VALLEY
ARIZONA CANAL
301A
301B
302
303A
303B
304
305A
305B
306
307A
307B
308
309A
309B
309C
309D
310A
310B
311A
311B
311C
311D
311E
311F
312
313A
313B
314A
314B
315A
315B
CP3001
CP3002
CP3003
CP3004
CP3005
CP3006
CP3007
LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
302 SUBAREA DESIGNATION
CP3001 CONCENTRATION POINT AND DESIGNATED NUMBER
SCALE IN MILES
0
1
2
3
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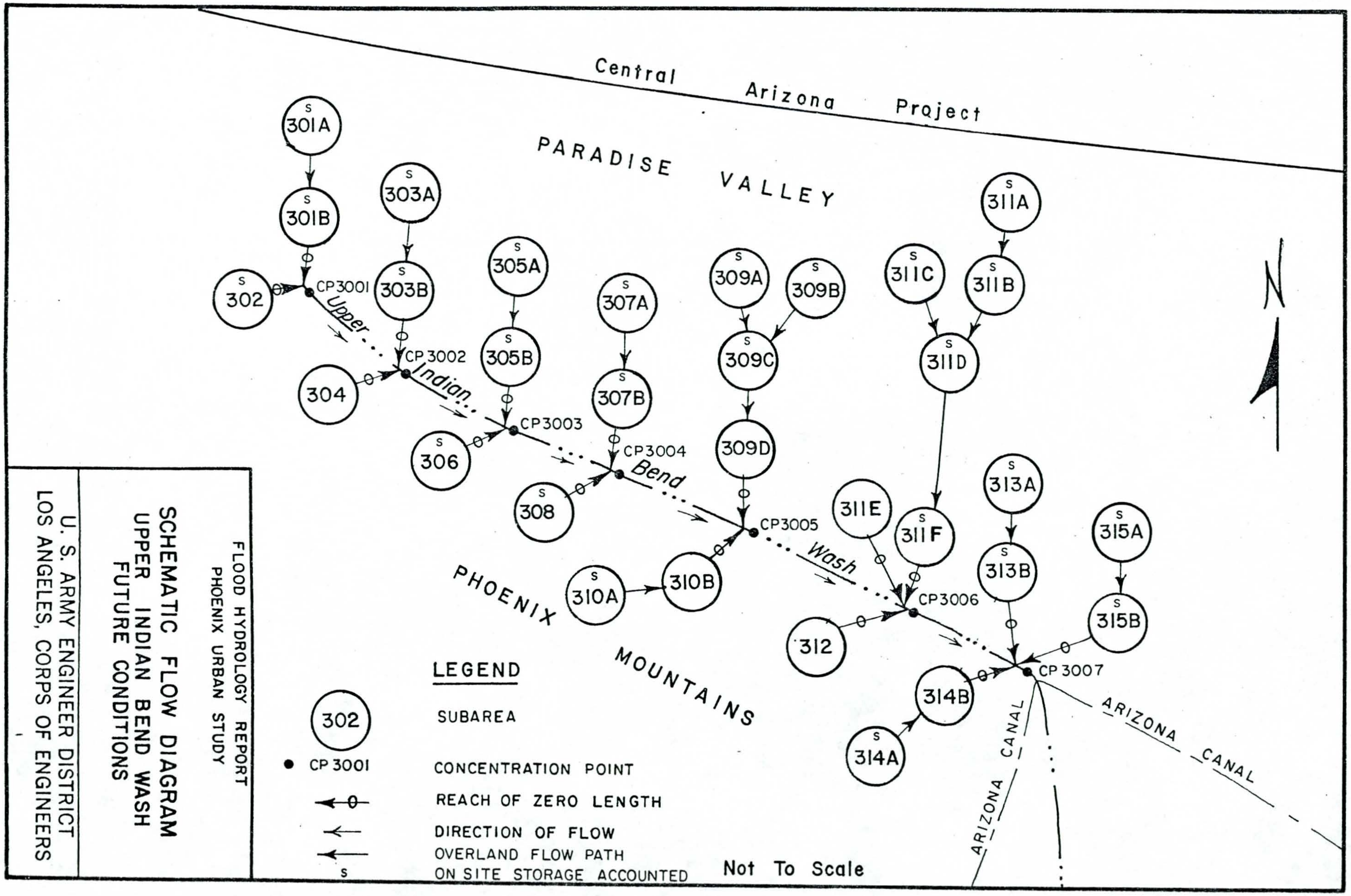
SUBAREA DELINEATION
UPPER INDIAN BEND WASH
FUTURE CONDITIONS
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

FIGURE A-13



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SCHEMATIC FLOW DIAGRAM
UPPER INDIAN BEND WASH
FUTURE CONDITIONS

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

FIGURE A-14

TABLE 4

Peak Discharges & Total Storm Runoff Volumes

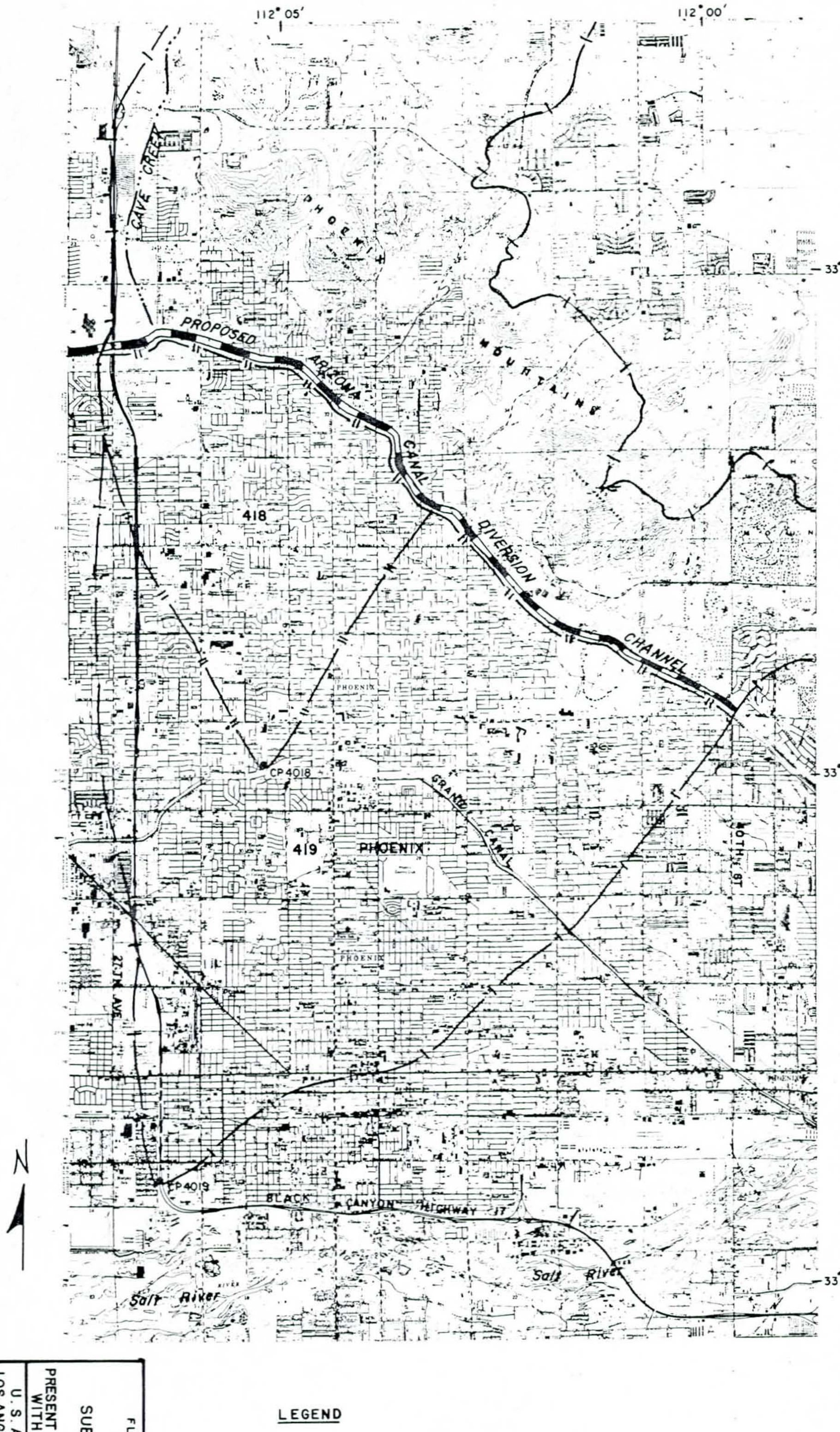
Cave Creek

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
			PRESENT CONDITIONS WITHOUT PROJECT			
Cave Creek at:						
4007	Arizona Canal	234*	50,000 (8,600)	26,000 (4,500)	14,000 (2,400)	7,000 (1,200)
4018	Grand Canal	255*	45,000 (9,400)	24,000 (5,000)	14,000 (2,900)	7,500 (1,600)
4019	Buckeye Road	302*	36,000 (11,000)	22,000 (6,800)	13,000 (4,000)	7,800 (2,400)
			FUTURE CONDITION WITHOUT PROJECT			
4007	Arizona Canal	234*	50,000 (14,200)	39,000 (7,000)	21,000 (4,500)	9,500 (2,900)
4018	Grand Canal	255*	46,000 (9,700)	38,000 (8,000)	21,000 (4,400)	10,000 (2,100)
4019	Buckeye Road	302*	37,000 (11,500)	35,000 (10,900)	21,500 (6,700)	12,000 (3,700)
			FUTURE CONDITIONS WITH PROJECT			
4007	Arizona Canal Diversion Channel	234	46,000 (14,200)	21,000 (7,000)	14,500 (4,500)	9,500 (2,900)
4071	Arizona Canal	234**	18,000 (1,400)	0	0	0
4018	Grand Canal	255**	22,000 (6,800)	6,300 (1,900)	4,500 (1,400)	2,900 (900)
4019	Buckeye Road	302**	31,000 (9,600)	14,000 (4,300)	9,900 (3,100)	6,500 (2,000)

Values in parentheses, (500), are total storm runoff volumes in acre-feet.

* For 25-year floods, runoff from 174 square miles diverted by Cave Creek Dam.

** For 100-year floods and smaller runoff from 234 square miles diverted by Arizona Canal Diversion Channel.



LEGEND

— I —	BOUNDARY OF DRAINAGE AREA
— II —	BOUNDARY OF DRAINAGE SUBAREA
419	SUBAREA DESIGNATION
• CP4019	CONCENTRATION POINT AND DESIGNATED NUMBER
▬	PROPOSED CHANNEL

SCALE IN MILES

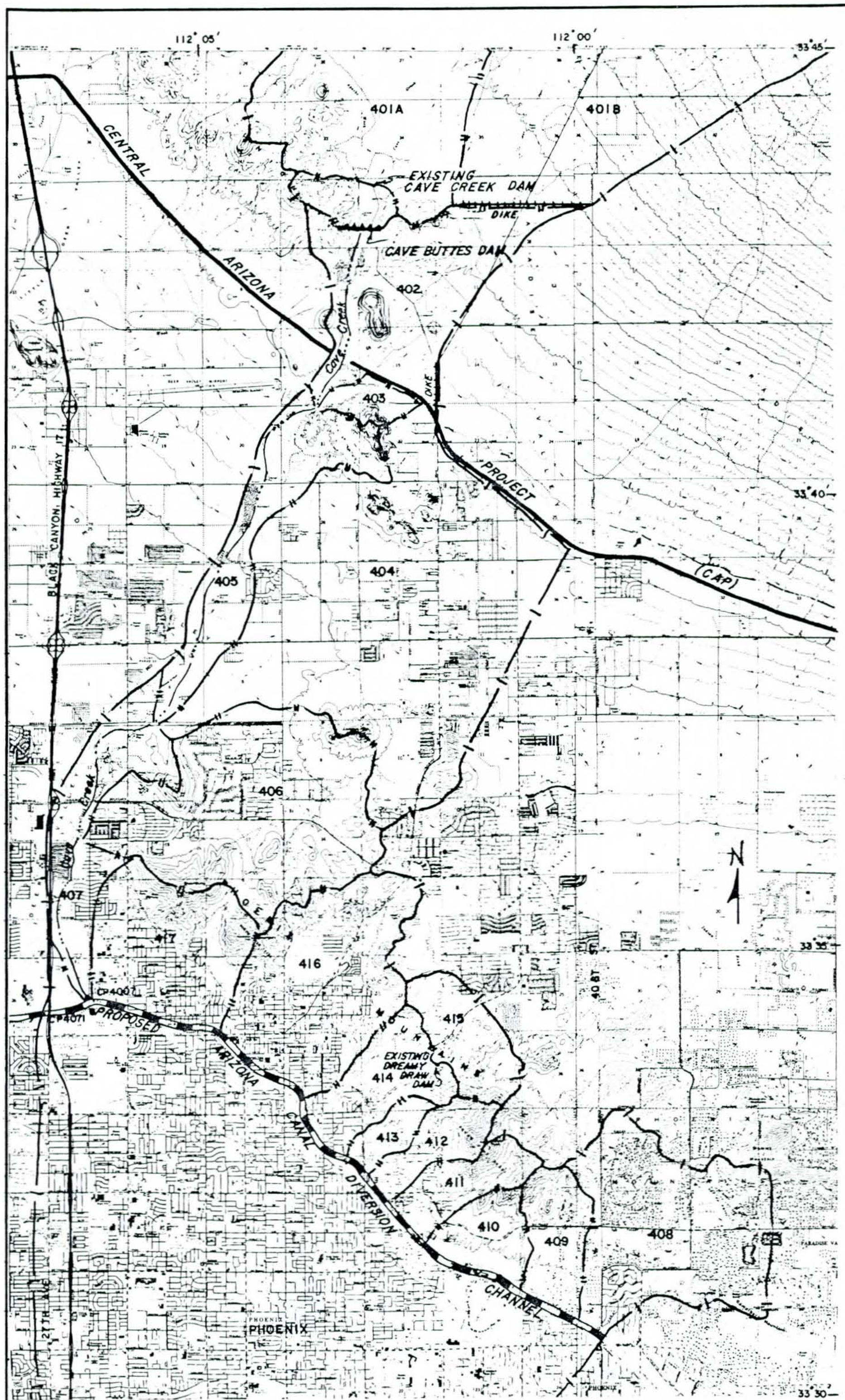
0 1 2 3

FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY

SUBAREA DELINEATION
CAVE CREEK
PRESENT AND FUTURE CONDITIONS
WITH AND WITHOUT PROJECT

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

FIGURE A-15



LEGEND

- BOUNDARY OF DRAINAGE AREA
- BOUNDARY OF DRAINAGE SUBAREA
- 407 SUBAREA DESIGNATION
- • CP4007 CONCENTRATION POINT AND DESIGNATED NUMBER
- PROPOSED CHANNEL
- PROPOSED DAM
- PROPOSED DIKE

SCALE IN MILES

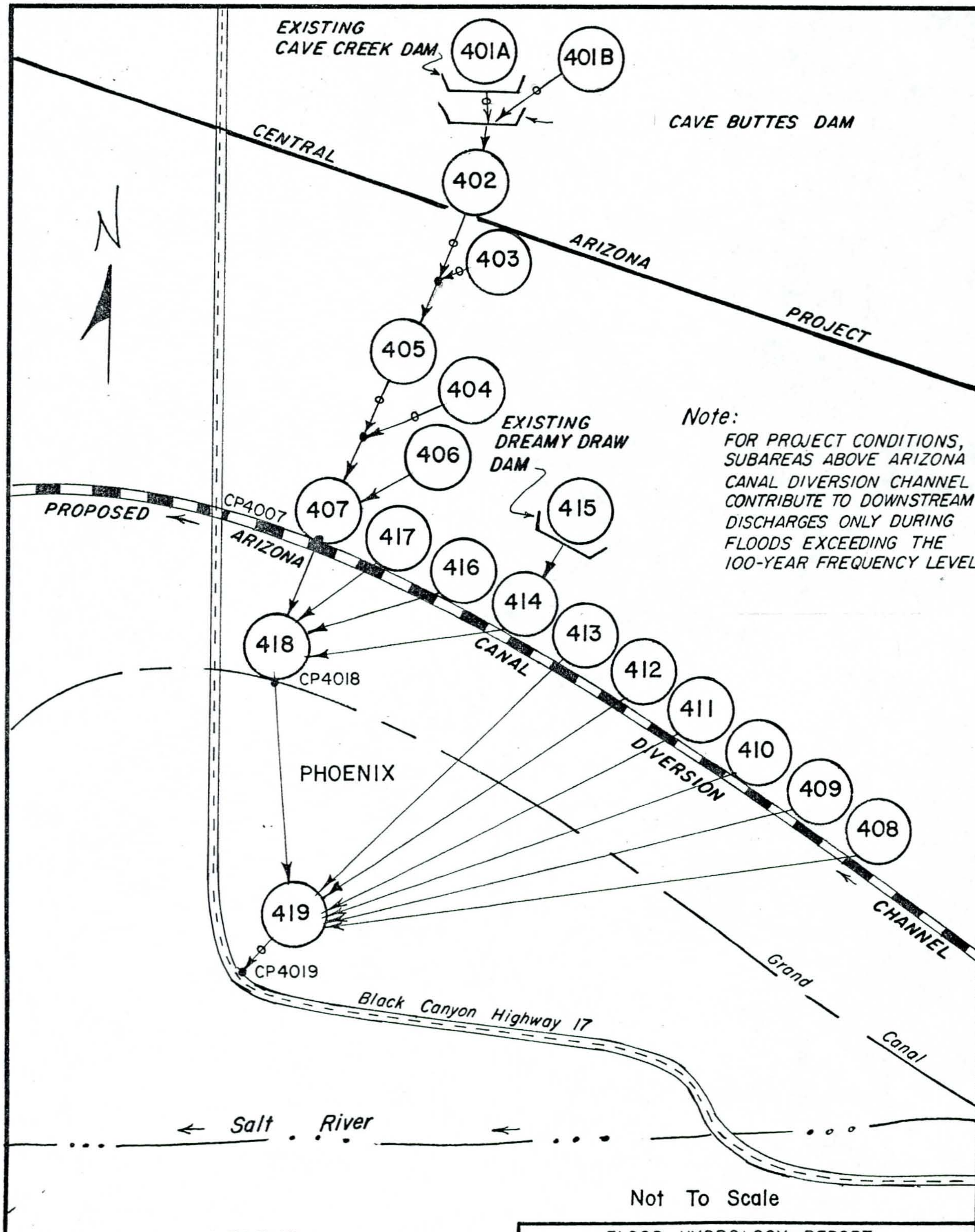
0 1 2 3

FIGURE A-16

FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY

SUBAREA DELINEATION
CAVE CREEK
PRESENT AND FUTURE CONDITIONS
WITH AND WITHOUT PROJECT

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



FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY

SCHEMATIC FLOW DIAGRAM
CAVE CREEK
PRESENT AND FUTURE CONDITIONS
WITH AND WITHOUT PROJECT

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

FIGURE A-17

TABLE 5-A

Peak Discharges & Total Storm Runoff Volumes

Old Cross Cut Canal, Present Conditions

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
5001*	Above Arizona Canal between Invergordon Rd. and Monte Vista Dr.	0.29	800 (75)	500 (35)	400 (30)	300 (25)
5002	Above Arizona Canal at 56th St.	1.15	2,600 (290)	1,400 (135)	1,200 (115)	1,000 (95)
5003*	Above Arizona Canal between Arcadia Dr. and 56th St.	0.98	2,100 (245)	1,200 (115)	1,000 (95)	800 (75)
5004*	Above Arizona Canal between 44th St. and Arcadia Dr.	0.90	2,100 (225)	1,200 (105)	1,000 (85)	800 (70)
5005*	Above Arizona Canal between 40th St. and 44th St.	0.47	1,300 (115)	700 (55)	600 (45)	500 (35)
5006	Upstream end of Old Cross Cut Canal	**	300+	300+	300+	300+

Values in parentheses, (500), are total storm runoff volume in acre-feet

* Sheet-flow front
** Flow front controlled diversion
\+ Maximum possible discharge through diversion.

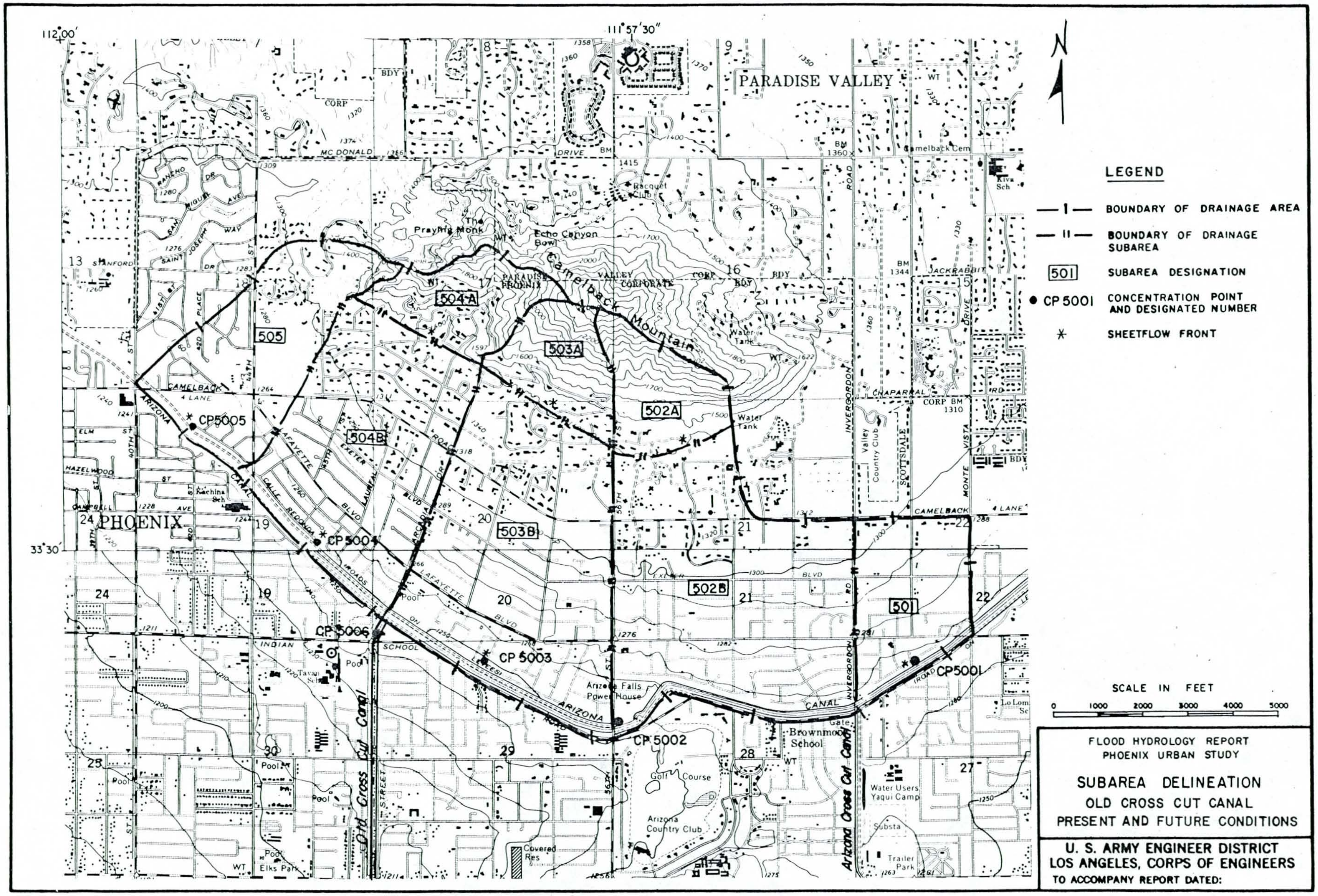
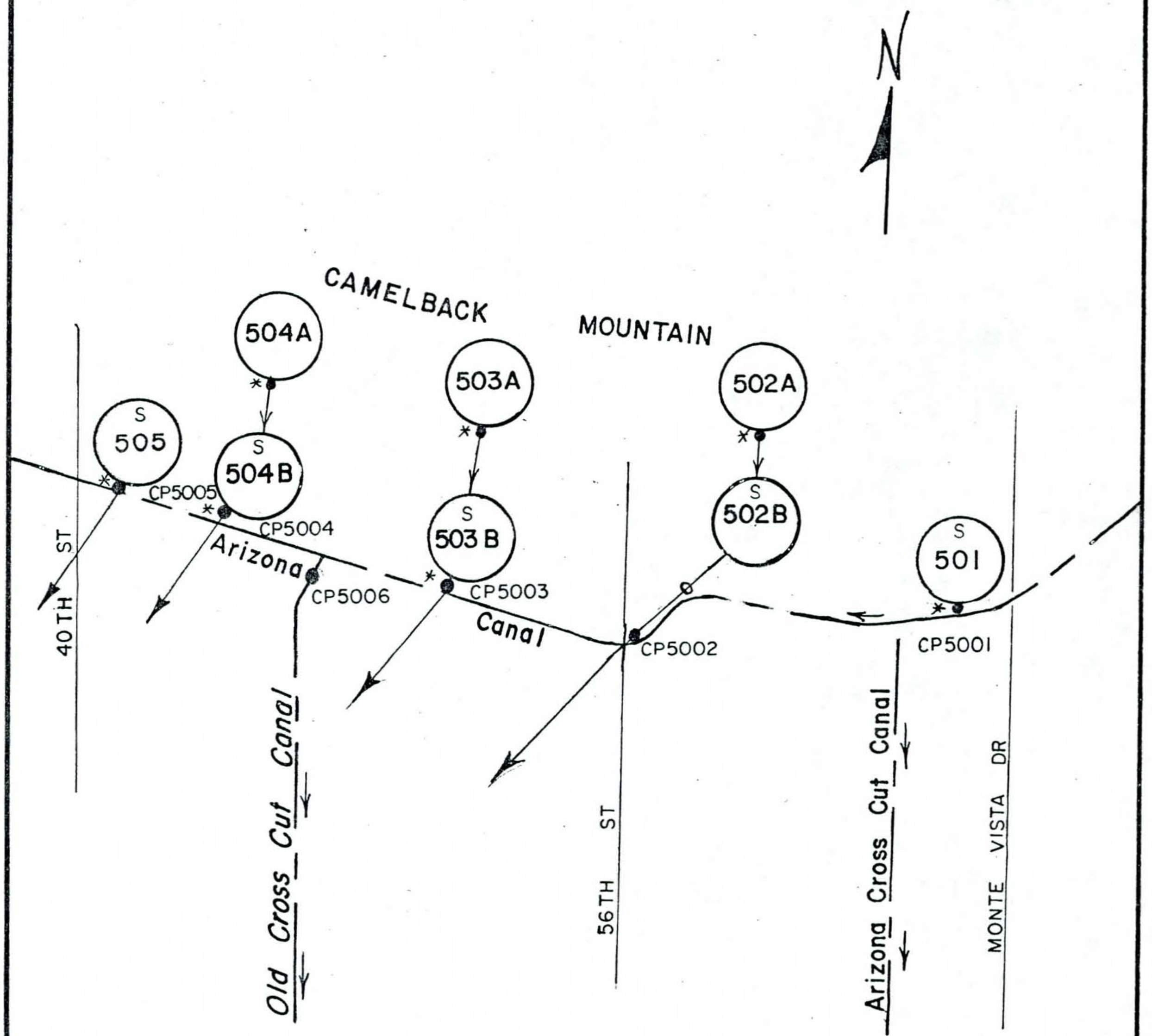

LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUBAREA
501 SUBAREA DESIGNATION
CP 5001 CONCENTRATION POINT AND DESIGNATED NUMBER
SHEETFLOW FRONT
SCALE IN FEET
0 1000 2000 3000 4000 5000
FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY
SUBAREA DELINEATION
OLD CROSS CUT CANAL
PRESENT AND FUTURE CONDITIONS
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:
PARADISE VALLEY
PHOENIX
Camelback Mountain
CP5005
CP5004
CP5006
CP 5003
CP 5002
CP5001
505
504A
504B
503A
503B
502A
502B
501
Old Cross Cut Canal
Arizona Cross Cut Canal
ARIZONA CANAL

FIGURE A-18



NOTE: DISCHARGE AT CP5006 IS LIMITED TO A MAXIMUM OF 300 CFS WHICH DUE TO THE INLET CAPACITY OF OLD CROSS CUT CANAL.

Not To Scale

LEGEND

501	SUBAREA
● CP 5001	CONCENTRATION POINT
	REACH OF ZERO LENGTH
	OVERLAND FLOW PATH
	DIRECTION OF FLOW
*	SHEETFLOW FRONT
S	ON SITE STORAGE ACCOUNTED IN FUTURE

FLOOD HYDROLOGY REPORT
PHOENIX URBAN STUDY

SCHEMATIC FLOW DIAGRAM
OLD CROSS CUT CANAL
PRESENT AND FUTURE CONDITIONS

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

FIGURE A-19

TABLE 5-B

Peak Discharges & Total Storm Runoff Volumes

Old Cross Cut Canal, Future Conditions

Concentration Point Number	Location	Contrib. Drainage Area sq. mi.	SPF CFS	100-Yr. Flood CFS	50-Yr. Flood CFS	25-yr. Flood CFS
5001*	**	0.32	900 (80)	500 (40)	400 (35)	300 (25)
5002*	**	1.15	2,600 (290)	1,400 (135)	1,200 (110)	1,000 (90)
5003*	**	1.14	2,600 (290)	1,400 (135)	1,200 (110)	900 (90)
5004*	**	0.90	2,100 (225)	1,200 (105)	1,000 (85)	800 (70)
5005*	**	0.47	1,300 (120)	700 (55)	600 (45)	500 (35)
5006*	**	***	300+	300+	300+	300+

Values in parentheses, (500), are total storm runoff volume in acre-feet.

* Sheet-flow front
** Same as present conditions, table 5A
*** Flow from controlled diversion
\+ Maximum possible discharge through diversion

TABLE 6

Subarea Characteristics in Glendale-Maryvale Area

Present and Future Conditions, With and Without Project

Subarea Designation Number	Drainage Area (sq.mi.)	Average L (mi)	Slope (ft/mi)	Basin n		Percent Imperviousness	
				Present	Future	Present	Future
101	3.71	3.40	22	0.035	0.035	30	30
102	0.36	1.50	25	0.050	0.040	15	25
103*	2.51	2.80	24	0.065	NA	5	NA
103A**	1.66	1.90	24	NA	0.040	NA	25
103B**	1.66	0.85	24	NA	0.040	NA	25
104	0.84	1.00	25	0.045	0.035	25	30
105*	6.74	3.30	24	0.070	NA	0	NA
105A**	3.54	1.60	24	NA	0.040	NA	35
105B**	3.20	1.70	24	NA	0.040	NA	25
106	4.29	3.50	22	0.065	0.030	5	40
171	0.50	0.95	18	0.035	0.035	35	35
172	5.40	4.55	18	0.040	0.035	25	30
109	3.83	2.14	23	0.0335	0.030	35	45
110	4.03	2.65	21	0.040	0.030	25	40
111	2.61	2.84	20	0.055	0.050	(10)	15
112	4.40	3.80	22	0.060	0.055	(5)	(10)
113	2.88	3.10	17	0.070	0.070	0	0

TABLE 6 (Continued)

Subarea Characteristics in Glendale-Maryvale Area

Present and Future Conditions, With and Without Project

Subarea Designation Number	Drainage Area (sq.mi.)	Average L (mi)	Slope (ft/mi)	Basin n		Percent Imperviousness	
				Present	Future	Present	Future
114	2/19	+2.39	20	0.050	0.040	15	25
115	4.19	+3.22	21	0.050	0.040	15	25
116*	4.90	3.11	20	0.060	NA	(5)	NA
116A**	1.96	1.70	17	NA	0.040	NA	25
116B**	2.94	2.85	20	NA	0.040	NA	25
117*	4.66	4.32	18	0.060	NA	(5)	NA
117A**	2.42	2.70	18	NA	0.040	NA	25
117B**	2.24	2.45	18	NA	0.040	NA	25
118	3.32	+3.88	12	0.070	0	30	

( ) Hydraulically not connected. Use zero percent impervious
NA Not applicable
* Present conditions only
** Division of present conditions subarea for future conditions because of on-site storage requirement.

TABLE 7

Subarea Characteristics in South Phoenix Area

Present and Future Conditions, With and Without Project

Subarea Designation Number	Drainage Area (sq.mi.)	L (mi)	Average L (mi)	Lea (mi)	Slope (ft/mi)	Basin n Present	Basin n Future	Percent Imperviousness Present	Percent Imperviousness Future
201	2.45	3.03	NA	1.55	370	0.045	0.045	0	0
202	0.85	1.59	NA	0.95	774	0.045	0.045	0	0
203	0.76	1.70	NA	0.80	688	0.045	0.045	0	0
204	0.27	1.74	NA	0.64	632	0.045	0.045	0	0
205	0.28	0.83	NA	0.42	928	0.045	0.045	0	0
206	0.65	1.44	NA	0.95	736	0.045	0.045	0	0
208	0.50	1.25	NA	0.61	736	0.045	0.045	0	0
209	0.15	0.76	NA	0.45	921	0.045	0.045	0	0
210	0.18	0.95	NA	9.53	768	0.045	0.045	0	0
211	0.18	0.76	NA	0.42	724	0.045	0.045	0	0
212	0.12	0.57	NA	0.15	877	0.045	0.045	0	0
213	0.18	0.61	NA	0.19	820	0.045	0.045	0	0
214	0.14	0.76	NA	0.15	579	0.045	0.045	0	0
215	0.38	1.44	NA	0.53	306	0.045	0.045	0	0
216	0.14	0.72	NA	0.19	403	0.045	0.045	0	0
217	0.30	1.55	NA	0.45	284	0.045	0.045	0	0
218	2.44	5.00	NA	1.74	236	0.045	0.045	0	0
220	0.57	NA	1.59	NA	377	0.050	0.045	0	15
221*	1.92	NA	2.34	NA	146	0.045	0.045	5	20
221**	1.10	NA	0.90	NA	100	NA	0.040	NA	25
221B**	0.63	NA	0.60	NA	333	NA	0.050	NA	0

TABLE 7 (Continued)

Subarea Designation Number	Drainage Area (sq.mi.)	L (mi)	Average L (mi)	Lea (mi)	Slope (ft/mi)	Basin n Present	Basin n Future	Percent Imperviousness Present	Percent Imperviousness Future
222	1.06	NA	1.62	NA	86	0.045	0.040	15	25
223	0.79	NA	1.28	NA	160	0.045	0.040	10	25
224	0.30	NA	1.51	NA	66	0.045	0.040	15	25
225	0.44	NA	1.40	NA	160	0.045	0.045	10	20
226	0.23	NA	1.17	NA	25	0.070	0.040	0	25
233	0.50	NA	0.41	NA	73	0.065	0.040	0	25
234	0.51	NA	0.60	NA	67	0.055	0.040	10	25
240	3.84	NA	3.29	NA	39	0.065	0.040	0	25
241	2.58	NA	1.66	NA	48	0.045	0.035	20	30
242	0.79	NA	0.83	NA	660	0.040	0.040	25	25
243	1.31	NA	0.87	NA	57	0.050	0.035	15	30
244	0.95	NA	0.94	NA	53	0.060	0.040	5	25
245	1.11	NA	1.36	NA	29	0.065	0.040	5	25
246	1.87	NA	1.51	NA	33	0.045	0.040	20	20
250	2.33	NA	2.12	NA	177	0.030	0.030	20	25
251	1.24	NA	1.61	NA	18	0.045	0.040	20	30

TABLE 7 (Continued)

Subarea Designation Number	Drainage Area (sq.mi.)	L (mi)	Average L (mi)	Lea (mi)	Slope (ft/mi)	Basin n Present	Basin n Future	Percent Imperviousness Present	Percent Imperviousness Future
252	0.85	NA	1.40	NA	14	0.025	0.020	10	70
260*	2.02	NA	3.25	NA	15	0.049	0.040	15	25
260**	1.16	NA	3.03	NA	15	NA	0.040	NA	25
261	3.63	NA	2.08	NA	10	0.030	0.030	40	40
271**	3.10	NA	1.90	NA	524	NA	0.045	NA	15
272**	1.07	NA	1.70	NA	32	NA	0.040	NA	25

NA Not applicable
* Present & future without project only.
** Future with project only.

Note:
1. Subarea 201 thru 218: Unit hydrograph method-method for confined flow used.
   Subarea 202 thru 272: Single linear reservoir-method for sheet-flow used.

TABLE 8-A

Subarea Characteristics in Upper Indian Bend Wash Area

Present Conditions

Subarea Designation Number	Drainage Area (sq.mi.)	Average L (mi.)	Slope (ft/mi)	Basin n	Percent Imperviousness
301	2.44	3.00	34	0.053	5
302	0.33	1.25	275	0.053	7
303	2.80	3.80	32	0.049	20
304	3.60	2.90	84	0.053	11
305	3.00	4.20	32	0.054	3
306	2.90	2.80	63	0.052	14
307	5.40	4.70	34	0.053	2
308	3.80	4.40	317	0.052	4
309	5.67	5.20	34	0.056	4
310	4.30	2.65	317	0.055	7
311	7.82	600	39	0.037	8
312	2.20	2.65	211	0.056	13
313	810	6.70	20	0.061	4
314	1.20	2.20	158	0.065	14
315	6.80	7.00	34	0.054	2
316	3.80	2.00	185	0.054	12

Note:
1. Single linear reservoir method for sheet-flow used for all subareas

TABLE 8-B

Subarea Characteristics in Upper Indian Bend Wash Area

Future Conditions

Subarea Designation Number	Drainage Area (sq.mi.)	Average L (mi.)	Slope (ft/mi)	Basin n	Percent Imperviousness
301A	1.32	1.30	40	0.040	26
301B	1.12	1.70	23	0.034	35
302	0.33	1.25	275	0.049	20
303A	0.62	1.10	37	0.039	29
303B	2.18	2.65	24	0.040	26
304	3.60	2.90	84	0.047	22
305A	0.82	0.80	42	0.052	16
305B	2.18	3.20	26	0.040	28
306	2.90	2.80	47	0.038	31
307A	0.34	0.40	42	0.050	0
307B	5.06	4.50	32	0.031	39
308	3.80	4.40	317	0.054	13
309A	3.65	3.80	35	0.046	23
309B	0.25	0.50	24	0.041	25
309C	1.32	1.00	26	0.054	19
309D	0.45	0.50	26	0.049	19
310A	1.10	.090	333	0.054	13
310B	3.20	2.40	46	0.048	19
311A	0.54	0.50	41	0.050	22
311B	1.00	2.10	45	0.034	34

TABLE 8-B (Continued)

Subarea Designation Number	Drainage Area (sq.mi.)	Average L (mi.)	Slope (ft/mi)	Basin n	Percent Imperviousness
311C	2.40	2.90	39	0.037	28
311D	2.10	1.60	32	0.040	27
311E	1.58	1.70	21	0.042	23
311F	0.20	1.60	24	0.041	25
312	2.20	2.60	211	0.049	19
313A	0.35	0.40	46	0.041	25
313B	7.75	5.60	33	0.046	21
314A	0.90	1.50	264	0.042	25
314B	0.30	0.90	28	0.066	11
315A	0.15	0.20	58	0.41	2
315B	6.65	7.00	46	0.043	25

Note:
1. Single linear reservoir method for sheet-flow used for all subareas.

TABLE 9

Subarea Charateristics in Cave Creek Area

Present and Future Conditions

Subarea Designation Number	Drainage Area (sq.mi.)	L (mi)	Average L (mi)	Lea (mi)	Slope (ft/mi)	Basin n Present	Basin n Future	Percent Imperviousness Present	Percent Imperviousness Future
401A	174.44	32.10	NA	17.10	88	0.038	0.038	5	5
401B	16.88	18.10	NA	7.20	117	0.030	0.030	5	5
402	4.56	4.73	NA	2.25	32	0.029	0.029	5	5
403	0.76	1.87	NA	0.82	43	0.030	0.030	5	5
404	22.04	7.69	NA	4.32	36	0.033	0.022	5	45
405	5.36	1.75	NA	0.85	29	0.033	0.030	10	40
406	6.52	7.10	NA	3.96	80	0.033	0.028	10	40
407	3.27	2.29	NA	1.16	34	0.033	0.030	10	40
408	4.91	3.95	NA	1.87	222	0.035	0.025	35	55
409	1.38	1.94	NA	0.97	227	0.035	0.028	20	40
410	1.22	1.19	NA	0.45	298	0.035	0.031	15	35
411	1.12	2.09	NA	0.97	268	0.038	0.028	20	40
412	0.91	2.24	NA	1.27	313	0.038	0.028	20	40
413	0.63	1.19	NA	0.60	101	0.035	0.028	20	40
414	1.56	2.24	NA	1.42	80	0.038	0.034	18	35
415	1.26	2.20	NA	1.00	332	0.045	0.040	5	13
416	4.81	3.28	NA	1.49	73	0.030	0.025	35	50
417	2.87	2.24	NA	0.75	225	0.035	0.025	50	50
418*	10.68	NA	4.74	NA	23	0.025	0.025	50	50
419*	36.71	NA	11.00	NA	20	0.025	0.025	50	50

* Simple linear reservoir method for sheet flow used. Unit hygrograph method for confined flow used for all other areas.

NA Not applicable

TABLE 10

Subarea Characteristics in Old Cross Cut Canal Area

Present and Future Conditions

Subarea Designation Number	Drainage Area (sq.mi.)	Average L (mi)	Slope (ft/mi)	Basin n Present	Basin n Future	Percent Imperviousness Present	Percent Imperviousness Future
501	0.29*	0.87	56	0.041	0.041	25	25
502A	0.20	0.38	2100	0.050	0.050	0	0
502B	0.95	1.30	78	0.042	0.041	22	25
503A	0.22	0.38	2890	0.050	2	2	
503B	0.76**	1.50	119	0.043	0.041	19	25
504A	0.20	0.38	1370	0.049	0.049	3	3
504B	0.70	1.14	132	0.043	0.041	19	25
505	0.47	1.21	168	0.045	0.043	13	20

* 0.29 sq. mi. in present, 0.32 sq. mi. in future
** 0.76 sq. mi. in present, 0.92 sq. mi. in future

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