FINAL REPORT
UPPER INDIAN BEND WASH
REGIONAL DRAINAGE AND FLOOD
CONTROL PLAN

VOLUME I

Prepared for:

CITY OF SCOTTSDALE
3939 Civic Center Plaza
Scottsdale, AZ 85251

Prepared by:

WATER RESOURCES ASSOCIATES, INC.
4041 N. Central Ave., Suite 1050
Phoenix, AZ 85012-3393

July 6, 1992

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ARIZONA, U.S.A.

July 6, 1992

TABLE OF CONTENTS

	PAGE NO.
IMPORTANT NOTICE	i
EXECUTIVE SUMMARY	iv
TABLE OF CONTENTS	vi
LIST OF FIGURES	ix
LIST OF TABLES	xi
LIST OF APPENDICES	xiii
LIST OF PLATES	xiv
1.0 INTRODUCTION	1
1.1 Background Information	1
1.2 Study Objectives	2
1.3 Organization of Study Results	3
2.0 DRAINAGE BASIN CHARACTERISTICS	4
2.1 Drainage Area	4
2.2 Land Use	5
2.3 Soil Type and Vegetation	6
2.4 Existing Drainage Improvements	7
3.0 HYDROLOGIC MODEL (HEC-1)	8
3.1 Overview of Hydrologic Modeling Assumptions	9
3.2 Delineation of Drainage Sub-basins	14
3.3 Interception and Infiltration	15
3.4 Overland Flow Parameters	17
3.5 Channel Routing Parameters	19
3.6 Rainfall Parameters	22
3.7 Verification of HEC-1 Model	23
3.7.1 North Phoenix Area	25
3.7.2 North Scottsdale Area	30

TABLE OF CONTENTS (Cont.)

		PAGE NO.
4.0	HYDROLOGIC AND HYDRAULIC ANALYSIS OF PROPOSED FLOOD CONTROL CHANNELS	35
4.1	Channel Design Assumptions	35
4.1.1	Design Discharge	35
4.1.2	Geometry	36
4.1.3	Channel Slope	37
4.1.4	Freeboard	37
4.1.5	Bank-Lining	37
4.1.6	Scour	39
4.1.7	Low-Flow Culverts	39
4.1.8	Maintenance Access to Channels	42
4.1.9	Bridges	43
4.2	Proposed Channel Alignments	43
4.2.1	Jomax Road Channel	44
4.2.2	Squaw Peak Parkway Channel	44
4.2.3	Pinnacle Peak Road Channel (East)	45
4.2.4	Pinnacle Peak Road Channel (West)	46
4.2.5	Rawhide Wash Channel	46
4.2.6	Scottsdale Road Channel	48
4.2.7	Pima Road Channel	48
4.2.8	Deer Valley Road Channel, Hayden to Pima	50
4.2.9	Deer Valley Road Channel, Hayden to Scottsdale to Beardsley	51
4.2.10	Reata Pass Channel	52
4.2.11	Core North Drainage System	54
4.2.12	Core South Drainage System	56
5.0	OUTER LOOP HIGHWAY DRAINAGE STRUCTURES	58
5.1	Squaw Peak Parkway Crossing	58
5.2	Desert Ridge Crossings	58
5.3	Rawhide Wash Crossing	59
5.4	Outer Loop Section 9A	60
6.0	SEDIMENT TRANSPORT AND SCOUR ANALYSIS	62
6.1	Scour Analysis	62
6.1.1	Summary of Scour Analysis	65

TABLE OF CONTENTS (Cont.)

		PAGE NO.
6.2	Equilibrium Slope Analysis	66
6.3	Drop Structure Requirements	73
7.0	**IMPACTS TO CAP DIKES**	75
8.0	**INTERIM SOLUTIONS**	78
9.0	**COST ESTIMATE**	82
10.0	**DISTRIBUTION OF INSTALLATION COSTS**	85
11.0	**INSTALLATION SCHEDULE**	88
12.0	**RECOMMENDATIONS FOR FINAL DESIGN**	89
12.1	Hydrology Analysis	89
12.2	Hydraulic and Sediment Transport Analysis	90
12.3	Miscellaneous Design Recommendations	92

FIGURES

TABLES

APPENDICES

PLATES

LIST OF FIGURES

FIGURE	DESCRIPTION
2.1	Study Boundary
3.1	Percent of Impervious Area vs Dwelling Units/Acre
3.2	Graphical Plot of SCS type IIA Rainfall Distribution
3.3	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3020
3.4	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3135
3.5	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3150
3.6	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3160
3.7	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3400
3.8	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3490
3.9	Unit Discharge vs Drainage Area, Non-Sheetflow Areas, City of Phoenix - Areas C and D
3.10	Unit Discharge vs Drainage Area, Sheetflow Areas, City of Phoenix - Areas C and D
3.11	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 2240
3.12	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 2255
3.13	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 2260
3.14	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 2270
3.15	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 2290

LIST OF FIGURES (Cont.)

FIGURE	DESCRIPTION
3.16	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 2300
3.17	Unit Discharge vs Drainage Area, Non-Sheetflow Areas, North Scottsdale
3.18	Unit Discharge vs Drainage Area, Sheetflow Areas, North Scottsdale
4.1	Geometry of 4-Point Channel Cross-Section
4.2	Geometry of 8-Point Channel Cross-Section
4.3	Channel Lining Cut-Off Wall
4.4	Schematic of Low-Flow Culvert Installation
4.5	Schematic of Channel Access Ramps
4.6	Alternate Channel Alignments for Reata Pass
4.7	Schematic of Core North Drainage System
6.1	Typical Soil-Cement Drop Structure Geometry
11.1	Project Installation Schedule

LIST OF TABLES

TABLE	DESCRIPTION
2.1	Summary of Hydraulic Soil Group Assigned to Soil Complex/Associations
3.1	Summary of SCS Curve Numbers as a Function of Storm Duration and Hydrologic Soil Group
3.2	Percent of Impervious Cover for Residential Land-Use, Adjusted for NAOS Requirements
3.3	Summary of Overland Flow Roughness Values
3.4	Summary of Rainfall Values
3.5	Summary of Independent Peak Discharge Calculations for Verification of HEC-1 Model Results, North Phoenix Area
3.6	Summary of Independent Peak Discharge Calculations for Verification of HEC-1 Model Results, North Scottsdale Area
4.1	Summary of Low-Flow Culvert Costs
4.2	Summary of Bridge Costs, No Low-Flow Culverts
5.1	Summary of Channel/Culvert Intersections with the Outer Loop Highway
5.2	Performance Summary, Detention Basin 53R-Core North
6.1	Jomax Road Channel, Equilibrium Slope Analysis, No Low-Flow Culverts
6.2	Squaw Peak Parkway Channel Equilibrium Slope Analysis, Channel No Low-Flow Culverts
6.3	Pinnacle Peak Road Channel, East of Squaw Peak Parkway, Equilibrium Slope Analysis, No Low-Flow Culverts
6.4	Pinnacle Peak Road Channel, West of Squaw Peak Parkway, Equilibrium Slope Analysis, No Low-Flow Culverts
6.5	Rawhide Wash Channel, Equilibrium Slope Analysis, No Low-Flow Culverts

LIST OF TABLES (Cont.)

TABLE	DESCRIPTION
6.6	Scottsdale Road Channel, Equilibrium Slope Analysis, No Low-Flow Culverts
6.7	Pima Road Channel, Equilibrium Slope Analysis, No Low-Flow Culverts
6.8	Deer Valley Road Channel, Pima Road to Hayden Road, Equilibrium Slope Analysis, Pima Road Channel Is Not In Place
6.9	Deer Valley Road Channel, Hayden Road to Scottsdale Road to Beardsley Road, Equilibrium Slope Analysis, Upstream Drainage Improvements Are In-Place
6.10	Reata Pass Channel, Equilibrium Slope Analysis, No Low-Flow Culverts
7.1	Summary of CAP, Detention Basin Performance Existing Drainage Pattern vs Proposed Regional Drainage Plan
9.1	Watershed Conditions Used for Developing Cost Estimates
9.2	Summary of Project Installation Costs, Scenario No. 1
9.3	Summary of Project Installation Costs, Scenario No. 2
9.4	Summary of Project Installation Costs, Scenario No. 3
9.5	Summary of Project Installation Costs, Scenario No. 4

LIST OF APPENDICES

APPENDIX	DESCRIPTION
Table A.1	Hydrologic, Hydraulic, and Construction Cost Data for Jomax Road Channel and Squaw Peak Parkway Channel North of Pinnacle Peak Road
Table A.2	Hydrologic, Hydraulic, and Construction Cost Data for Pinnacle Peak Road Channel (East) and Squaw Peak Parkway Channel South of Pinnacle Peak Road, Road, No Low-Flow Culverts
Table A.3	Hydrologic, Hydraulic, and Construction Cost Data for Pinnacle Peak Road Channel (West), No Low-Flow Culverts
Table A.4	Hydrologic, Hydraulic, and Construction Cost Data for Rawhide Wash Channel, No Low-Flow Culverts
Table A.5	Hydrologic, Hydraulic, and Construction Cost Data for Scottsdale Road Channel, No Low-Flow Culverts
Table A.6	Hydrologic, Hydraulic, and Construction Cost Data for Pima Road Channel, No Low-Flow Culverts, No Reata Pass Flows
Table A.7	Hydrologic, Hydraulic, and Construction Cost Data for Pima Road Channel, No Low-Flow Culverts, with Reata Pass Flows
Table A.8	Hydrologic, Hydraulic, and Construction Cost Data for Deer Valley Road Channel, Pima to Hayden, No Low-Flow Culverts
Table A.9	Hydrologic, Hydraulic, and Construction Cost Data for Deer Valley Road Channel, Hayden to Scottsdale to Beardsley, No Low-Flow Culverts
Table A.10	Hydrologic, Hydraulic, and Construction Cost Data for Reata Pass Channel, Thompson Peak Parkway Alignment, No Low-Flow Culverts

LIST OF PLATES

PLATE	DESCRIPTION
1	Hydrologic Soil Group Map
2	Watershed Sub-Basin Delineations and HEC-1 Routing for Existing Drainage Pattern
3	Watershed Sub-Basin Delineations and HEC-1 Routing Schematic for Proposed Master Drainage Plan
4	Aerial Photograph of Proposed Master Drainage Plan
5	Existing Ground Profile, Squaw Peak Parkway Channel, Jomax Road Channel, East Pinnacle Peak Road Channel
6	Existing Ground Profile, West Pinnacle Peak Road Channel, Deer Valley Road Channel - West, Deer Valley Road Channel - East
7	Existing Ground Profile, Rawhide Wash Channel, Scottsdale Road Channel
8	Existing Ground Profile, Pima Road Channel
9	Existing Ground Profile, Reata Pass Channel, East Side of Thompson Peak Parkway

IMPORTANT NOTICE
(PLEASE READ)

A preliminary draft report for the Upper Indian Bend Wash Regional Drainage and Flood Control Plan was published on September 5, 1990. A "Notice" in the front of that report discussed five unresolved issues that would require further discussion and/or analysis prior to publication of a final report for the regional drainage plan concept. Each of these five issues is reprinted in the following paragraphs along with a response indicating what action has been taken to resolve the issue since publication of the preliminary draft report.

<u>Issue No. 1</u> - A workable alignment has not been identified for the Reata Pass channel. The Thompson Peak Parkway alignment does not include sufficient slope to allow the channel to pass the incoming sediment flows. Some alternate alignments are discussed in this report, but they have not been studied from an engineering standpoint. Accordingly, the costs shown in this report for the Reata Pass channel may be substantially different from what will actually be incurred.

Response - The City of Scottsdale created the Reata Pass Task Force to evaluate alternate alignments for the Reata Pass channel. The Task Force published a draft report in May 1991 entitled the *Reata Pass/Beardsley Wash Alignment Study*. As a result of this study, the "Reata Pass Alignment" was identified as the preferred alternative.

The Task Force study did not include the preparation of an engineering study to analyze the hydraulic performance of this channel alignment. However, the study did include a rough construction cost estimate of about $36,000,000. This cost includes channel construction, bridge crossings, tributary channels, an apex structure, an outlet structure, landscaping, right-of-way, and an allowance for engineering fees.

For comparison purposes, the costs shown in this report for the Reata Pass channel (along the east side of the Thompson Peak Parkway) ranged from $42 million to $43.5 million (see Tables 9.2 and 9.3).

This alternate channel alignment that has been recommended by the Reata Pass Task Force is shown on Plate 4 and Figure 4.6 of this report.

Issue No. 2 - The Core North detention basins need to be subjected to a rigorous engineering analysis. Using the reservoir geometry and outlet configuration provided by the Core North planning team, the basins were found to overtop when linked to the proposed regional drainage plan. Although basin 53R has been reconfigured (as part of this study) to perform satisfactorily, basin 38R-2 still overtops during certain scenarios. This preliminary analysis also indicates that basin 53R may have to be designed as a dam if it is to contain the Reata Pass flows.

Response - Since the September 5, 1990 publication of the preliminary draft report for the Upper Indian Bend Wash Regional Plan, the City of Scottsdale has been working with the Core North planning team to develop an acceptable drainage plan for this parcel of State Trust Land. This recent planning effort has resulted in revisions to the detention basin configurations that are different from those published in the September 1990 report.

New HEC-1 models have been developed by the City of Scottsdale to incorporate these detention basin revisions. These revisions were included in a draft copy of an in-house City of Scottsdale engineering memorandum dated June 19, 1992. These revisions have not been reviewed nor endorsed by Water Resources Associates, Inc. (WRA) and are not included in the HEC-1 models published with this final report.

When all interested parties (City of Scottsdale, Arizona State Land Department, Arizona Department of Transportation, and the Core North planning team) are satisfied with the hydraulic performance and technical accuracy of these detention basin modeling revisions, the master plan HEC-1 models in this report should be updated to reflect such revisions as part of any final design effort.

Issue No. 3 - Changes to the locations and discharge capacity of Core North detention basin 53R will require a new Core South outlet channel, and may impact the peak design discharge in other Core South channels. Revisions to other Core South channel alignments are also reportedly being considered. None of these potential impacts have been evaluated as part of this study.

Response - The recent City of Scottsdale planning study referenced under Issue No. 2 also re-configured Detention Basin 53R. The outlets from Detention Basin 53R have also been changed to direct flows into the Core South channel system at different locations than were originally depicted in a 1987 drainage study prepared for Core South by Simons, Li and Associates, Inc. (SLA).

None of the Detention Basin 53R configuration changes, nor any of the outlet channel alignment changes, have been reviewed or endorsed by WRA. The HEC-1 models published in this final report reflect the Core South channel system as presented in the 1987 SLA report. This includes routing flows from Sub-basins 21A and 21B to the Central Arizona Project (CAP) Dike 3.

The HEC-1 models published in this final report should be updated to reflect any approved changes to basin 53R and/or the Core South channel system prior to initiating final design of any of these drainage system features.

Issue No. 4 - Additional coordination is required with ADOT, the State Land Department, and Core North - Core South interests to ensure that local drainage along the north side of Section 9A of the Outer Loop Highway has been adequately addressed. This coordination also needs to ensure that unanimous agreement has been reached on the size and location of all highway culverts.

Response - As part of the in-house City of Scottsdale planning effort, coordination has been maintained with ADOT, the State Land Department, and Core North-Core South interests in order to reach agreement on the disposition of drainage along the north side of Section 9A of the Outer Loop Highway. At the time this final report was published, drainage system details were still under discussion in an effort to satisfy the interests of all concerned parties.

Issue No. 5 - Section 9.0 of this report lists several items that are excluded from the cost estimate for the regional drainage plan. Task force members may prefer that some of these items be included in the cost estimate. If this occurs, additional analysis will have to be conducted to generate these costs.

Response - The City of Scottsdale did not request the consultant to include any additional cost items beyond those published in Section 9.0 of the September 5, 1990 report.

Revised per Bob Ward 10/30/92

EXECUTIVE SUMMARY

Portions of both Phoenix and Scottsdale, located north of the CAP aqueduct, are on the verge of undergoing major urbanization. This region of the Sonoran Desert is composed of pediments, alluvial plains and alluvial fans. These landforms create unique drainage problems because of their unpredictable distributary flow patterns. The random distribution of floodwaters across such landforms creates a need for the development of regional drainage plans that possess the capability of capturing runoff from any location within the watershed and safely conveying such runoff to a suitable outlet. Such regional plans provide control over an otherwise random and uncontrolled drainage pattern. The purpose of this report is to present the engineering analysis that was undertaken to develop an integrated regional drainage and flood control plan for a portion of the Upper Indian Bend Wash watershed.

The success of regional drainage plans is somewhat dependent upon developing the plan prior to the occurrence of extensive urbanization. This prevents the construction of "band-aid" drainage plans that may be scattered across the watershed. Such plans are not normally linked together, and often simply transfer a drainage problem from one site to another. In contrast to this somewhat unorganized approach, an integrated regional plan links the entire watershed into a single, synchronized system of channels and detention basins.

The regional plan presented in this report is a combination of concepts presented in previous general planning studies prepared for both the cities of Phoenix and Scottsdale. These concepts have been supplemented with additional drainage features developed by consultants for Desert Ridge, Core North, and Core South. The plan has also been coordinated with ADOT to ensure compatibility with off-site drainage structures required for the Outer Loop Highway. Throughout this planning process, input was also provided by a multi-agency Task Force, which included representatives from federal, state, county, municipal, and private organizations.

The regional drainage system that emerged from this coordinated planning effort consists of approximately 35 linear miles of major interceptor channel and three detention basins. Although some details of the system will require further refinement, the installation cost of the system is estimated to range from about $160,000,000 to $175,000,000.

EXECUTIVE SUMMARY

Portions of both Phoenix and Scottsdale, located north of the CAP aqueduct, are undergoing urbanization in accordance with land-use plans developed by both the City of Scottsdale and the City of Phoenix. This region of the Sonoran Desert is composed of pediments, alluvial plains and alluvial fans. These landforms create unique drainage problems because of their unpredictable distributary flow patterns. The random distribution of floodwaters across such landforms creates a need for the development of regional drainage plans that possess the capability of capturing runoff from any location within the watershed and safely conveying such runoff to a suitable outlet. Such regional plans provide control over an otherwise random and uncontrolled drainage pattern. The purpose of this report is to present the engineering analysis that was undertaken to develop an integrated regional drainage and flood control plan for a portion of the Upper Indian Bend Wash watershed.

The success of regional drainage plans is somewhat dependent upon developing the plan prior to the occurrence of any proposed urbanization. This prevents the construction of "band-aid" drainage plans that may be scattered across the watershed. Such plans are not normally linked together, and often simply transfer a drainage problem from one site to another. In contrast to this somewhat unorganized approach, an integrated regional plan links the entire watershed into a single, synchronized system of channels and detention basins.

The regional plan presented in this report is a combination of concepts presented in previous general planning studies prepared for both the cities of Phoenix and Scottsdale. These concepts have been supplemented with additional drainage features developed by consultants for Desert Ridge, Core North, and Core South. The plan has also been coordinated with ADOT to ensure compatibility with off-site drainage structures required for the Outer Loop Highway. Throughout this planning process, input was also provided by a multi-agency Task Force, which included representatives from federal, state, county, municipal, and private organizations.

The regional drainage system that emerged from this coordinated planning effort consists of approximately 35 linear miles of major interceptor channel and three detention basins. Although some details of the system will require further refinement, the installation cost of the system is estimated to range from about $160,000,000 to $175,000,000.

The report that accompanies this Executive Summary presents details of the engineering analysis used to develop the plan, and discusses possible interim solutions, funding sources, an installation schedule, and recommendations for final design.

1.0 INTRODUCTION

1.1 BACKGROUND INFORMATION

Portions of both Phoenix and Scottsdale are on the verge of experiencing major urbanization through natural desert areas located north of the CAP aqueduct and west of the McDowell Mountain Drainage divide. Both municipalities have developed "General Plans" to guide urbanization of these areas in a controlled and coordinated manner that will preserve the natural beauty of the desert environment.

An integral part of this urbanization is the installation of a flood control and drainage system. Since 1988, both Phoenix and Scottsdale have commissioned engineering studies to investigate the feasibility of installing various drainage concepts for this region (*General Drainage Plan for North Scottsdale, Arizona*, June 7, 1989, and *Concept Drainage Study, Paradise Valley Fan Terrace, Part of Phoenix Peripheral Areas C and D*, February 1990). Each of these studies examined a wide range of possible drainage concepts. These concepts included channelization, detention basins, retention basins, as well as a "no action" alternative. during this same period, the Arizona Department of Transportation (ADOT) was conducting engineering studies to develop an off-site drainage system for that reach of the Outer Loop Highway that is to be located north of the CAP.

In addition to these government-sponsored drainage studies, several private entities were simultaneously proceeding with preliminary planning efforts for the development of large tracts of land within the study area, for example, Desert Ridge, Core North, Core South, and so forth. This abundance of planning activity, both at the government and private level, clearly indicated an urgent need to synchronize these multi-entity efforts into a coordinated, regional drainage plan. Accordingly, the City of Scottsdale organized an inter-agency task force to oversee the development of such a regional plan. The task force included representatives from the following organizations.

- City of Scottsdale
- City of Phoenix
- Arizona State Land Department
- Arizona Department of Water Resources
- Arizona Department of Transportation

- Flood Control District of Maricopa County
- Bureau of Reclamation
- Corp of Engineers
- BRW, Inc. (Desert Ridge)
- Carter Associates, Inc. (Core North)
- Santa Fe Management (Core North)
- Stanley Consultants
- DeLeuw, Cather & Company (ADOT management consultant for Outer Loop Highway)

WRA was retained by the City of Scottsdale to provide the hydrologic and hydraulic analyses of the drainage system concepts that would ultimately comprise the regional drainage plan presented in this report. WRA met with the Task Force at periodic intervals (approximately every 4 to 6 weeks) to present intermediate progress on the drainage system analyses. These meetings were also used to obtain input and comments from the Task Force regarding preferred channel alignments, crossing locations of the Outer Loop Highway, and on any other matters that Task Force members felt were relevant to the successful design of the regional drainage plan. Throughout this process, revisions were made to channel alignments and detention basin locations until a regional plan emerged that had the full support of the Task Force. This report describes the regional drainage plan and the technical analyses, assumptions, and so forth, upon which the plan is based.

1.2 STUDY OBJECTIVES

The purpose of this study was to develop a regional drainage plan that would address the needs of both existing and future urbanization of the watershed. The plan would be an integrated system of channels and detention basins that would intercept floodwaters from within any area of the watershed and safely convey such waters to a suitable outlet. The following issues were considered to be important objectives in the development of such a plan.

1. Develop computerized rainfall/runoff models of the study area in order to provide an efficient tool to use in evaluating the required sizes of channels and detention basins.

2. Address the impact that any drainage system will have on the proposed Outer Loop Highway.

3. Evaluate the potential for interim solutions that could, at a later date, be wholly or partially incorporated into the final drainage plan.

4. Incorporate existing concept drainage plans (to the maximum, feasible extent) into the regional drainage plan.

5. Assume the watershed is in a "future" developed condition that corresponds to the "General Plans" published by Scottsdale and Phoenix.

6. Perform engineering analyses to a level of technical detail that will provide approximations of: 1) typical channel and detention basin sizes; 2) excavation and lining quantities and costs; 3) drop structure requirements; 4) right-of-way requirements; 5) bridge costs; and 6) low-flow culvert costs.

7. Prepare an estimated installation schedule.

8. Develop criteria for distributing costs of the proposed drainage plan among government and private entities.

These objectives have been accomplished as part of the regional drainage and flood control plan presented in this report.

1.3 ORGANIZATION OF STUDY RESULTS

This report presents a substantial amount of technical information. An attempt has been made to organize this information into a format that will allow the reader to access specific topics of interest without having to read through extensive amounts of non-related verbiage. This has been accomplished by sub-dividing the report into specialized subsections devoted exclusively to a single topic or drainage feature.

Separate appendices are provided to present the numerical results of hydraulic analyses, construction quantities, and cost estimates. Summary sheets of key data are included in the tables accompanying the main text of the report. A complete listing of all HEC-1 input and output data is published in a separate volume.

Readers are encouraged to review the Table of Contents when searching for specific information.

2.0 DRAINAGE BASIN CHARACTERISTICS

2.1 DRAINAGE AREA

The project study limits and watershed boundaries are illustrated in Figure 2.1. As stated previously, the contributing watershed encompasses approximately 103 square miles.

The drainage area exhibits considerable variation in topographic features. The eastern part of the watershed includes the McDowell Mountains, which are characterized by very rocky, steep-sloped terrain that serves as the source area for the creation of several alluvial fans. The steep slopes and highly impervious soils of these mountains are conducive to generating rapid and large rates of runoff when subjected to excessive rates of rainfall. Alluvial fans exist along the toe of the mountain slopes and coalesce in a southwesterly direction, forming a piedmont.

The central portion of the watershed, located west and north of the McDowell mountains, might best be defined as a piedmont plain. This region contains both a pediment and alluvial plain. The soils of this region have also been referred to in a recent Soil Survey Report (Soil Conservation Service, 1986) as being characteristic of an alluvial fan terrace. The SCS defines an alluvial fan terrace as an inactive remnant of an old alluvial fan that has been incised by younger and lower alluvial surfaces and is no longer a site of active sediment deposition.

The pediments, alluvial fans, and fan terraces in the watershed are difficult to analyze from both a hydrologic and hydraulic perspective because of the absence of well-defined channels and the transitory flow patterns across these landforms. Although the majority of the pediment surface has fairly well defined swales and washes, the alluvial plain (fan terrace) is characterized by literally hundreds of small, braided washes which have bankfull channel capacities ranging from approximately 25 cubic feet per second (cfs) to 250 cfs. During major floods, such as the 100-year event, the flow characteristics across this surface will most probably exhibit a wide, shallow sheetflow pattern. The channel patterns on an alluvial fan, or fan terrace, are very unpredictable and unstable because of the limited channel capacity and alterations to channel geometry that often accompany the rapid erosion and sediment deposition processes that occur during periods of flooding.

North from Jomax Road to about Dixileta Road, the small washes on the alluvial fan terrace begin to exhibit more definition and corresponding channel capacity that is typical of inactive alluvial fan surfaces. This region represents a transition area between the fan terrace and the pediment.

As one enters the pediment area, the more well-defined channel geometry decreases the potential for sheetflow.

2.2 LAND USE

The majority of the watershed is presently undeveloped. However, there are several pockets of scattered residential development, most notably south of Carefree along Scottsdale Road, and within an approximate 2-mile radius of the Pinnacle Peak/Pima Road intersection. Dense residential development is also underway in Ironwood Village, which is located along the east side of Pima Road, just north of Union Hills Drive.

For the purpose of this study, future land-use conditions were used as the basis for generating peak discharge estimates for the design of all drainage and flood control improvements proposed in this report. Future land use conditions were based on projections published in the following documents.

1. *Land Use Element, General Plan, Scottsdale, Arizona*, January 1989 (maps updated to July 1, 1987).

2. *Tonto Foothills, Background Report, Scottsdale, Arizona*, April 1985, (maps updated to February 18, 1986).

3. *General Plan, Peripheral Areas C and D*, City of Phoenix Planning Department, October 1987.

4. Proposed land-use densities provided by BRW, Inc. for Desert Ridge.

As will be discussed in Section 3.3, the hydrologic impact of future land use conditions was simulated by adjusting the percent of impervious cover for each sub-basin in the HEC-1 models.

2.3 SOIL TYPE AND VEGETATION

Soils information is needed in order to model the infiltration characteristics of the watershed. Such information is generally available from Soil Survey Reports published by the Soil Conservation Service (SCS). The watershed for this project was included in the *Soil Survey of Aguila-Carefree Area, Parts of Maricopa and Pinal Counties, Arizona*, U.S. Department of Agriculture, SCS, 1986.

Using the standard SCS hydrologic soil group classification system, an estimate can be made of the runoff potential of the soils within any given sub-basin of the project watershed. The SCS system is based on four hydrologic soil groups, A through D. Soils in group A have very low runoff potential (that is, high infiltration rate), those in group B have moderately low runoff potential, those in group C have moderately high runoff potential, and those in group D have high runoff potential (that is, very slow infiltration rate).

The composition of the project watershed, in terms of hydrologic soil groups, is presented on Plate 1. The information on this plate is based on the Aguila-Carefree soil survey (SCS, 1986). As can be noted on Plate 1, the watershed is composed of all four hydrologic soil groups (A,B,C, and D).

A review of the SCS soil survey maps indicated that several of the soil map units consisted of two or more major soils. Such combinations, which are referred to as either a soil complex, or soil association, often consist of multiple hydrologic soil groups. When multiple soil groups were encountered, the percentage of each soil (and its corresponding hydrologic soil group) within the soil complex/association was identified from the SCS mapping unit descriptions. These percentage figures were then combined with engineering judgement to select a single hydrologic soil group that was considered to be most representative of a specific soil complex/association. In two cases, the percentage of different hydrologic soil groups was so evenly balanced that the soil complex/association was used in the analysis as a function of two hydrologic soil groups.

Table 2.1 summarizes the soil complex/associations that were evaluated, and lists the hydrologic soil group(s) that were ultimately selected to represent a specific mapping symbol (number) that is published on the SCS soil maps. The application of this data to the selection of SCS curve numbers will be discussed in Section 3.3 of this report.

Due to its predominantly undeveloped nature, the vegetation community in the watershed is typical of the Sonoran Desert and includes such species as mesquite, catclaw, creosote bush, palo verde, ironwood, cacti, and so forth. For those portions of the watershed that have been developed, there has been an attempt to preserve, as much as possible, the natural desert landscape. Preservation of the natural character of the land is in concert with the "General Plans" adopted by Phoenix and Scottsdale.

2.4 EXISTING DRAINAGE IMPROVEMENTS

The low density of development that presently exists in the watershed has not been accompanied by any major flood control or drainage improvements, with the exception of the interceptor channel around the upstream boundary of Ironwood Village. The majority of existing improvements consist of small channels and/or raised levees (berms) alongside some of the roadways or through portions of residential developments. The drainage ditches that exist along the side of some of the major roads (Scottsdale Road, Pima Road, Pinnacle Peak Road, and so forth) may create some minor diversion of runoff during the more frequent floods, such as a 2-year event. However, runoff from severe floods, such as a 50- or 100-year event, will greatly exceed the capacity of these small channels and will continue to flow along its natural drainage path. During field inspections of the watershed, it was also observed most of the roads utilize "dip" sections at their intersection with the natural desert washes. This practice promotes the tendency for floodwater runoff to continue along its natural course rather than being diverted by the roadways.

Large flood control dikes are located along the north side of the CAP, but these are at the downstream limits of the study area and offer no protection to upstream areas.

Remnants of the Old Verde Canal are also located through portions of the watershed north of the CAP alignment. Due to frequent breaks in the canal embankment, no attempt was made to model any floodwater diversions that this man-made feature might create. The impact on existing flooding potential would probably be negligible due to the canal's location near the southern boundary of the study limits. For future watershed conditions, it was assumed the canal remnants would be destroyed as part of any development scenario.

3.0 HYDROLOGIC MODEL (HEC-1)

A computerized rainfall/runoff model was developed for the watershed using the U.S. Army Corps of Engineers Flood Hydrograph Package (HEC-1). HEC-1 uses numerical parameters to describe the amount and temporal distribution of rainfall, the runoff characteristics of the watershed, and the hydraulic properties of overland flow planes and channels that collect and convey the direct runoff to concentration points. The computer output provides a runoff hydrograph at user-selected locations. These hydrographs can be used to design drainage channels, detention/retention basins, or to evaluate the capacity of existing drainage facilities.

The 1985 version of HEC-1 was used for this study. The models presented in this report should only be run with the 1985 program. Any attempt to use the 1988 (or later) version of HEC-1 to execute the input files developed for this project will result in significantly different peak discharge values.

The kinematic wave option was used to determine the hydrologic response of the sub-basin areas and for routing the resulting hydrographs through the tributary channels of the basin. This option was selected because runoff processes can be simulated using measurable geographic features such as overland flow elements and the shape, boundary roughness, length, and slope of channel elements. Unlike unit hydrograph techniques, the kinematic wave approach also provides a non-linear response of runoff characteristics, that is, peak discharge does not necessarily increase linearly with direct runoff when using the kinematic wave methodology.

A network of sub-basins and connecting channels was configured that simulates the natural drainage pattern in the basin. Plate 2 presents an illustration of the existing drainage patterns, sub-basin boundaries, and concentration points used to model future runoff conditions. Plate 3 presents the same type of information for the proposed regional drainage plan.

This section of the report presents a detailed discussion of specific components of the computer model that were created to simulate the rainfall/runoff response of the watershed. Complete listings of the input and output data associated with the HEC-1 models developed for this project are presented in a separate volume that supplements this report.

3.1 OVERVIEW OF HYDROLOGIC MODELING ASSUMPTIONS

The computerized rainfall/runoff models (HEC-1) used for this study are based on the previous HEC-1 models developed for the *General Drainage Plan for North Scottsdale, Arizona* and the *Concept Drainage Study, Paradise Valley Fan Terrace, Part of Peripheral Areas C and D*, Phoenix, Arizona. However, several important revisions have been made to these previous models to allow an investigation of several hydrologic issues that are unique to this current study. Some of these important revisions and modeling assumptions are summarized in the following paragraphs.

The initial modeling effort focused on the development of baseline models that could be used to simulate the runoff response that would be expected to occur if the watershed were to develop in accordance with the land-use classifications and densities forecast in the previously referenced land use plans published by Phoenix and Scottsdale. The baseline models assume that the existing drainage pattern would remain intact, but that the percent of impervious ground cover would increase in response to the forecast urbanization. None of the baseline models include any of the proposed drainage improvements presented in this report.

The primary purpose of these models was to investigate the impact that the proposed regional drainage plan would have on the inflows to the existing CAP dikes. Specifically, the baseline models would provide a benchmark reservoir water surface elevation for each of the four CAP dikes. These benchmark elevations can then be compared to those occurring with the proposed drainage plan in place to see if hazardous conditions are being created in the detention basins as a result of possible, man-made watershed diversions. Section 7.0 of this report presents a complete discussion of impacts to the CAP dikes.

The baseline HEC-1 models utilize the following hydrologic assumptions.

1. SCS Type IIA, 24-hour rainfall distribution

2. Kinematic wave methodology for overland flow and channel routing operations

3. SCS curve number methodology for interception, depression, and infiltration losses, using Antecedent Moisture Condition 2

Revisions to the baseline models used for the previously referenced Phoenix and Scottsdale studies are listed as:

1. Drainage area boundaries for numerous sub-basins in the lower portion of the watershed were adjusted to tie into the tails of the four CAP detention basins. This adjustment was required to allow an accurate assessment of the reservoir performance for each CAP basin. The CAP dikes were not evaluated in the two previously referenced drainage studies for this watershed.

2. Reservoir routing operations were added to the model to simulate the performance of CAP Dikes 1, 2, 3, and 4. The physical characteristics of each detention basin were based on recent (1987) data published by the Bureau of Reclamation. Details of this data are discussed in Section 7.0 of this report.

3. Three new divert operations have been added to reflect probable flow diversions: a) southeast of the Pima Road/Dynamite Road intersection; b) through the Pinnacle Peak Country Club golf course; and c) through the Desert Highlands golf course.

4. Curve number, impervious cover, and channel routing parameters have been revised to reflect all adjustments in sub-basin boundaries.

5. Sub-Basin 49 has been deleted from CP 51 and is now routed directly into Sub-Basin 51.

6. Percentages of impervious cover have been adjusted to reflect Natural Area Open Space (NAOS) requirements for residential land use.

The four baseline HEC-1 models are described as:

1. **File: F3.24I** - This model includes all portions of the watershed that drain to CAP Dikes 1, 2, and 3. Flows from Reata Pass are assumed to be totally diverted to CAP Dike 3. A 25-square mile areal reduction factor is applied to the rainfall values.

2. **File: F4.24I** - Identical to File: F3.24I except that 100 percent of the Reata Pass flows are assumed to be diverted to CAP Dike 4.

3. **File: CAP4.24I** - This model includes all portions of the watershed that drain to CAP Dike 4. All flows from Reata Pass are assumed to be diverted to CAP Dike 4. A 25-square mile areal reduction factor is applied to the rainfall values.

4. **File: CP4R.24I** - Identical to File: CAP4.24I except that 100 percent of the Reata Pass flows are assumed to be diverted to CAP Dike 3.

The baseline models were used as the starting point for all subsequent HEC-1 models that were developed to simulate the performance of the proposed regional drainage plan. Accordingly, the models for the regional plan include the same basic data revisions previously listed for the baseline models. However, development of the HEC-1 models for the regional drainage plan also required a tremendous number of revisions to the baseline models to reflect new sub-basin boundaries, revised curve numbers and impervious cover estimates, channel routing operations, low-flow culverts, and so forth, that were associated with the proposed interceptor channels. A substantial portion of the study effort was devoted to the creation of revised HEC-1 models that could simulate the 35 miles of proposed channelization.

It should be emphasized that the HEC-1 models developed for this study cover a very large watershed area. Accordingly, it was not practical to field inspect each sub-basin and wash in the watershed. However, a substantial number of field investigations were conducted during preparation of the models for the previously referenced Phoenix and Scottsdale drainage studies. These previous field investigations were supplemented with additional site inspections conducted during this current study. As a result, a large data base has been assembled in the form of measured channel cross-sections and watershed photographs. This data base, in conjunction with aerial photographs, quadrangle maps, and engineering judgement, was used to develop typical hydrologic and hydraulic parameters for those sub-basins in the study area that were not site inspected.

Although the level of modeling detail in this study is considered adequate for the design of major interceptor channels, it may not provide sufficient resolution for localized development that may only occupy a portion of one of the delineated sub-basins shown on Plates 2 and 3. It is recommended that

these models be delineated in further detail if they are ever used for the hydrologic analysis of more site-specific commercial or residential development.

Peak discharge calculations have been found to be sensitive to the channel geometry used in the kinematic wave channel routing operations. For the majority of cases, only one main channel has been used in these models to collect runoff from the overland flow planes in each sub-basin. Since this channel geometry is based on a limited number of typical cross-sections that have been applied throughout the watershed, detailed topographic mapping of a proposed development site will undoubtedly provide more accurate information to use for channel routing operations. Depending upon the drainage system design that might accompany such development, collector channels might also be added to the model.

Some of the major modeling components added to the HEC-1 models for the regional drainage plan include:

1. Addition of the Core North drainage system proposed by Carter Associates

2. Addition of the Core South drainage system as presented in a 1987 study prepared by Simons, Li & Associates, Inc.

3. Addition of new sub-basins to account for runoff from the TPC golf course located in the reservoir area of CAP Dike 3

4. Addition of Sub-Basin 2125 to the drainage area captured by CAP Dike 4

5. The use of modified Puls storage routing for channel routing operations involving the 35 miles of interceptor channel proposed in this study

6. The addition of collector channels to the lower sub-basins serviced by the proposed Rawhide Wash channel

Any attempts to further modify the HEC-1 models used for this study should only be done with a thorough understanding of the modeling assumptions, divert routines, and channel routing operations that were used to create the models.

A list of the HEC-1 models developed for the regional plan is discussed as follows.

1. **File: MP3.24I** - This model includes the Pima Road channel, the Rawhide Wash channel, the Scottsdale Road channel, the Deer Valley Road channels, the Core North system, the Core South system, and reservoir routing operations for CAP Dikes 2 and 3. Low-flow culvert diverts are de-activated. All of the Reata Pass flows are diverted to CAP Dike 3. A 25-square mile areal reduction factor is applied to the rainfall values.

2. **File: MP5.24I** - Identical to File: MP3.24I except that the divert operations for the low-flow culverts are activated.

3. **File: MP8.24I** - Identical to File: MP3.24I except that the divert operations for the low-flow culverts are activated and 100 percent of the Reata Pass flows are diverted to CAP Dike 4.

4. **File: MP9.24I** - Identical to File: MP3.24I except that 100 percent of the Reata Pass flows are diverted to CAP Dike 4.

5. **File: PSJ3.24I** - This model includes the Jomax Road channel, the Squaw Peak Parkway channel, the east Pinnacle Peak Road channel, the west Pinnacle Peak Road channel, and a reservoir routing operation for CAP Dike 1. Low-flow culvert diverts are de-activated. One hundred percent of the flow from Sub-Basin 1480 is assumed to be diverted into Sub-Basin 3660. A 25-square mile areal reduction factor is applied to the rainfall values.

6. **File: PSJ5.24I** - Identical to File: PSJ3.24I except that the divert operations for the low-flow culverts are activated.

7. **File: RP4.24I** - This model includes the Thompson Peak Parkway channel and a reservoir routing operation for CAP Dike 4. Low-flow culvert diverts are activated and 100 percent of the Reata Pass flows are diverted to CAP Dike 4. A 25-square mile areal reduction factor is applied to the rainfall values.

8. **File: RP5.24I** - Identical to File: RP4.24I except that the low-flow culvert diverts are de-activated.

9. **File: RP6.24I** - Identical to File: RP4.24I except that the low-flow culvert diverts are de-activated and 100 percent of the Reata Pass flow is diverted to CAP Dike 3.

10. **File: RP7.24I** - Identical to File: RP4.24I except that 100 percent of the Reata Pass flows are diverted to CAP Dike 3.

3.2 DELINEATION OF DRAINAGE SUB-BASINS

As stated previously, the project watershed consists of approximately 103 square miles. Such a large area encompasses a wide range of topographic features, soil groups, and drainage patterns. In order to increase the accuracy of the hydrologic modeling process, it is necessary to sub-divide the watershed into smaller sub-basins of relatively homogeneous hydrologic characteristics. The number and size of sub-basins are also dictated by the number of locations at which hydrologic output data is desired, that is, detention basin outlets, channel locations, and so forth.

In order to meet this criteria, the overall watershed was divided into numerous sub-basins. Plates 2 and 3 illustrate the sub-basin delineations that were used in the models for the existing and post-project conditions, respectively.

A major factor in the delineation of the watershed sub-basins was the alluvial plain (fan terrace) which becomes a prominent geologic feature through the southwestern portion of the watershed. Above (north and northeast) this area, the desert washes are fairly well defined and tend to exhibit a drainage network characterized by a tributary pattern that feeds a dominant, or main, channel within a sub-drainage area.

This is in sharp contrast to the fan terrace (alluvial plain) portion of the watershed which is characterized by a dense network of narrow, shallow, sinuous channels which intermittently mingle and then separate from each other. As a result, there is no dominant channel on the fan terrace portion of the watershed which can be used as a main channel for the concentration of upstream flows.

Because of the discontinuity in drainage pattern, the fan terrace portion of the watershed was modeled as wide strips (with runoff assumed to be uniformly distributed across a portion of each strip), while the more defined channels in the upper reaches of the watershed were modeled in the conventional riverine format where flows are routed via an existing, incised channel (using much narrower widths than on the fan terrace) to a concentration point at the sub-basin outlet.

3.3 INTERCEPTION AND INFILTRATION

Precipitation losses due to interception and infiltration were modeled using the SCS curve number option in HEC-1. Selection of curve numbers was based on information gathered on type of soil cover, vegetation density, land use, and soil moisture conditions. An average curve number was developed for each sub-basin to account for the combined effect of these drainage basin characteristics.

Antecedent Moisture Condition II (AMC II) was used as the basis for all curve number selections. AMC II is defined by SCS as having 0.5 to 1.1 inches (dormant season) or 1.4 to 2.1 inches (growing season) of rainfall during the 5 days preceding the design storm.

A base curve number was developed for each of the four hydrologic soil groups (A,B,C, and D) under the assumption of 15 percent cover density and a "desert brush" vegetation community. Figure 2-15, from the City of Scottsdale *Drainage Report Preparation, Section 2, Design Procedures and Criteria*, was used for the base curve number selection of hydrologic soil groups B, C, and D. Since this reference does not include hydrologic soil group A, a second technical reference was required. Accordingly, Table 2-2d, *Urban Hydrology for Small Watersheds*, SCS Technical Release 55, June 1986 was used to develop a curve number for soil group A.

An important distinction between these two curve number references should be noted. Curve numbers in the City of Scottsdale reference are based on short duration storms (approximately 1 hour), while those in Technical Release 55 (TR-55) are based on long duration storms (approximately 24 hours). Accordingly, the soil group A curve number from TR-55 must be converted to a short duration value in order to be compatible with the B, C, and D soil group values taken from the Scottsdale reference.

The variation in curve number, as a function of storm duration, has been documented by Woodward (*Runoff Curve Numbers for Semiarid Range and Forest Conditions*, ASAE, 1973). Based on an analysis

of actual rainfall/runoff data, Woodward developed a set of curves relating changes in curve number to storm duration. These curves were used to convert the 24-hour, group A soil, curve number in TR-55 to a 1-hour curve number that would be consistent with the Scottsdale data.

As will be discussed in Section 3.6 of this report, a 24-hour storm duration was ultimately selected for use in this study. Accordingly, the 1-hour curve numbers discussed in the preceding paragraphs were converted, using Woodward's curves, to 24-hour duration values. A summary of curve number variation (by storm duration) is presented in Table 3.1.

The 24-hour curve numbers in Table 3.1 were used to develop a weighted curve number for each sub-basin in the watershed. Weighted sub-basin curve numbers were based on a visual estimate of the percentage of each sub-basin area occupied by each of the six following hydrologic soil group (HSG) categories.

1. 100 percent HSG A (CN = 60)
2. 50 percent HSG A + 50 percent HSG B (CN = 67)
3. 100 percent HSG B (CN = 74)
4. 50 percent HSG B + 50 percent HSG C (CN = 78)
5. 100 percent HSG C (CN = 82)
6. 100 percent HSG D (CN = 86)

These six categories reflect adjustments made for the multiple HSG soil complex/associations discussed previously in Section 2.3.

The "area-weighted" curve numbers that were obtained from this procedure were rounded to the nearest whole number for each sub-basin and were considered to be a final baseline curve number representative of natural desert conditions (that is, no development). These final baseline curve numbers were used in all the HEC-1 models created for this study.

Modeling adjustments for increased runoff due to future urbanization were made by increasing the percent of impervious cover input to the LS card for each sub-basin; no changes were made to the sub-basin curve numbers.

The relationship between percent of impervious cover and land-use classification was primarily based on "average percent of impervious area" taken from Table 2-2a, TR-55 (SCS, 1986). The 85 percent and 72 percent impervious area values for commercial/business and industrial districts, respectively, were used without any adjustments. However, the percents of impervious area for residential districts were revised slightly upwards. The revision was made through a visual adjustment to a graphical plot of the residential lot sizes versus the percent of impervious area for each lot size. A smooth, visually fitted curve was then superimposed onto the TR-55 data in order to extend the data to span the entire range of zero to 100 percent impervious area. This graphical plot is illustrated in Figure 3.1. Combining the information from Figure 3.1, Table 2-2a (TR-55), and land-use classifications from the Tonto Foothills Plan, Scottsdale General Plan, and Phoenix General Plan, an area-weighted percent of impervious cover was established for each land-use category used in this study.

The residential percentages of impervious cover were further adjusted to reflect City of Scottsdale requirements for NAOS. Table 3.2 lists the criteria that were used to make adjustments for NAOS requirements.

3.4 OVERLAND FLOW PARAMETERS

Overland flow represents the shallow, sheetflow conditions that occur while runoff is moving from the point of raindrop impact to a channel. HEC-1 simulates this component of flow with input data describing the overland flow length, slope, and roughness.

Except for the lower portions of the alluvial fan terrace, overland flow lengths were measured from a USGS 7.5-minute quadrangle map. These measurements, which utilized considerable engineering judgement, were based on a close examination of the topographic contour lines in order to determine the approximate distance that water would have to travel before reaching an indent in a contour line that could be considered representative of a channel. As many as four measurements were made in some sub-basins to determine an average length that could be considered typical of the entire sub-basin. In a few instances, two overland flow planes were input to the HEC-1 model to describe a sub-basin.

On the lower portions of the alluvial fan terrace, a different approach was taken to measure overland flow lengths. This approach was based on a 1 inch = 1,000 feet, 1984 aerial photograph of the watershed. Each sub-basin was delineated on this photo and lines were drawn perpendicular to the average flow

pattern within each sub-basin. The number of rills or channels intercepted by each line was then made through a visual inspection of the photo. An average width between rills was then obtained by dividing the total length of the line by the number of rills intercepted by the line. The average overland flow length was then computed as one-half the distance between rills, based on the assumption that one-half this distance will drain to one rill while the other half will drain to the adjacent rill. As many as two or three lines were drawn on sub-basins in order to establish an average overland flow length for the entire sub-basin.

This fan terrace analysis was originally performed by Mr. Robert L. Ward, P.E., in 1986 and published as part of a report entitled *Final Hydrology Report, Outer Loop Freeway, North of the CAP Aqueduct*, Simons, Li and Associates, Inc. (SLA), April 1987. This overland flow length data has been adopted from the SLA report, with no changes, for use in this study.

With the exception of the lower portion of the alluvial fan terrace, overland flow slopes were computed from the length and elevation measurements taken from the 1 inch = 2,000 feet USGS quadrangle maps previously referenced for measuring overland flow lengths.

Due to the large contour interval (20 feet) on the quadrangle maps, and the relatively flat topographic relief across the lower fan portion of the watershed, a different technique had to be employed for computing overland flow slopes. Accordingly, seven wide (200 foot to 400 foot) cross-sections were field surveyed on the fan portion of the watershed. Once these cross-sections were plotted, typical cross-slopes to individual rills could be easily computed.

This was done for several cross-sections and an average cross-slope was found to be 0.0213 ft/ft. This value was then used as the average overland flow slope for all sub-basins on the fan terrace area. These cross-sections and fan slope measurements were also based on data from the 1987 SLA report prepared by Mr. Ward.

As with the length measurements, overland flow roughness values require considerable judgement. No values have been published specifically for desert land surfaces. Depths of overland flow may be on the order of 0.25 inches or less. Under such conditions, the texture or surface composition of the ground has a significant impact on the travel time required for overland flow to reach a channel element. Field inspections of the watershed revealed distinct differences in surface soil composition and vegetation

density from the lower to upper portions of the watershed. On the lower fan portion of the drainage area, the ground surface is relatively smooth and flat and is composed of a much finer (smaller grain-size) material than exists in the upper basin. The upper portion of the basin exhibits gravel size surface materials, along with scattered rocks and boulders, and a much more rugged surface topography. Vegetation also appears to be slightly more dense in the upper part of the basin than in the lower part of the basin.

Based on these observations, five categories of surface topography and overland flow roughness were selected as being representative of the watershed. This data is summarized in Table 3.3.

3.5 CHANNEL ROUTING PARAMETERS

Runoff from overland flow planes is concentrated in the numerous dry washes that drain the watershed. Once the water enters these washes, it is routed downstream as open-channel flow. For the kinematic wave option, this routing procedure is a function of: 1) channel length; 2) channel slope; 3) channel shape; and 4) channel roughness. HEC-1 is capable of using as many as three different channel routing segments within a given sub-basin in order to simulate different channel geometries that occur as small collector channels drain to larger collector channels and, ultimately, to a main trunk channel.

Channel lengths and slopes were measured directly from the 1 inch = 2,000 feet USGS quadrangle maps. For the existing drainage pattern (baseline) model, a trapezoid was used to model channel geometry throughout the watershed. The bottom width and side slopes of the trapezoid were based on extensive field measurements, aerial photographs, and engineering judgement (due to the large watershed size, it was not possible to measure every channel).

A modified Puls routing operation, using an 8-point cross-section, was used for all the major interceptor channels proposed as part of the regional drainage plan. Channel geometry was based on normal depth calculations for typical reaches of each channel.

After the initial HEC-1 runs, the peak discharge values at numerous channel concentration points were used, with Manning's Equation, to compute the channel depth, velocity, and Froude Number. If these computed hydraulic parameters did not appear reasonable, the channel bottom widths and/or side-slopes were adjusted in the proper direction.

Due to the wide, sheetflow characteristics anticipated on the lower portions of the alluvial fan terrace, special consideration was given to the selection of the channel geometry for this landform.

This fan terrace is characterized by hundreds of small, braided washes which are 1 foot to 2 feet deep and have average top-widths ranging from 4 feet to 30 feet. The bankfull capacity of these washes ranges from approximately 25 to 250 cfs.

Certain portions of this terrace are subjected to relatively large inflows at the upstream end of the terrace where more well-defined drainage systems are capable of delivering 100-year peak discharges (for a fully urbanized watershed) of approximately 10,000 to 16,000 cfs. Flows of this magnitude are not capable of being conveyed across the fan terrace within the bankfull capacity of the braided washes. Accordingly, large portions of the terrace can be expected to be inundated by shallow sheetflow during these large floods. This type of flow condition can be expected to produce substantial hydrograph attention due to infiltration (transmission) losses and overbank storage effects. This attenuation was artificially simulated in the HEC-1 model by using a very wide channel bottomwidth to route water down the fan terrace. The following steps were used to select suitable channel geometry.

1. Cross-sections were surveyed for several typical washes on the fan terrace. Manning's Equation was then applied to the surveyed channel geometry in order to compute a bankfull discharge for each wash. From this data, an average bankfull capacity was determined for a "typical" wash. This average capacity was 80 cfs.

2. Using aerial photographs, lines were drawn perpendicular to the average flow pattern through each sub-basin. The number of washes intersected by this line was then counted from the photo. As many as two or three lines were drawn on some sub-basins in order to establish an average number of washes for that particular area.

3. The average bankfull capacity from Step 1 was then multiplied by the average number of washes from Step 2 in order to determine the <u>total</u> bankfull capacity of all the washes within a given sub-basin.

4. Once the total channel capacity per sub-basin was known (from Step 3), the HEC-1 model was executed (using estimated channel geometry for the fan terrace) to determine

how much water would be delivered to the upstream end of each sub-basin on the terrace. If this rate of flow was found to be in excess of the total bankfull wash capacity of the sub-basin, then the water was assumed to spread across the sub-basin as wide, shallow sheetflow. The channel geometry for the sub-basin was then adjusted to simulate this condition and the model re-run.

When sheetflow was predicted for a sub-basin, the channel geometry was selected so as to provide realistic depths and velocities of flow across the terrace. For these wide sheetflow areas, realistic depths of flow (within the artificial channel used for the simulation) were considered to be on the order of 1.5 feet or less, while average velocities were assumed to range from 3 to 7 fps, with the higher velocities being encountered in the steeper, upper portions of the terrace where the sheetflow unit discharge was higher. As the water moved down the terrace, it was assumed to spread laterally in a widening fan shape. This resulted in a slight decrease in both depth and velocity of flow in the down-terrace direction. Flow was maintained near critical conditions on the steeper parts of the terrace and was allowed to go subcritical as flatter slopes were encountered on the lower portions of the terrace.

5. For those sub-basins on the terrace that were found to have total wash capacities approximately equal to the incoming flow, a trapezoidal cross-section with a 50-foot bottomwidth was used. Side-slopes for this artificial channel were varied from 50:1 to 200:1, as the water was routed down the terrace. The side-slopes were flattened in order to keep the depth of flow to less than 2 feet (the approximate maximum depth of a typical wash) and the average velocities in the 3 to 5 fps range. Due to the dense braiding pattern on the terrace, and the fact that additional runoff was being intercepted in the down-terrace direction, it was assumed that as the water moved down-slope, it would feed into more and more small washes, thus causing an increase in the total channel perimeter and width of flow. The flattening of channel side-slopes in adjacent downstream sub-basins provides a degree of simulation of this phenomenon, since such channel geometry also produces an increase in perimeter and topwidth.

The preceding discussion of channel routing procedures obviously has no means of physically simulating the increase in infiltration (transmission) losses that will undoubtedly occur as floodwaters transition into

a sheetflow condition; however, the procedure does create hydrograph attenuation. Although the kinematic wave routing option, which was used in this study, is reportedly not capable of simulating hydrograph attenuation due to channel storage effects, the manipulation of channel geometry can artificially induce such attenuation. In this case, the channel geometry was manipulated to produce hydrograph attenuation to account for transmission losses and channel storage effects. The only problem with this technique is the non-availability of measured flow data that could be used to calibrate these adjustments to provide a proper degree of attenuation to correlate with actual flood events on fan terraces.

In the absence of such data, extensive engineering judgement must be used to make such adjustments.

The adjustment of channel geometry dimensions across the fan terrace was found to be extremely influential in the attenuation of peak discharge as the floodwave moved down the terrace. For example, the 100-year peak discharge (existing drainage pattern with future land-use conditions) at SUB 27 is 10,640 cfs, while approximately 4.5 miles downstream at SUB 29, it is only 7,938 cfs. This attenuation was created by increasing the channel bottom width from 1,500 feet in sub-basin 27 to 2,500 feet in sub-basin 28, and to 3,500 feet in sub-basin 29. This sensitivity justifies a careful examination of the channel hydraulics across the fan terrace.

Nearly all the natural channels in the watershed were modeled with a Manning's roughness value of 0.045. In some isolated cases, values of 0.050 and 0.055 were used. These roughness values were based on extensive field observations compared to calibrated "n" values presented in a photo report entitled *Roughness Coefficients for Stream Channels in Arizona* by Aldridge and Garrett, USGS, February 1973.

3.6 RAINFALL PARAMETERS

The hydrologic response of a watershed is dependent upon rainfall characteristics such as depth, duration, and the spatial and temporal distribution of the rainfall event. The rainfall depth is a function of the probability of occurrence and the duration of the event. This probability is expressed as a recurrence interval (50-year, 100-year, and so forth), which is defined as the average interval of time within which the magnitude of an event will be equaled or exceeded once. Mathematically, recurrence interval is defined as the reciprocal of the probability of occurrence.

Rainfall depths for the study area were developed using isopluvial maps and regression equations presented in the *Precipitation - Frequency Atlas of the Western United States*, Volume VIII - Arizona, 1973. Due to the large drainage area size, sufficient variations in rainfall depths were noted on the isopluvial maps to warrant using different rainfall values for different areas of the watershed. Table 3.4 summarizes areally reduced rainfall depths for different portions of the watershed. The rainfall depths in Table 3.4 are based on a 100-year, 24-hour storm.

For the proposed regional channelization system, the total drainage area intercepted by any one of the four independent systems ranged from approximately 18 to 26 square miles. Accordingly, a 25-square mile areal reduction factor was used for the drainage system design.

The rainfall values in Table 3.4 were distributed over a 24-hour duration using the SCS Type IIA rainfall distribution. This distribution is graphically depicted in Figure 3.2.

The selection of the rainfall parameters presented in this section was based on an extensive sensitivity analysis which is discussed in the previously referenced drainage studies prepared for Phoenix and Scottsdale.

3.7 VERIFICATION OF HEC-1 MODEL

In order to establish confidence in the results of computerized hydrology analyses, it is important to develop some procedure to calibrate and/or verify the computer results with measured data. Normally, the preferred approach is a two-step process, that is, calibration followed by verification.

Calibration is the process of changing model coefficients, or other judgmental input parameters, until the model matches (with reasonable accuracy), the results from a measured event. Verification is the process of checking a calibrated model against a data set not used in the calibration process.

As might be expected, the scarcity of measured data makes the calibration/verification process a difficult achievement. However, the absence of measured data can be overcome, to some extent, by employing several independent methodologies to calculate peak discharge values at the same concentration points used in the HEC-1 model. These independent estimates can be compared to the HEC-1 results to see if sufficient differences result that would warrant adjustments to the model input parameters. In the absence

of measured rainfall/runoff data, the verification process can only be used as a guide to ensure that the model is not producing gross inaccuracies in the calculation of peak discharge values.

Four independent calculation procedures were selected to verify the results of the HEC-1 models used for this project. These procedures are listed as:

1. Peak discharge regression equations presented in *Estimation of Magnitude and Frequency of Floods in Pima County, Arizona, With Comparisons of Alternative Methods*, USGS Water Resources Investigations report 84-4142, Table 1, J. H. Eychaner, August 1984.

2. Peak discharge regression equations presented in *Methods for Estimating the Magnitude and Frequency of Floods in Arizona*, USGS Report: ADOT-RS-15(121), R. H. Roeske, September 1978.

3. Graphical peak discharge method presented in *Urban Hydrology for Small Watersheds*, Technical Release 55, Soil Conservation Service, USDA, June 1986.

4. Peak discharge methodology presented in *Hydrology Manual for Engineering Design and Floodplain Management Within Pima County, Arizona*, Pima County Department of Transportation and Flood Control District, September 1979.

Although the regression equations developed under Procedure 1 were based primarily on stream gage data in and around Pima County, their use in the north Phoenix/Scottsdale area is justified on the basis of similar watershed characteristics in both areas.

Procedure 2 utilizes different regression equations for five geographical regions of Arizona. Although the north Phoenix/Scottsdale drainage area physically lies within the delineated boundaries of Region 3, its watershed characteristics are more representative of the Southwest Desert Area defined as Region 2. Accordingly, the Region 2 regression equations were used for this study. However, as a matter of technical interest, both Region 2 and Region 3 calculations are shown in Tables 3.5 and 3.6.

Procedure 3 (TR-55) is based on an SCS Type II rainfall distribution and uses a time of concentration that evaluates sheetflow, shallow concentrated flow, and open channel flow. Where applicable, the same

overland flow and channel routing parameters that were used in the HEC-1 model were used in this procedure. The same SCS curve numbers were also used in the TR-55 procedure as were used in the HEC-1 model.

Procedure 4 is a semi-empirical, peak discharge equation that acknowledges such watershed characteristics as watercourse length, mean slope, basin roughness, length to center of gravity, drainage area size, and infiltration rate (SCS curve number). Although this procedure was developed in Pima County, it is based on physical watershed characteristics that allow it to be used in any semi-arid environment. It should be noted, however, that the procedure is limited to individual sub-basins whose times of concentration are less than 3 hours. Since this procedure is based on short duration storms, all SCS curve numbers used for this procedure were taken from the curve number figure in the Pima County Hydrology Manual.

As discussed previously, the HEC-1 models used for this study were based on the HEC-1 models developed for the *North Scottsdale General Drainage Plan and the Concept Drainage Study for the Paradise Valley Fan Terrace*. Model verification analyses were completed for each of these two previous studies. Since this current study uses the same basic modeling data as for the previous studies, there is no reason to conduct additional verification studies as part of this report. Instead, a complete recapitulation of the two previous verification analyses (with some minor revisions and additional text) will be presented in the following subsections.

It should be noted that the verification sites used for both studies are slightly outside the actual drainage area boundaries of the regional flood control plan developed for this study. However, the sites are part of the general region that was investigated in this study, and exhibit similar landform characteristics. Since the same modeling assumptions and logic were used throughout this region, the conclusions reached from the verification sites should be applicable for the entire watershed.

As a matter of technical interest, it should be noted that the verification analyses were performed for the watershed in its existing state of urbanization, not forecast future urban conditions.

3.7.1 North Phoenix Area

This test site consists of six sub-basins ranging in size from 0.04 square miles to 5.87 square miles. The concentration points used for the peak discharge calculations are located both east and west of Cave Creek

Road, between Jomax Road and the Carefree Highway. Some of the contributing drainage areas extend several miles east of Scottsdale Road.

Table 3.5 presents a summary of the independent peak discharge calculations that were performed for each of these six sub-basins. For comparison purposes, the peak discharge values from the HEC-1 model (using the 24-hour, SCS Type IIA rainfall distribution) are also listed in this table. Figures 3.3 through 3.8 graphically illustrate the data presented in Table 3.5.

When interpreting the results in Figures 3.3 through 3.8, consideration must be given to the watershed characteristics. This is especially important when judging the results of the two regional regression equation methods. Peak discharge regression equations reflect an average response from all watersheds used in the regression data base. Accordingly, when applied to small, homogeneous sub-basins, such as those used in this verification analysis, they may significantly over or under predict discharges if the test sub-basins have extremely steep or flat slopes, or have infiltration characteristics that are extremely pervious or impervious, or exhibit sheetflow characteristics.

Even though the regression equations are regionalized, they do not have the capability to make good predictions for small basins that exhibit hydrologic characteristics towards the extreme ends of the spectrum.

Brief comments are provided for each sub-basin subjected to the verification process:

- **CP 3020 (Figure 3.3)** - This single sub-basin is composed of 25 percent HSG B and 75 percent HSG C, which weights it toward the more impervious side of the soil groups. The predicted HEC-1 discharge of 433 cfs is positioned about midway between the extreme values predicted by TR-55 and the Roeske regression equation. There is excellent correlation between HEC-1 and the Pima County method and relatively good correlation between HEC-1 and the Eychaner regression equation.

- **CP 3135 (Figure 3.4)** - This is a very small, single sub-basin that is composed of 50 percent HSG B and 50 percent HSG C, placing it at the midpoint of the hydrologic soil group classification. Accordingly, the sub-basin should not exhibit extreme infiltration characteristics towards either end of the spectrum. Although the independent peak

discharge calculations show considerable variation, HEC-1 is located comfortably within the range of estimates, and shows good agreement with the Pima County method and TR-55. It should be noted that the small size of the basin (0.04 square miles) is below the minimum drainage area size of 0.09 square miles recommended for use with the Roeske regression equation.

- **CP 3150 (Figure 3.5)** - This is a very long and narrow drainage basin and actually consists of three separate sub-basins which are linked together and routed to the outlet of Sub-Basin 3150. The area-weighted curve number is 79.8, which places the basin just slightly greater than a curve number of 78, which represents the midpoint of HSG A, B, C, and D for desert conditions. The shape factor for this basin was computed as 25.67, which exceeds the maximum recommended shape factor of 20.6 associated with the Eychaner regression equation. The fact that the Roseke equation provides a substantially higher peak discharge than HEC-1 could be easily explained by the failure of the Roeske equation to account for the basin shape, that is, long, narrow basins produce low peak discharges at the basin outlet. However, the Eychaner equation, which does address the basin shape factor, also predicts a much higher peak discharge than HEC-1. This anomaly is difficult to explain other than to note that the basin shape factor exceeds the envelope limits for the Eychaner equation. Fortunately, there is much better correlation between HEC-1 and the Pima County method and TR-55. Again, HEC-1 is positioned comfortably between the extreme values predicted by the independent calculations, and, as a result, is considered to be a reasonable simulation of the basin's runoff response.

- **CP 3160 (Figure 3.6)** - This is a single sub-basin consisting of 25 percent HSG B and 75 percent HSG C. Accordingly, the basin is biased towards more impervious soil characteristics. As with the previous sub-basins, HEC-1 continues to provide peak discharge estimates that are near the middle of the range generated by the independent calculations. For this sub-basin, HEC-1 shows excellent comparison with the Pima County Method and good correlation with the Eychaner regression equation.

- **CP 3400 (Figure 3.7)** - This concentration point drains two linked sub-basins which create a long, narrow basin geometry. The entire basin is composed predominantly of HSG B, as evidenced by an area-weighted curve number of 74.5. The relatively pervious

soils associated with this basin are suspected as the primary reason for the poor correlation between HEC-1 and the two non-curve number dependent regression equations. The Pima County method and TR-55, both curve number dependent, provide good correlation with HEC-1. The basin shape factor (19.63) is undoubtedly responsible for a large portion of the difference between the Roeske equation results and HEC-1 (that is, Roeske does not address shape factor). However, the Eychaner equation, which does include shape factor, is still producing a surprisingly higher estimate than HEC-1, even when considering the pervious soil conditions. This same trend was noticed for Concentration Point 3150.

- **CP 3490 (Figure 3.8)** - This concentration point drains an extremely long, narrow basin. The basin is over 14 miles long and extends to the northeast corner of the watershed near Wildcat Hill. The drainage area has a shape factor of 34.37, which is well beyond the envelope of the Eychaner equation. The entire basin encompasses five separate sub-basins and multiple hydrologic soil groups. The area-weighted curve number is 78.65, which places the overall basin at about the midpoint of the hydrologic soil groups. With the exception of the Pima County method, the independent peak discharge calculations show excellent correlation with the HEC-1 results. The low discharge predicted by the Pima County method is attributed to the long time of concentration (T_c) produced by this method. The computed T_c of 4.66 hours exceeds the 3-hour limit associated with this method. Since the Pima County procedure computes rainfall intensity (and the runoff supply rate) as a function of time of concentration, the large T_c value is creating very low rainfall intensities, which in turn leads to low peak discharge estimates. Accordingly, the Pima County method can be dismissed as non-applicable to this odd shaped basin.

In summary, the independent peak discharge calculations indicate that the HEC-1 model is providing reasonable results for these six test sites. In those instances where other methods are providing significantly different results than HEC-1, there is usually a logical explanation related to physical watershed characteristics.

As a final step, in the verification process, 17 additional sub-basins in the study area were selected for comparison to a 100-year peak discharge envelope curve (Boughton, Renard, Stone, 1987). The purpose

of this step was to determine if the model was producing excessively high peak discharges beyond the limits of the six sub-basins used for the independent verification calculations.

The results of this analysis are presented in Figures 3.9 and 3.10. Figure 3.9 utilizes sub-basins that lie within the northern half of the watershed where sheetflow is less prone to occur. Figure 3.10 uses three sub-basins on the alluvial plain (fan terrace) that is located in the sheetflow-prone area south of Dynamite Road.

In addition to the 100-year envelope curve, both figures include the peak discharge regression line associated with the USGS 100-year, primary regression equation developed by Eychaner. The peak discharge data points from the HEC-1 model are also shown on each figure.

When interpreting the results of these figures, the Boughton Q_{100} envelope curve should be considered as a reasonable upper limit for a 100-year peak discharge, although this does not mean that it is impossible for 100-year events to exceed this line. There may indeed be watersheds with physical characteristics (steep slopes, impervious surface, and so forth) that could generate peaks beyond the envelope curve. However, if this occurs, one should carefully examine the watershed features to see if there is a rational reason for this to happen.

The USGS Eychaner curve represents a 100-year event (not an envelope curve). Accordingly, ideal correlation would occur if the HEC-1 data points were found to plot directly on the Eychaner curve. This will rarely (if ever) happen because of different watershed characteristics in the study area versus those used in the development of the regression equation.

A review of the information presented in Figure 3.9 supports the previous conclusion that the HEC-1 model is producing very realistic results. All HEC-1 data points plot comfortably <u>below</u> the Q_{100} envelope curve, and are scattered around the USGS Eychaner curve. Nine of the data points that plot the farthest <u>above</u> the USGS line are from sub-basins that have very impervious soils (composed of 70 percent to 100 percent HSG C and D). Sub-basins with such impervious soils would be expected to produce higher than average runoff rates. This is exactly what Figure 3.9 illustrates. At the other end of the spectrum, the two data points that plot significantly below the USGS line are composed nearly 100 percent HSG B soils, which are relatively pervious, thus non-conducive to generating large runoff rates.

For those readers who may feel that the physical characteristics of the sub-basins do not totally account for the majority of the HEC-1 data points plotting above the USGS regression line in Figure 3.9, it should be recalled from Section 3.3 that AMC II was used for all curve number selections in this study. AMC II is admittedly atypical for this desert region and will undoubtedly generate peak discharge data that will normally be greater than that which might occur under a more natural antecedent moisture condition, that is, AMC I. However, the purpose of this study is to produce design recommendations for drainage system concepts. Under such conditions, the use of AMC II is highly recommended and supported by SCS as a prudent design assumption.

A review of Figure 3.10 confirms the expected result that the sheetflow prone alluvial plain (fan terrace) should produce peak discharge values that are well below the USGS regression line, which is more representative of incised riverine channel conditions. These low peak discharge values are due to the pervious soils in this area (100 percent HSG B) and the hydrograph attenuation that accompanies flow through wide, shallow floodplains.

The combination of several independent peak discharge calculations and the comparison of unit peak discharge values in Figures 3.9 and 3.10 provides sound technical justification for concluding that the HEC-1 model developed for this study is producing very reasonable results. Since the same modeling logic was used in generating the input data for the remaining sub-basins in the watershed, it can be justifiably concluded that the complete watershed model is providing output data that is reasonably representative of the rainfall/runoff response from major storms over the project study area.

3.7.2 North Scottsdale Area

This verification site consists of six sub-basins from an approximately 7 square mile drainage area located immediately south of Thompson Peak (McDowell Mountains). This area was chosen as being typical of three different landform classes that compose the majority of the study area. These landforms, and their associated verification sub-basins, are listed as:

1. Mountains - Sub-basin 2300
2. Mountain Foothills - Sub-basins 2255, 2270
3. Alluvial Fan/Fan Terrace - Sub-basins 2240, 2260, and 2290

Table 3.6 presents a summary of the independent peak discharge calculations that were performed for each of these six sub-basins. For comparison purposes, the peak discharge values from the HEC-1 model (using the 24-hour, SCS Type IIA rainfall distribution) are also listed in this table.

Figures 3.11 through 3.16 graphically illustrate the data presented in Table 3.6. The previous comments cited in Section 3.7.1, regarding interpretation of verification results, also apply to the data in Table 3.6. The following discussion is provided to assist in the interpretation of the results listed in Table 3.6.

- **CP 2240 (Figure 3.11)** - This sub-basin is located on an alluvial fan, or fan terrace area, and has a curve number of 74, which reflects 100 percent Hydrologic Soil Group B. The HEC-1 peak discharge of 664 cfs shows excellent correlation with the Pima County Peak method, and fair to good correlation with the TR-55 calculation. Both of these methods are curve number dependent and are, therefore, capable of reflecting the above average permeability of the soil. Both regression equations predict slightly higher peaks than HEC-1. This is attributed to the inability of the regression equations to account for the pervious nature of the soil in this sub-basin. Considering this factor, the overall correlation among the five independent procedures is good.

- **CP 2255 (Figure 3.12)** - Nearly 40 percent of this sub-basin is composed of mountain foothills with a curve number of 86. However, the remaining 60 percent of the basin is more typical of an alluvial fan and has a much lower curve number of 74. The area weighted curve number for the entire sub-basin is 78.8, which is almost exactly at the midpoint of the hydrologic soil groups, that is, halfway between a B and C soil.

  Accordingly, there is no extreme soil infiltration characteristic to bias the curve number dependent methods away from the non-curve number dependent regression equations. As a result, there is relatively good correlation among all five methods. The HEC-1 peak discharge is positioned comfortably between the extreme values predicted by TR-55 and the Roeske regression equation.

- **CP 2260 (Figure 3.13)** - This is a very long, narrow sub-basin that is located on the alluvial fan/fan terrace portion of the test area. The basin has only slightly more previous soils than average, as evidenced by a curve number 76. The verification

analysis indicates excellent correlation among all methods, except for the Roeske regression equation. This single deviation is undoubtedly due to the fact that the Roeske equation is solely a function of drainage area size and is not capable of reflecting the basin's shape factor. The other four methods incorporate variables to reflect this characteristic. Since long narrow basins generally produce lower than average peak discharges, this characteristic easily explains the poor prediction produced by the Roeske equation.

- **CP 2270 (Figure 3.14)** - This sub-basin is very typical of the mountain foothills area. The area-weighted curve number of 78.8 represents a very average soil infiltration characteristic, although about 40 percent of the basin is composed of steep hillslopes and highly impervious soils (CN = 86). The basin also has a very round shape, as evidenced by a computed shape factor 2.02. This factor, along with a substantial portion of steep-sloped, highly impervious soils, should lead to higher than average peak discharge values. The verification results indicate very good agreement between HEC-1, the Eychanger regression equation, and the Pima County Peak method. Since the area-weighted curve number of 78.8 is very average, the Eychanger equation is not being biased by its inability to account for soil characteristics. The steep slopes and basin shape factor are accounted for in all three of these procedures.

  The low discharge predicted by the Roeske equation is attributed to its failure to acknowledge steep slopes and shape factor. The low discharge predicted by TR-55 is difficult to explain since the time of concentration and curve number used for this procedure is based on the HEC-1 input data. A possible explanation might be that the shape of the SCS dimensionless unit hydrograph does not accurately reflect the runoff response of a steep-sloped environment.

  Overall, the excellent correlation provided by two of the four independent methods indicates the HEC-1 results are very reasonable.

- **CP 2290 (Figure 3.15)** - This basin is typical of the alluvial fan environment, although in the HEC-1 routing schematic it is not connected to an upstream source area. The basin has a curve number of 82, which is representative of a group C soil. The predicted

HEC-1 value of 1,130 cfs is about 24 percent higher than the two closest independent predictions. The higher than average impervious nature of soils would possibly explain the increase in the HEC-1 value over the non-curve number dependent regressions, but not over the values obtained with TR-55 and the Pima County method. The HEC-1 model is probably producing a slightly high peak from the basin because of the narrow 10 foot bottomwidth that was used for the main channel routing operation. A sensitivity analysis was performed in which the channel bottomwidth was alternately changed to 25 feet, and then to 100 feet. These changes decreased the peak discharge to 1,080 cfs and 843 cfs, respectively, which is more in concert with the four independent analyses. The Boughton 100-year envelope for this basin produces a peak discharge of 1559 cfs.

- **CP 2300 (Figure 3.16)** - This is a very steep-sloped, highly impervious mountain sub-basin that joins Thompson Peak. The basin has a curve number of 84 (very near to 86, which represents 100 percent soil group D) and 48 degree slopes on the overland flow planes. The main channel slope is nearly 6 percent. The high curve number is undoubtedly responsible for the large variation between HEC-1 and the two regression equations. The nearly 21 percent variation from the Pima County method may be due to the ability of the HEC-1 model to better simulate the slope changes that occur in this sub-basin (that is, three separate channel routing operations were used in the HEC-1 model to simulate an intricate system of collector channels and a main channel). The Pima County method only produced a mean basin slope of 7.4 percent, while the HEC-1 model used 21.5 percent and 14.5 percent slopes for the two collector channels.

  As discussed for CP 2270, the large deviation from the TR-55 method may be related to non-applicability of the SCS unit hydrograph shape to steep mountain environments.

  The large variation in discharge among the five procedures makes an accurate assessment somewhat difficult for this basin. However, the basin characteristics would suggest a high peak discharge, when compared to basins of similar size in other environments. Both HEC-1 and the Pima County method support this conclusion. As a matter of technical interest, the Boughton 100-year envelope predicts a peak discharge of 3,411 cfs for this sub-basin. However, the severe topographic and impervious soils of the basin provide justification for exceeding the envelope in this case.

It is interesting to note that application of the TR-55 procedure to the mountainous terrain did not provide better agreement with the HEC-1 model results, since essentially identical input data was used in both methodologies. The resulting differences in peak discharge can only be attributed to different data processing algorithms in HEC-1 versus TR-55.

As with the North Phoenix verification process, 12 additional sub-basins in the North Scottsdale area were also compared to the Boughton 100-year envelope curve. Figures 3.17 and 3.18 illustrate this comparison for non-sheetflow and sheetflow areas, respectively. The results are very similar to those previously discussed for the Phoenix test basins.

Overall, the results of this analysis lend confidence in the performance of the HEC-1 model when using the input data logic that was used to describe the physical drainage basin characteristics of the six test sub-basins. Since this same logic was used in generating the input data for the remaining sub-basins in the watershed, it can be concluded that all the watershed models are providing output data that is reasonably representative of the true rainfall/runoff response of the project study area when subjected to extreme storm events that are recommended for the design of major flood control structures.

4.0 HYDROLOGIC AND HYDRAULIC ANALYSIS OF PROPOSED FLOOD CONTROL CHANNELS

Channelization is the primary flood control alternative that was pursued under this study. However, portions of the channelization system are very dependent upon three detention basins being constructed within the boundaries of Core North. These basins will be discussed in more detail in Section 4.2.11 of this report.

Approximately 35 linear miles of major channelization are being proposed as part of this study. Due to the conceptual nature of this project, as well as the large magnitude of channel length, a final engineering analyses for each reach of channel is beyond the scope of work for this study. However, the approximate size and cost of each channel segment was evaluated through application of standard, simplifying assumptions. The majority of these assumptions were adaptable to programmable spreadsheets that provided a very efficient mechanism for investigating channel performance and cost.

The following subsections describe the standard assumptions used in the channel analysis.

4.1 CHANNEL DESIGN ASSUMPTIONS

4.1.1 Design Discharge

The design discharges used for the channel analyses were taken from the post-project HEC-1 models discussed in Section 3 of this report. These discharges are based on the 100-year, 24-hour storm, with forecast future land-use conditions in place. The HEC-1 models are considered suitable for final design of the proposed regional channelization system. However, during final design, there may be some minor alteration of channel alignments that may require slight revisions to the HEC-1 models in order to obtain a discharge at a different channel concentration point.

As discussed in Section 3, it is recommended that the HEC-1 models developed for this study be utilized as a baseline condition for evaluating any future changes to the project watershed. Such an approach would allow an "apples to apples" comparison of any potential impacts that such changes may have on the design capacity of the proposed channel system.

The spreadsheets contained in Appendix A list the design discharges for each channel. The variation in discharge, along each channel, is referenced to channel stations. Approximate channel stationing is shown on the existing ground profiles for each channel alignment (Plates 5 through 9).

4.1.2 Geometry

All channel analyses presented in this report utilize a 4-point trapezoidal cross-section, with the singular exception of the Reata Pass channel which uses a benched, 8-point cross-section with a large low-flow channel. At the request of the City of Scottsdale, a small, stabilized low-flow channel (1 foot deep, 12 foot bottomwidth, soil-cement banks) has also been included in the cost estimates for both the 4-point and 8-point sections. However, the hydraulic influence of this small low-flow channel is not reflected in the hydraulic performance characteristics, (depth, velocity, Froude No., and so forth) of either the 4-point or 8-point channel sections. The larger, low-flow channel in the 8-point section for Reata Pass is reflected in the hydraulic calculations. Figures 4.1 and 4.2 illustrate the geometry of the 4-point and 8-point cross-sections, respectively.

The 4-point section utilizes a constant 4-foot flow depth with 2H:1V side-slopes. The channel bottomwidth is varied as a function of discharge, in order to maintain a 4-foot flow depth. Bottomwidths were computed with Manning's Equation, using an assumption of normal depth.

A Manning's "n" value of 0.045 was used to develop the hydraulic parameters listed in Appendix A. This roughness value provides an allowance for some vegetation within the channel boundaries. However, as will be discussed in Section 6.0, an "n" value as high as 0.045 may not be feasible, in some channels, because of the need to maintain high sediment transport rates. The use of n = 0.045 should generate some degree of conservatism in quantifying channel excavation costs and right-of-way requirements. At this conceptual level of study, such conservatism is prudent. Accordingly, this higher "n" value was used for all cost estimates, even though it may ultimately be decreased to address sediment transport issues.

The 8-point cross-section used for the Reata Pass channel utilizes a fixed cross-section geometry that consists of a 3-foot deep low-flow channel with a constant 75-foot bottomwidth. The upper benches are each 100-feet wide. Side-slopes are 5H:1V for the entire cross-section. Flow depth was allowed to vary in this channel as a function of slope and discharge. A Manning's roughness value of 0.045 was used for the low-flow channel, while n = 0.055 was used for the upper benches.

4.1.3 Channel Slope

The only continuous topographic mapping available for the study area was USGS 7.5-minute quadrangle maps (1 inch = 2,000 feet). The vertical contour interval on these maps varied from 10 to 20 feet. Accordingly, the development of channel profiles is very approximate, but adequate for developing rough channel dimensions and cost estimates.

Due to the length (35 miles) of channelization involved in this study, and the crude topography available, it was not considered practical to establish a final invert profile for each channel segment. For the purpose of developing estimates of channel widths and excavation costs, the channel slope was assumed to parallel the existing ground slope along each channel alignment. This assumption forms the basis for all the hydraulic calculations, construction quantities, and cost estimates presented in Appendix A.

The sediment transport analysis presented in Section 6.2 attempts to make a more realistic analysis of potential channel slopes that may be required for final design. However, no attempt was made to transpose these equilibrium slopes onto existing ground profiles so that average end-area cross-sectioning could be done to generate a more refined estimate of excavation costs and channel widths.

4.1.4 Freeboard

Channel freeboard is defined as the additional channel depth extending from the design water surface elevation to the top of the channel bank. Freeboard provides a safety factor for variations in the assumed hydrologic and hydraulic design assumptions, as well as for containment of wave action associated with flowing water.

For the purpose of this study, 2 feet of freeboard was included in the cost estimates for all channel analyses. During final design, there may be localized areas of certain channels that may warrant additional freeboard above 2 feet. However, for the purpose of this study, a 2-foot freeboard allowance is considered satisfactory for below-ground channels with no above-ground levee embankments.

4.1.5 Bank-Lining

The high channel velocities generated during passage of the 100-year event are capable of causing substantial erosion to the channel banks. Based on existing ground slopes and "n" values of 0.045, these velocities may exceed 15 fps. Accordingly, it will be imperative that the channel banks be constructed of an erosion resistant material, such as soil-cement.

Failure to stabilize the channel banks will not only lead to bank erosion and channel widening, but will also be conducive to generating lateral headcuts, as intercepted overland flows cascade over the banks of the channels. For the purpose of this study, an 8-foot wide soil-cement lining is included along each side of the channel. As illustrated in Figure 4.1, this lining extends from the top of the channel bank to the estimated scour depth for the 4-point cross-section.

The 8-point cross-section utilizes a similar design for the low-flow channel, but only includes bank-lining above the low-flow bench for the upstream side of the channel (see Figure 4.2). It is assumed that the heavily vegetated bench sections in the 8-point cross-section will keep velocities in the channel within a non-erosive range. Accordingly, bank protection above the bench should not be needed for channel flows. However, the interception of sheetflow and natural washes, along the upstream bank of the channel, will generate rilling and headcutting as these flows cascade down the channel bank.

To prevent this kind of erosion, some type of bank protection should be provided along this upstream bankline. For the purpose of generating a cost estimate for this study, it was assumed that 50 percent of the length of the Reata Pass channel would include a soil-cement lining along on side of the upper channel bank. This bank lining was assumed to extend to the top of the bank to 3 feet below the bench elevation. For cost estimating purposes, this dimension was assumed to be 45 feet, measured along a 5:1 sideslope. This corresponds to a vertical distance of 9 feet.

The actual design of this upper bench slope protection should be closely evaluated during final design. Various combinations of sideslope and erosion-resistant materials should be considered in order to find an economical and aesthetically pleasing solution to this type of erosion problem. It may be found that such protection may only be required in the vicinity of actual washes that are intercepted by the channel.

The soil-cement lining has been wrapped over the top of the bank for both cross-sections. This was done in an effort to reduce the possibility that local drainage, running along the top of the banks, would not undercut the top edge of the bank lining. Figure 4.3 illustrates two concepts that could be employed for this purpose. For purposes of generating a cost estimate for this study, a 2-foot deep, soil cement cut-off wall was assumed to be in-place along both channel banks. It may be possible during final design to consider elimination of this wrap-around feature along the down-slope side of a channel. This should only be done if assurances can be provided that water will not be flowing along this edge of the channel bank.

Although all lining costs in this report are based on an 8-foot wide thickness of soil cement, some projects have reportedly been constructed on 2H:1V side-slopes with a horizontal, 4-foot wide thickness of soil-cement. In an effort to reduce construction costs, the final design study for this project should investigate the feasibility of using a narrower thickness of soil-cement. The 8-foot dimension, which has been a quasi-standard for major channel construction, is based on equipment width limitations. This thickness of soil-cement is known to have a high degree of durability, as evidenced by its successful performance during the 1983 floods in Pima County. Since any decrease in durability is not known for thicknesses less than 8 feet, physical model testing might be a cost-effective option to pursue in order to develop reliable design data for lesser soil cement dimensions.

4.1.6 Scour

The design of a bank protection system must consider the potential for scour of the channel bed. Failure to do so could lead to the toe of the bank protection material being undercut by scour processes that will be induced by flowing water. Should this situation occur, the bank lining material may collapse into the scour hole, thus exposing the bank to erosive velocities and possible lateral movement.

Due to the concept level nature of this study, it was not practical to conduct a detailed scour analysis of 35 linear miles of channel. However, a general assessment of scour potential was conducted in order to establish a typical toedown dimension that could be used in the channel cost estimate. This analysis led to the selections of 6 feet for 4-point sections and 8.5 feet for the 8-point section at Reata Pass. A detailed discussion of scour processes, and the calculation of the 6 and 8.5 foot toedown values, is presented in Section 6.1 of this report.

4.1.7 Low-Flow Culverts

The majority of the channels evaluated for this study will intercept the southwesterly flow of natural desert washes. In order to preserve the natural vegetation community along that portion of those washes that are downstream from the man-made interceptor channels, it is recommended that the man-made channels include low-flow outlets (culverts) that will continue to feed water to the downstream remnants of these natural washes. Figure 4.4 illustrates the low-flow culvert concept.

Due to the high density of natural washes that exist throughout the project area, the number of low-flow outlets could be quite high. As a result, the low-flow culverts could have a significant impact on the required main channel capacity. The actual number of low-flow outlets will have to be determined during

final design. This will require an analysis of aerial photographs and field inspections to determine which washes will receive low-flow culvert releases.

For the purpose of this study, 1 inch = 400 feet aerial photographs were reviewed in order to determine the average spacing of natural washes through different regions of the watershed. These average distances between washes were then applied to specific channel reaches in order to estimate a probable number of required low-flow outlets for each channel system.

Discharge estimates through the low-flow culverts were based on a single 36-inch RCP at each natural wash intersection with the man-made channel. Release rates were computed as 60 percent of the total culvert discharge occurring under inlet control. The 60 percent factor was applied to account for debris blockage and flow reduction due to the momentum of the channel flow being parallel to the channel alignment, rather than parallel with alignment of the intercepted wash.

The rate of water lost through each culvert during the peak discharge of the 100-year event was based on a 4-foot headwater depth, which corresponds to the maximum assumed channel flow depth of 4-feet (see Section 4.1.2). The culvert discharge at a 4-foot headwater depth was then multiplied by the total number of culverts allocated to a specific reach of channel in order to compute the total low-flow releases for that reach of channel.

A portion of the Pima Road channel will be used to clarify this procedure. Approximately 1,800 linear feet of the Pima Road channel extends through Sub-Basin 30R. Based on an analysis of aerial photographs, the average spacing between washes for this sub-basin is 165 feet, measured in a north-south direction along Pima Road. This measurement accounts for the skew angle formed by Pima Road with the natural washes. Accordingly, 11 low-flow culverts (1,800 ÷ 165 = 10.9, rounded to 11) will be required for this segment of the Pima Road channel. With a computed discharge of 29.4 cfs/culvert (at a flow depth of 4 feet), the total low-flow culvert releases for this reach of channel will be 323 cfs (4 x 29.4 = 323.4, rounded to 323).

Low-flow culvert releases are simulated in the HEC-1 model with the use of divert operations. For this example, a divert operation was placed at the downstream boundary of Sub-Basin 30R. Low-flow culvert releases are then removed from the Pima Road channel at this location. The flow remaining in the channel is then routed downstream where the sequence of steps is repeated, as described above. The

flows diverted from the channel are subsequently retrieved for routing through the adjacent downstream sub-basin, which, in this example, would be Sub-Basin 31B.

The diversion ratios, that are used to remove flows from the main channels, are difficult to accurately define. Should the concept of low-flow culverts be adopted for this project, the final channel design should analyze this concept in much more detail than has been allotted to it as part of this study. The divert ratios used in this present analysis assume the computed low-flow culvert discharge will occur at the instant of peak discharge in the channel. The divert ratio established at that instant is also used for all channel discharges equal to or less than that peak. One fallacy with the approach used in this report is that all peak channel discharges used for the divert ratios were taken from results of the HEC-1 models that did not include low-flow culvert releases. This introduces an error that causes downstream divert ratios to be based on peak channel discharges that are higher than what will actually occur. This may cause divert ratios to discharge less water from the channel (through the culverts) than would actually occur.

Perhaps a more accurate, but much more time consuming, approach would be to incrementally insert the low-flow divert operations into the HEC-1 model, run the model, then use the results of that previous run to define the peak channel discharge through the next downstream channel segment. This new peak discharge would be used to set the divert ratio for this next channel segment. The model would then be run again to determine the peak discharge through the next downstream channel segment. Another divert ratio would be established and the procedure repeated until the downstream end of the channel was reached.

Unfortunately, another variable that would have to be addressed during this iteration sequence would be the channel geometry and depth of flow associated with these incremental reduced channel flows. Flow depth will become an important variable because of its effect on the headwater depth that determines the low-flow culvert discharge.

Certainly, the entire process could become very complex. As a simplifying, conservative approach one might ignore the low-flow culvert releases, and design the main channel on a worst-case scenario that assumes all the low-flow culverts are plugged with sediment or debris during the design storm. For those channels that might <u>receive</u> water from the low-flow culverts, the culverts could be assumed to flow completely unobstructed, in order to develop a maximum inflow scenario for these downstream channels.

Another very important factor in the design of the low-flow culverts will be the establishment of a controlled channel invert elevation that can be used as a reliable reference for generating headwater depths at the culvert inlet. At the present time, the channels are proposed to have earth bottoms. These channel beds will be prone to vertical movement as degradation, scour, and sediment deposition processes occur. Such vertical bed movement may cause changes in the channel flow depth, which in turn influences the headwater depths that control the depth/discharge ratings for the culverts. Although grade control structures may be included at certain locations in the final channel design, it may be impractical to place such controls at every low-flow culvert entrance. In the absence of such controls, the discharge capacity of the low-flow culverts may be severely compromised. The existence of this problem may be another good reason to design the channels on a worst-case scenario that the low-flow culverts are inoperative during the design storm.

Table 4.1 summarizes the calculation of low-flow culvert costs for the proposed regional drainage plan. These calculations are based on an assumption that the average slope and depth of intercepted washes are 0.0125 ft/ft. and 2 feet, respectively. The intercepting channel is assumed to be 6 feet deep. Accordingly, the differential channel depth (column 4 in Table 4.1 is 6-2 = 4 feet. The low-flow culvert is assumed to be placed on a slope of 0.0050 ft/ft.

The geometric relationship between these assumed depths and slopes can be used to compute the culvert lengths listed in Table 4.1 (a channel sideslope of 2:1 is used for all calculations). These lengths represent the distance from the culvert inlet in the channel to the exit point in the natural wash. This geometric relationship is schematically illustrated in Figure 4.4.

The computed culvert lengths are multiplied by a unit cost of $78.50/lf, and then added to the headwall cost of $3,900, to arrive at the cost for each culvert. The total number of culverts for each channel system is multiplied by the culvert cost to get the total culvert cost for the system.

4.1.8 Maintenance Access to Channels

As previously illustrated in Figure 4.1, the 4-point channel cross-section includes a 15-foot wide maintenance road buffer along each side of the channel. In order to provide access to the channels for periodic maintenance work, access ramps are recommended at 0.25-mile intervals along each channel. These ramps, which would be constructed of soil cement, are illustrated in Figure 4.5.

4.1.9 Bridges

Channel construction will usually be accompanied by requirements for bridges at road intersections, or to restore access to properties that may be severed from existing roadway access. Accordingly, the channelization plans presented in this report provide a cost allowance for bridges at major roadway crossings.

It is beyond the scope of this study to provide engineering design for such bridges, however, approximate bridge lengths and widths have been estimated for the purpose of developing cost estimates. These costs were based on a simple computation of bridge deck area, that is, the bridge length times the bridge width. A standard bridge width of 62 feet was used for 4-lane roadways and a width of 88 feet was used for 6-lane roadways. The bridge length was computed as the channel topwidth plus 16 feet. The channel topwidth includes 2 feet of freeboard above the design water surface elevation. The additional 16 feet of length is provided for the deck slab to extend over the 8 foot wide soil-cement bank protection that is provided on each side of the channel. The computed deck area was multiplied by $45/sf to obtain the total bridge cost.

Table 4.2 summarizes the bridge locations, widths, and assumed channel dimensions that were used to develop bridge costs. Channel dimensions are based on the hydraulic data presented in Appendix A. For conservatism, the no low-flow culvert scenario was used; this condition will require the longest bridge spoon. These dimensions assume the channel slope parallels the existing ground slope.

4.2 PROPOSED CHANNEL ALIGNMENTS

Plate 4 presents an aerial view of the complete master drainage system proposed under this study. The channel alignments shown on Plate 4 should still be considered approximate. Minor alignment shifts may be required during final design in order to accommodate right-of-way problems or other physical constraints that might be identified during final design. Minor channel alignment shifts are acceptable, as long as new water diversions are not created or contributing drainage area boundaries changed.

The following subsections provide a brief discussion of each channel alignment. Hydrologic, hydraulic, and construction quantity and cost data for each channel are presented in Appendix A.

4.2.1 Jomax Road Channel

The channel begins approximately one-quarter mile west of the intersection of Scottsdale and Jomax Roads. The channel is located south of Jomax Road and is aligned in a westerly direction towards an intersection with the future Squaw Peak Parkway.

The upstream terminus of this channel is located on the watershed boundary between CAP Dikes 1 and 2. Accordingly, construction of this channel does not cause any unnatural water transfers between Dikes 1 and 2.

The Jomax Road channel is approximately 3.36 miles long. The intercepted drainage area varies from 0.58 square miles (at the upstream end) to 9.19 square miles at the intersection with the Squaw Peak Parkway.

Plate 5 depicts the existing ground profile along this channel alignment. When viewing Plate 5, the reader should be aware that this plate also includes the Squaw Peak Parkway and Pinnacle Peak Road (east) ground profiles along their respective channel alignments. The beginning of the Jomax Road channel is marked on Plate 5.

Table A.1 (Appendix A) lists the hydraulic and cost estimating data for both this channel and that portion of the Squaw Peak Parkway Channel located north of Pinnacle Peak Road. This data assumed the channel will parallel the existing ground profile. When referring to Table A.1, the Jomax Road channel extends from Stations 230+60 to 407+90.

4.2.2 Squaw Peak Parkway Channel

This channel extends north from the detention basin behind CAP Dike 1 to the intersection with the Jomax Road channel discussed in Section 4.2.1. Approximately 2.36 miles of this channel are located between the CAP dike and Pinnacle Peak Road, while an additional 1.27 miles of channel extends north of Pinnacle Peak Road to the intersection with the Jomax Road channel; the total channel length is 3.63 miles.

The design of this channel is based on a key assumption that no additional flows intercepted by the future Squaw Peak Parkway, north of the Jomax Road channel intersection, will be diverted south along the east

side of the Parkway, that is, it is assumed those intercepted flows will be passed through the parkway along their natural, southwesterly flow path.

The drainage area intercepted by this channel varies from 9.19 square miles at the intersection with the Jomax Road channel, to approximately 14.38 square miles at CAP Dike 1. The off-site drainage design for the Outer Loop Highway/Squaw Peak Parkway interchange may cause a slight change in the assumed drainage area boundaries near the interchange. It is important that the final channel design be closely coordinated with ADOT to ensure compatibility between this interceptor channel and the off-site drainage design for the Outer Loop.

Plate 5 depicts the existing ground profile for this channel. The Squaw Peak Parkway channel extends from Station 38+80 to Station 230+60. The reader is again cautioned to distinguish between the multiple channel alignments shown on Plate 5.

Table A.1 (Appendix A) lists the hydraulic and cost estimating data for that reach of the Squaw Peak Parkway channel located north of Pinnacle Peak Road (Stations 163+30 to 230+60), while Table A.2 lists similar data for that reach south of Pinnacle Peak Road (Stations 38+80 to 163+30).

4.2.3 Pinnacle Peak Road Channel (East)

As with the Jomax Road channel, this portion of the Pinnacle Peak Road channel extends west, from the drainage area divide between CAP Dikes 1 and 2, to the Squaw Peak Parkway channel. This channel, which is presently aligned about 300 feet north of (and parallel to) Pinnacle Peak Road, is approximately 2.24 miles long.

The intercepted drainage area ranges from 1.04 square miles to 3.78 square miles at Squaw Peak Parkway. These areas assume the Jomax Road channel is in place and operating to provide 100 percent interception of upstream runoff.

Plate 5 depicts the existing ground profile for this channel. This channel lies between Stations 163+30 and 281+80 on Plate 5. The reader needs to distinguish between the multiple profiles presented on Plate 5.

Table A.2 (Appendix A) lists the hydraulic and cost-estimating data for this channel. When referring to Table A.2, the east segment of the Pinnacle Peak Road channel extends from Station 163+30 to Station 281+80. The remaining data in this table applies to the Squaw Peak Parkway channel.

4.2.4 Pinnacle Peak Road Channel (West)

This channel begins immediately west of the proposed Squaw Peak Parkway and extends west, along the north side of Pinnacle Peak Road, to Cave Creek Road. The channel then turns southwesterly and parallels the east side of Cave Creek Road to an outlet in CAP Dike 1.

This channel intercepts runoff from an approximate 4.05 square mile drainage area that lies north of this segment of Pinnacle Peak Road. The drainage area excludes that area intercepted by the Jomax Road and Squaw Peak Parkway channels. The total channel length is 3.07 miles. This channel is not connected to the Squaw Peak Parkway channel.

Plate 6 illustrates the existing ground profile along the west Pinnacle Peak Road channel alignment. Table A.3 (Appendix A) lists the hydraulic and cost estimating data for this channel.

4.2.5 Rawhide Wash Channel

In terms of peak discharge, this is the second largest of the channel systems proposed for the master drainage plan. The Rawhide Wash channel is also somewhat unique in that it will require a system of training dikes at the upstream end in order to divert water around residential development located southwest of the Jomax Road/Hayden Road intersection. The natural channel of Rawhide Wash also begins a transition to a braided flow pattern in this general vicinity. Accordingly, the training dikes will serve a dual purpose of funneling the braided flow to the entrance of the proposed man-made channel.

A major tributary to Rawhide Wash crosses Jomax Road approximately 2,100 feet west of Pima Road. In order to intercept this flow, prior to it crossing Jomax Road and entering a residential area, the east training dike is extended along the north side of Jomax Road for approximately 2,400 feet. This extended dike will divert this tributary into Rawhide Wash at Jomax Road.

Detailed topographic mapping will have to be obtained in order to establish an exact alignment and length of the training dikes. Based on 1 inch = 2,000 feet (10 feet C.I.) quadrangle maps, it would appear the

west training dike might be about 4,000 feet long, while the east training dike would be approximately 7,000 feet long (including the Jomax Road interceptor levee).

The training dikes will transition to the proposed channel entrance at a location along Hayden Road, about 500 to 800 feet north of Happy Valley Road. The channel will then continue in a southwesterly direction to an ultimate outfall into the detention basin behind CAP Dike 2.

Excluding the training dikes, the Rawhide Wash Channel is approximately 6.26 miles long. The intercepted drainage area varies from 14.24 square miles, at the channel inlet, to 22.59 square miles at the channel outlet behind CAP Dike 2. This total area of 22.59 square miles includes 2.97 square miles of drainage area intercepted by the Scottsdale Road Channel, which discharges into the Rawhide Wash channel.

After the Rawhide Wash Channel crosses to the west side of Scottsdale Road, it remains on State Trust land until reaching an outfall at the CAP detention basin. This strip of State land (between Scottsdale Road and 64th Street, referred to as the Scottsdale-Beardsley parcel) is currently being considered by the State Land Department for detailed planning. Accordingly, the final alignment of the channel through this property will probably be changed from what was assumed for this report. Such a change should not create any problems, as long as any alignment shift does not infringe on the drainage boundaries for adjacent channels.

The HEC-1 model that was developed for Rawhide Wash assumes that a system of collector channels will ultimately be constructed to drain all of Sub-Basins 29A and 29.1 (see Plate 3) into the Rawhide Wash Channel. Accordingly, the main channel could be aligned anywhere within these two sub-basins without causing any significant change to the peak discharge for this reach of the channel. However, if the channel were re-aligned outside the boundaries of Sub-Basins 29A and 29.1 (for example, down Scottsdale Road) the model would have to be revised to reflect new drainage boundaries.

Plate 7 illustrates the existing ground profile along the Rawhide Wash alignment. Table A.4 (Appendix A) lists the hydraulic and cost estimating data for this channel.

4.2.6 Scottsdale Road Channel

As stated in the previous section, the Scottsdale Road channel is actually part of the Rawhide Wash channel system. The Scottsdale Road channel is aligned along the east side of Scottsdale Road and extends from a point approximately 2,200 feet north of Jomax Road to an intersection with the Rawhide Wash channel at a location about 900 feet south of Pinnacle Peak Road.

The total channel length is 2.5 miles. The intercepted drainage area ranges from 1.48 to 2.97 square miles. The upstream drainage boundary of this channel is located on the watershed divide between CAP Dikes 1 and 2.

Plate 7 presents an existing ground profile along the channel alignment. Table A.5 (Appendix A) lists the hydraulic and cost estimating data for this channel.

4.2.7 Pima Road Channel

This channel begins approximately 1,100 feet north of Jomax Road and extends south for 5.31 miles along Pima Road. The channel will be located on the east side of Pima Road from the upstream end to Deer Valley Road. At the Deer Valley/Pima Road intersection, the channel crosses to the west side of Pima Road and connects to the Core North segment of the Pima Road channel. The channel then remains on the west side of Pima Road and ultimately outlets to a large detention basin (D.B. 53R) along the north side of the Outer Loop Highway (approximately midway between Pima and Hayden Roads).

The total drainage area intercepted by the Pima Road channel is dependent upon the disposition of flows from the Reata Pass alluvial fan apex. That section of the Pima Road channel that extends north of Deer Valley Road should not be impacted by flows from the Reata Pass fan. For this northern section of channel, the intercepted drainage area ranges from 1.31 square miles to 5.07 square miles at the Deer Valley Road intersection. The Pima Road channel segment south of Deer Valley Road will intercept an additional 2.75 square miles if the Reata Pass flows are diverted south to CAP Dike 4. If the Reata Pass flows are not diverted to CAP Dike 4, the Pima Road channel (south of Deer Valley Road) will have to be designed for an additional 7.88 square miles of intercepted drainage area. In other words, the <u>total</u> intercepted drainage area at the <u>outlet</u> of the Pima Road channel will be 7.82 square miles, if Reata Pass is diverted to CAP Dike 4, and 15.70 square miles if Reata Pass is not diverted to Dike 4.

The upstream terminus of the Pima Road channel is located on the eastern drainage boundary of the Rawhide Wash channel watershed. Approximately the northern 6,800 feet of the Pima Road channel will intercept about 2.37 square miles of area (Sub-Basins 30R, 31A and 34R, Plate 3) that presently drains to CAP Dike 2. The proposed channel will divert runoff from this area to Core North Detention Basin 53R, which outlets to CAP Dike 3.

Existing drainage improvements in and around the Pinnacle Peak County Club have also altered the natural drainage pattern of this area. Historically, runoff from Sub-Basins 35N, 35L, and 36L (refer to Plate 2) passed through Sub-Basins 37, 38 and 38.1 en route to CAP Dike 3. Construction of drainage channels along the north side of Pinnacle Peak Road (extending west from Pima Road) has intercepted this natural flow path and diverted it through the golf course at the Pinnacle Peak County Club. The existing contouring of the golf course further directs this flow into an existing wash that crosses Hayden Road approximately 1,500 feet north of Deer Valley Road.

This sequence of man-made diversions effectively diverts portions of the outflow from Sub-Basin 36L to Sub-Basin 32 rather than to Sub-Basin 37. The result is a transfer of runoff from CAP Dike 3 to Dike 2.

Given the braided flow pattern in this area, and the lack of high resolution topography, it is difficult to accurately quantify the amount of diversion that these man-made alterations create. The occurrence of a major storm, such as the 100-year event, will most probably exceed the bankfull capacity of these man-made drainage diversions. As a result, these overflows (especially through the golf course) may return to their historic flow path. In recognition of this possibility, a divert operation was placed at the outlet of Sub-Basin 36L when modeling the existing condition scenario (that is, without the channels proposed under this master drainage plan). This judgementally derived divert ratio directs 412 cfs from Sub-Basin 36L to Sub-Basin 37 during the peak 100-year outflow of 1,224 cfs from Sub-Basin 36L. When the Pima Road channel was inserted in the model, this divert ratio was removed because the flows into the Pinnacle Peak County Club golf course were substantially reduced by the interceptor channel along Pima Road. The reduced flows were assumed to be of insufficient magnitude to spill over the drainage boundaries through the golf course area.

Construction of the Pima Road channel will return the majority of the runoff from Sub-Basins 35N, 35L, and 36L1 back to its historical outfall behind CAP Dike 3. Only Sub-Basin 36L2 (see Plate 3) and a

small section of the northeast portion of Sub-Basin 32A will continue to be diverted to CAP Dike 2 after installation of the Pima Road channel.

In summary, construction of the Pima Road channel will divert 2.37 square miles of drainage area from CAP Dike 2 to Dike 3, while restoring the majority of runoff from about 1.31 square miles of drainage area to its historical outlet behind CAP Dike 3.

A complete reservoir routing analysis for all four CAP dikes is presented in Section 7 of this report. The reservoir routing analysis quantifies the impact that the proposed master drainage plan has on each CAP dike.

As indicated previously, the final design of that portion of the Pima Road channel located south of Deer Valley Road will be dependent upon the installation of the Reata Pass channel. Should the south half of the Pima Road channel be constructed prior to the Reata Pass channel being operational, regulatory agencies will most likely require the southern portion of the Pima Road channel (as well as Core North Detention Basin 53R) to be designed to accommodate the additional inflows from Reata Pass. Conversely, if the Reata Pass channel precedes the Pima Road channel, the Pima Road channel could exclude the Reata Pass flows from its design capacity.

Plate 8 presents an existing ground profile along the proposed Pima Road channel alignment. Table A.6 (Appendix A) lists the hydraulic and cost estimating data for this channel, with no inflows from Reata Pass. Table A.7 lists similar data for the condition that includes Reata Pass flows in the Pima Road channel.

4.2.8 Deer Valley Road Channel, Hayden to Pima

This channel is part of the drainage system proposed by the developers for Core North. Deer Valley Road forms the north boundary of Core North. Accordingly, an east-west aligned channel is proposed along the south side of Deer Valley Road to intercept off-site drainage. The Deer Valley Road channel is actually composed of two non-connected alignments. The western segment will be discussed in the following sub-section.

The eastern section of this channel extends from Pima Road to Hayden Road. The east end of this channel is not connected to the Pima Road channel, that is, it will only intercept that runoff that is generated west of the Pima Road channel.

The channel is approximately one mile long and intercepts a drainage area of about 0.68 square miles (excluding any low-flow culvert releases from the Pima Road channel).

At Hayden Road, the channel turns south and becomes part of the interior drainage system for Core North. The channel is ultimately connected to Detention Basins 38R-1 and 38R-2, both of which are part of the Core North system.

Only that part of this channel that lies adjacent to Deer Valley Road was investigated as part of this study. Plate 6 presents an existing ground profile for that portion of the channel that extends from Pima Road to Hayden Road. Table A.8 (Appendix A) lists the hydraulic and cost-estimating data for this channel. The peak discharges listed in Table A.8 assume the Pima Road channel is in place north of Deer Valley Road.

4.2.9 Deer Valley Road Channel, Hayden to Scottsdale to Beardsley

This channel is the western segment of the off-site drainage system for the north and west boundaries of Core North. The upstream end of the channel is located on the watershed divide between CAP Dikes 2 and 3.

The channel begins at Hayden Road (with no connection to the channel discussed in the preceding section) and extends west along the south side of Deer Valley Road to an intersection with Scottsdale Road. The channel then turns south and parallels the east side of Scottsdale Road for one mile (that is, to Beardsley Road). As originally proposed by Carter Associates, Inc., the channel would terminate at this point with some type of design that would sheet the water in a southwesterly direction across Scottsdale Road.

This channel is approximately 2 miles long and intercepts an off-site drainage area of about 2.22 square miles. This area includes Sub-Basins 31B, 36L2, 32A, and 20 percent of Sub-Basin 28B (see Plate 3). This drainage area boundary assumes the Pima Road channel is in place north of Deer Valley Road. That portion of the channel that extends south along Scottsdale Road intercepts an additional 0.55 square miles (approximate area) of interior drainage from Core North.

Plate 6 presents an existing ground profile along the Deer Valley Road channel alignment extending from Hayden Road to Scottsdale Road to Beardsley Road. Table A.9 (Appendix A) lists the hydraulic and cost estimating data for the same reach of channel.

4.2.10 Reata Pass Channel

This is the largest of the channels proposed for the regional drainage plan. At the present time, this channel is also unique in that it will utilize a very wide, benched cross-section which would include provisions for hiking and riding trails. Figure 4.2 illustrates this cross-sectional geometry. The reader is urged to read the "Important Notice" at the front of this report before proceeding further with this section.

At the time this report was prepared, the available data on the Reata Pass channel was incomplete. The inter-agency Task Force had originally identified the Thompson Peak Parkway as the preferred alignment for this channel. Under this scenario, the channel would begin at the apex of the Reata Pass alluvial fan, which is located at the intersection of Pinnacle Peak Road with the west side of the McDowell Mountains. The channel would leave the apex in a southerly direction until being intercepted by a large natural wash located approximately 3,300 feet south of the apex (Pinnacle Peak Road). It would then follow this natural wash until intersecting the proposed Thompson Peak Parkway alignment. The channel would parallel the east (upstream) side of the parkway until reaching an outfall at CAP Detention Basin 4. Based on this alignment, the channel would be approximately 5.53 miles long.

This original channel alignment is identified on both Plate 3 and Plate 4. The intercepted drainage area varies from 7.88 square miles at the fan apex to 18.27 square miles at the outlet to CAP Dike 4. Plate 9 depicts the existing ground profile for this channel, along the Thompson Peak Parkway alignment.

Serious engineering problems were identified for this alignment during the latter phase of this study, when the preliminary sediment transport analysis was performed. These problems are created by two curves in the proposed parkway that lie along a flat gradient, that is, parallel to elevation contour lines. These two curves encompass about 3,600 feet of the total channel alignment.

In order to carry the sediment loads through these curves, the channel must maintain a slope of approximately 2.5 to 2.9 percent. Attempts to extend these slopes through the flat curves result in a deeply incised channel. For example, the channel would reach depths of nearly 60 feet through the first

curve. Once the channel is this deep into the ground, the problem only becomes worse as the channel is extended further south. By the time the channel is through the second flat curve, the incisement depth is in excess of 100 feet. Such channel depths are totally impractical for this project. Even if the depths were tolerable, from a cost and esthetics standpoint, the channel would never be capable of outletting to CAP Detention Basin 4, that is, by the time the channel reached the detention basin, it would probably be nearly 200 feet below the basin floor. Accordingly, alternate alignments will have to be considered for the Reata Pass channel.

A recommended approach would be to abandon plans for a continuous channel along the east side of the parkway and pass floodwaters through the parkway at its intersection with major washes. This would eliminate the flat gradient problems encountered at the roadway curves. The natural washes could be enlarged to carry the concentrated flows to an outlet at CAP Dike 4.

Under this alternate scenario, there would still be a channel along the upstream side of Thompson Peak Parkway, but it would not be a continuous link to the Reata Pass alluvial fan apex. After flows have been accumulated and passed through the parkway at a major wash intersection, a new channel would begin immediately south of the pass-through point. This new channel would begin intercepting runoff until the next major wash was encountered, at which point the channel would again pass through the parkway and continue along a natural wash alignment.

Figure 4.6 presents a schematic drawing of possible channel alignments that might be considered for this alternate approach. These alternatives only address alignment changes west of Thompson Peak Parkway. The connection from the parkway to the Reata Pass alluvial fan apex would be the same for all alternatives (see Plate 4). Figure 4.6 also shows the preferred alternative recently recommended by the Reata Pass Task Force. It should be noted that these concepts may require multiple channels west of Thompson Peak Parkway.

Should any of these alternate scenarios be pursued, it is recommended that the use of any existing channels be done with attention to preserving the esthetics along such channels. The flows in the upper (northern-most) channel might be large enough to retain the benched, 8-point cross-section shown in Figure 4.2. Some of the smaller channels might use a single bench, or a simple 4-point trapezoidal section. Maximum use of natural landscaping should be applied to all channel cross-sections.

It should be noted that any of these alternate channel concepts can still utilize the same channel alignment north of the intersection with the Thompson Peak Parkway. This northern section of channel has sufficient natural slope to work very well in transporting the sediment inflows. The majority of this channel segment follows the natural alignment of a well-defined wash.

As stated previously, engineering analyses of alternate channel alignments were not pursued as part of this report. Accordingly, cost estimates and hydraulic performance data for such alternatives are not available. However, a hydraulic and cost estimating spreadsheet had been prepared for the channel alignment along Thompson Peak Parkway prior to conducting the sediment transport analysis which indicated the alignment was unfeasible. The City of Scottsdale requested that the channel excavation and lining data for this original alignment be included in this report to provide some allowance for the Reata Pass channel. Accordingly, Table A.10 presents the hydraulic and cost estimating data for the Reata Pass channel along the Thompson Peak Parkway alignment. The data in this table is based on a constant, supercritical bed-slope of 0.020 ft/ft, that is, it does not parallel the existing ground profile, as do the other tables in Appendix A.

Plate 9 presents an existing ground profile along the Thompson Peak Parkway alignment. The flat spots in the gradient are clearly visible near Stations 96+00 and 184+00.

Table 6.10, which will be discussed in detail under Section 6.2 of this report, presents the results of the sediment transport and equilibrium slope analysis that generated the data which indicated the Thompson Peak Parkway alignment was unsuitable.

4.2.11 Core North Drainage System

Numerous references have previously been made to the Core North drainage system, as part of the discussion of the Pima Road and Deer Valley Road channels. Core North is the name given to a 4-square mile block of State Trust Land located between Scottsdale and Pima Roads and extending north from the Outer Loop Highway to Deer Valley Road.

A preliminary drainage plan had been developed for Core North prior to the initiation of the study effort for the Upper Indian Bend Wash regional drainage plan. As stated previously, one of the objectives of this effort was to integrate into the regional plan, as much as possible, any existing drainage plans for

the watershed. Accordingly, the previously developed Core North drainage system became a key element of the proposed regional plan.

Figure 4.7 is a schematic of the Core North drainage system. The interior sub-basin boundaries (CN1 - CN6) are only approximate and may be subject to change as more detailed planning occurs.

The proposed development is to be protected from off-site flows by interceptor channels along Deer Valley Road and Pima Road. An interior channel system is also provided for the interception of on-site runoff.

Detention basins are also an integral part of this plan. Off-site drainage intercepted by the Deer Valley Road channel, between Pima and Hayden Roads, will drain to Detention Basins 38R-1 and 38R-2. Off-site drainage intercepted by the Pima Road channel (south of Deer Valley Road) will be routed to Detention Basin 53R. These three basins will also provide detention storage for on-site runoff.

Basins 38R-2 and 53R will outlet, via box culverts, through the Outer Loop Highway and tie into the Core South channel system. These detention basins are essential elements of the regional drainage plan because of the attenuation they provide to the peak discharge associated with the off-site drainage. The Outer Loop cross-drainage system, and the Core South channel system, are both dependent upon these basins being operational and limiting their release rates to specified maximum values.

The Core North drainage system, illustrated in Figure 4.7, was conceived on the basis of providing complete protection of the 4-square mile tract of land without dependence on any of the upstream channel systems proposed in this study, that is, without the northerly extension of the Pima Road channel and construction of the Reata Pass channel.

Construction of the proposed regional drainage plan would extend the Pima Road channel north to Jomax Road. This extension would decrease the inflows to the Deer Valley Road channel, but would increase the flows to the Pima Road channel, south of Deer Valley Road. However, if the Reata Pass channel is constructed with an outfall to CAP Dike 4, there would be a substantial reduction to the peak discharge in the Pima Road channel, as well as to the required storage volume in detention basin 53R.

As part of the HEC-1 data base developed for this study, the Core North drainage plan has been modeled both as a stand alone system, that is, with none of the upstream channels proposed as part of the regional drainage plan, as well as with these upstream channels in place.

At the time this report was prepared, the planning team for Core North had not finalized the detention basin geometry for basins 38R-1, 38R-2, and 53R. As a result, the HEC-1 models used for this study may not reflect the final release rates from these basins. Section 5.0 discusses criteria that were used to size the Outer Loop Highway culverts that tie into the outlets of these detention basins.

4.2.12 Core South Drainage System

Core South is another block of State Trust Land located between Scottsdale and Pima Roads and lies adjacent to the south side of Core North. Core South is served by an intricate network of drainage channels which are dependent upon the Core North detention basins being in place.

The Core South drainage system was originally planned by the Forest City Scottsdale Company (in association with Collar, Williams and White Engineering, Inc.), and subsequently integrated into an Outer Loop Highway off-site drainage study performed by Simons, Li & Associates, Inc. (SLA) in June, 1987. Accordingly, the Core South drainage plan was an existing concept that was integrated into the master planning for the Upper Indian Bend Wash regional drainage plan.

No additional engineering analyses have been conducted for the Core South system as part of this study. The entire Core South channel system is included in all master plan HEC-1 models that were developed for the regional drainage plan study. The HEC-1 components for the Core South system were taken directly from the 1987 SLA HEC-1 models labeled D4 and D5 (see Table 4.2 of 1987 SLA report entitled *Final Report, Drainage Analysis for State Trust Lands, Outer Loop Highway, Scottsdale Road to Pima Road*). The upstream watershed was stripped from these models so that only the Core South drainage system remained. The remnants of the SLA models were then inserted at the Core North detention basin outlets, that is, detention basins 38R-2 and 53R. Three additional sub-basins (PR1, TPC1 and TPC2), were added to the model to simulate runoff from those areas lying between CAP Dike 3 and Core South. These three sub-basins are shown on Plate 3.

As a matter of technical interest, it should be noted that on-going planning studies have proposed changes to the Core South channel system. These changes include alterations to channel alignments, and to

detention basin outlet geometry and locations. Such changes will alter the design discharges previously specified in the 1987 SLA report for the Core South channel system. Accordingly, prior to any further analysis of Core South channel alignments, this portion of the HEC-1 model needs to be modified for any agreed to changes in channel location and detention basin performance.

None of the Core South channel system is included in the cost estimate prepared for this study. Design of the Core South drainage system is being pursued by private interests, in cooperation with the Arizona State Land Department. However, since the Core South system is only being designed to accommodate peak discharges that assume detention basins 38R-2 and 53R are in place in Core North, it is extremely important that the Core South system be recognized as an integral part of the overall regional drainage plan. It is equally important that any further refinement of the Core North system recognize the discharge limitations to the Core South system.

5.0 OUTER LOOP HIGHWAY DRAINAGE STRUCTURES

Construction of the proposed regional drainage and flood control plan will have a significant impact on the design of the cross-drainage system for the Outer Loop Highway. The channelization and detention basin system will allow a substantial decrease in both the number and size of culverts required for the highway.

The following subsections of this report present a brief summary of the cross-drainage structures that will be required with the regional plan in place. Table 5.1 summarizes the structure sizes and locations. The data in Table 5.1 is still subject to change as part of the final design process.

5.1 SQUAW PEAK PARKWAY CROSSING

As discussed in Section 4.2.2 of this report, an interceptor channel is recommended along portions of the proposed Squaw Peak Parkway, south of Jomax Road. This channel will have to be incorporated into the design of a major interchange at the intersection of the Outer Loop Highway and the Squaw Peak Parkway. Discussions with ADOT's management consultant for the Outer Loop, DeLeuw, Cather and Company (DCCO), indicate the possibility of two alternate channel alignments through the interchange. One option would retain a straight north-south channel alignment through the interior portion of the interchange. The second alternate would loop around the east end of the interchange. DCCO indicates that right-of-way costs and ramp structure requirements would probably dictate which alignment would be most feasible. A detailed analysis of these issues will have to be performed as part of the final design for the interchange. For the purpose of this study, Table 5.1 lists a design discharge and open channel parameters that could be used to approximate channel costs through the interchange.

5.2 DESERT RIDGE CROSSINGS

Desert Ridge is an approximate 8.5 square mile, planned community development located between Pinnacle Peak Road and the CAP aqueduct. The area is bounded on the west by 32nd Street and on the east by a northern extension of 64th Street.

Site planning and drainage system design are being performed by BRW, Inc. As part of this planning process, BRW has coordinated the drainage system design with both the City of Phoenix and ADOT. This planning effort has already identified specific sizes and drainage structure locations to pass floodwaters through the Outer Loop. This information is listed in Table 5.1.

A concept drainage system for Desert Ridge was already developed at the time the study was initiated for the *Upper Indian Bend Wash Regional Drainage and Flood Control Plan.* Accordingly, the culvert sizes and locations for the Desert Ridge segment of the Outer Loop is not dependent upon the regional plan being installed. BRW has proposed a system of both permanent and temporary "wing berms and channels" to intercept sheetflow and divert such flow into designated drainage corridors. These drainage corridors are linked to the culvert crossings of the Outer Loop. Table 5.1 lists three culvert crossings through Desert Ridge and one bridge crossing at 52nd Street.

5.3 RAWHIDE WASH CROSSING

The proposed Rawhide Wash channel will cross the Outer Loop at some point between 64th Street and Scottsdale. Although a possible channel alignment is presented in this report, that alignment is only intended to be used for general planning and cost estimating purposes.

That portion of the Rawhide Wash channel that is located west of Scottsdale Road is situated on state trust land (previously referred to in Section 4.2.5 as the Scottsdale - Beardsley parcel). As stated previously, this parcel is currently being considered for detailed planning. Accordingly, there is a high probability that the channel alignment will be shifted to some other location during this pending planning effort. With the exception of the Outer Loop station location, the Rawhide Wash channel data listed in Table 5.1 should be applicable to any location that may ultimately be selected for this crossing.

Depending upon the type of drainage system that is ultimately designed for the Scottsdale-Beardsley parcel, there will probably still be a need for some small drainage channels along the upstream side of the Outer Loop. These small channels would collect any remaining drainage, that is not diverted into the main trunk channel upstream of the highway crossing, and carry such drainage to the culvert or bridge crossing provided for the Rawhide Wash channel.

As a matter of technical interest, the HEC-1 model for this channel includes collector channels through Sub-Basin 29A, which is located on the upstream side of the Outer Loop.

5.4 OUTER LOOP SECTION 9A

This section of the Outer Loop Highway, which extends from Scottsdale Road to Bell Road, will be protected by the Core North detention basins and the upstream channel system that discharges to these basins. The outlets from Detention Basins 38R-2 and 53R are the only two crossings through this section of the highway.

In addition to the two detention basin outlets, there will be a series of small channels along the north side of the Outer Loop to intercept local runoff that occurs downstream of the basin inlets. These local interceptor channels will outlet to the same box culverts that receive the discharge from the detention basins.

Planning studies for Core North were not finalized at the time this report was written. There are still several unresolved issues relating to detention basin design (Core North) and receiving channel alignments in Core South. Until these "loose ends" are tied down, the information in Table 5.1 for section 9A is subject to revision.

As part of the original 1987 SLA drainage study for Core South, one, 10 feet x 4 feet concrete box culvert (CBC) outlet was proposed at the east end of Detention Basin 53R. An emergency spillway was proposed to outlet at the west end of the basin and drain into a proposed channel along the east side of Hayden Road.

During 1990, Stanley Consultants revised the detention basin geometry and outlet configuration. This revision removed the emergency spillway at Hayden Road and replaced the single eastern box culvert outlet with two CBC outlets (three, 10 feet x 4 feet and one, 10 feet x 4 feet). During the reservoir routing operations that were performed with HEC-1, these new outlet configurations were found to exceed the maximum allowable outlet discharge of 2,518 cfs to Core South. Accordingly, for the purpose of this study, the outlets were down-sized to two, 10 feet x 4 feet and one, 5 feet x 4 feet CBCs in order to stay under the 2,518 cfs limit. Both culverts have an invert elevation of 1,600 feet mean sea level (MSL). Table 5.2 lists the performance characteristics of this basin (with the downsized outlets) with and

without the diversion of Reata Pass flows to CAP Dike 4. If Reata Pass is not diverted to CAP Dike 4, the embankment height and storage requirements for basin 53R will cause it to be classified as a dam. Such a classification will require compliance with ADWR dam safety criteria. It is anticipated that further refinements will be made to the configuration of Detention Basin 53R.

Final geometry for Detention Basin 38R-2 has not yet been completed by the Core North planning team. Using the current storage and discharge relationships provided by the Core North consultant, the basin was found to overtop when subjected to the 100-year event with the upstream regional channelization plan in-place. The maximum discharge during this overtopping condition was 694 cfs, which slightly exceeds the maximum allowable discharge of 652 cfs published in the previously referenced 1987 SLA report for the Core South drainage system. Accordingly, some minor revisions will be required to this basin prior to the Core North planning effort being completed. The present box size of one, 10 feet x 4 feet may be changed, depending on the results of the revised reservoir routing analysis for this basin.

6.0 SEDIMENT TRANSPORT AND SCOUR ANALYSIS

The arid landscapes of the southwest deserts are notorious sediment producers. The typically cohesionless soils of the desert, sparse vegetation, and high intensity rainfall cause large amounts of sediment to be moved through the washes, arroyos, and rivers of the southwest.

There is a continual, dynamic interaction between sediment particles and the transporting medium, water. As water moves sediment through a drainage system, there is a constant struggle to achieve a state of equilibrium, or balance, between sediment supply and sediment transport capacity. In seeking this balance, the drainage system is in a continual mode of change as both vertical and horizontal adjustments are made to the channel boundaries of the system's watercourses. In natural, undisturbed watersheds, these changes may take place very slowly (hundreds or thousands of years), but when man-made urbanization disrupts such watersheds, large magnitude changes can occur very rapidly. Failure to anticipate, quantify, and design for these adjustments can lead to serious damage and/or a poorly functioning flood control system.

Even though the proposed channel systems presented in this report are at a conceptual level of planning, a preliminary sediment transport analysis was performed in order to identify any potential problem areas. Accordingly, the following sub-sections address the potential for bed scour and long-term bed-slope adjustments.

6.1 SCOUR ANALYSIS

The design of a bank protection system must consider the potential for scour of the channel bed. Failure to do so could lead to the toe of the bank protection material being undercut by scour processes that will be induced by flowing water. Should this situation occur, the bank lining material may collapse into the scour hole, thus exposing the bank to erosive velocities and possible lateral movement.

Due to the concept level nature of this study, it was not practical to conduct a detailed scour analysis of 35 linear miles of channel. However, a general assessment of scour potential was conducted in order to establish a typical toedown dimension that could be used in the channel cost estimate.

Vertical incisement of the channel bed can occur in response to the following six processes:

$$\Delta Z_{TOT} = \Delta Z_{DEG} + \Delta Z_{LS} + \Delta Z_{GS} + \Delta Z_{BS} + \Delta Z_{I} + \Delta Z_{AD} \tag{6.1}$$

where: ΔZ_{TOT} = Total vertical adjustment in bed elevation

ΔZ_{DEG} = Vertical change due to long-term degradation

ΔZ_{LS} = Vertical change due to local scour

ΔZ_{GS} = Vertical change due to general scour

ΔZ_{BS} = Vertical change due to bend scour

ΔZ_{I} = Vertical change due to low-flow incisement

ΔZ_{AD} = Vertical change to antidune troughs

A brief discussion of each of these phenomena, and its applicability to this project, is presented in the following paragraphs.

1. **Long-Term Degradation** - This process occurs over a long period of time in response to an imbalance between the sediment transport capacity of the channel and the dominant sediment supply to the channel. When such imbalances occur, the channel will naturally adjust its slope to restore equilibrium between the transport capacity and incoming supply of sediment. If the transport capacity of the channel exceeds the sediment supply, the channel will flatten its slope (degrade). However, should the sediment supply exceed the transport capacity of the channel, the channel slope will increase (aggrade) in order to generate higher velocities that are capable of moving the sediment inflows.

   Long-term degradation is very difficult to quantify because of the many complex variables that drive this process. Accordingly, numerous assumptions have to be made on the basis of engineering judgement. A preliminary equilibrium slope analysis has been conducted in an effort to establish an approximate magnitude of long-term degradation for the flood control channels proposed in this study. The assumptions and results of that analysis are presented in Section 6.2.

2. **Local Scour** - Local scour will occur in response to objects being placed in the path of flowing water. The most common form of local scour is that occurring at bridge piers

and protruding bridge abutments or spur dikes. Accordingly, this process will not be applicable to the vast majority of the channel reaches discussed in this report. During final design, local scour will have to be evaluated on a site-specific location for any proposed bridge crossings of the channels.

3. **General Scour** - This scour process occurs in response to changes in channel geometry from one reach of a channel to the next. As a channel contracts and expands, its flow velocity (and thus sediment transport capacity) will change. General scour will occur when a channel contracts (in the downstream direction) and causes an increase in velocity through the contracted section. The increase in sediment transport capacity through the contracted reach will begin to remove more sediment from the bed of the contracted reach than is being delivered to the contraction by the wider, upstream reach. The result is a lowering (general scour) of the channel bed through the contracted reach. When the channel geometry expands in the downstream direction, the opposite effect can occur, that is, sediment deposition will take place in the wider channel section.

   General scour, and/or sediment deposition is usually quantified with a mobile-boundary sediment routing model. Such models are capable of predicting scour and deposition patterns as a function of bed-material size, channel geometry, and changes in discharge that occur during passage of a specific flood hydrograph.

   Unless changes in channel geometry are extreme, typical values of general scour are usually in the 0.5 to 2.0 feet range. The final design of the channels proposed in this study will probably attempt to minimize numerous changes in channel geometry and slope. Such changes will probably only occur in response to variations in discharge and existing ground profile. As a result, the potential for general scour should be confined to these channel transitions, rather than occurring throughout reaches of channel that maintain constant geometry and slopes. Accordingly, only a minimal allowance of 0.5 feet of general scour was included in the constant scour dimension that is being applied to all 35 miles of the proposed channel system.

4. **Bend Scour** - As the name implies, this process only occurs in the vicinity of channel curvature. There are very few bends in the 35 miles of channel proposed in this report.

Although bend scour should definitely be included for any channel bends during final design, it is an infrequently occurring, localized phenomenon that does not warrant inclusion in the total scour depth applied to the cost estimate for the entire channel system.

5. **Low-Flow Incisement** - Man-made channels with large width to depth ratios are very vulnerable to the formation of low-flow channels. When trapezoidal channels, designed to carry large events such as the 100-year flood, are exposed to smaller, more frequent flows (2 to 5 year floods), the wide channel bottomwidths may cause a shallow sheetflow condition to exist. Rather than transporting these smaller flows in this manner, the channel will develop a low-flow channel that provides a more efficient conveyance of these small discharges. Low-flow channels will meander across the bottom of the larger, parent channel, thus randomly coming into contact with the channel bank. Accordingly, it is important to acknowledge low-flow incisement when computing the total scour depth for bank-lining design. For the purpose of this study, 1 foot of low-flow incisement is included in the total scour depth for use in the bank-lining toedown dimension.

6. **Antidune Troughs** - Sand bed channels are prone to the development of bedforms, such as dunes and antidunes. Such bedforms create troughs, or depressions, below the natural bed of the channel. In order to account for the possibility of these troughs forming adjacent to the toe of the bank, it is prudent to include bedform troughs in the estimate of total scour. Based on laboratory flume studies, the maximum depth of these troughs (below the existing channel bed) is approximately equal to $0.0135\ V^2$ or one-half the depth of flow, whichever value is less. A trough depth of 2 feet is the maximum depth that could occur in a channel flowing 4-feet deep.

6.1.1 Summary of Scour Analysis

Based on the discussions in the preceding paragraphs, the following allowance was made for computing the amount of bank-lining that would be needed to prevent undercutting at the toe of the channel bank.

1.	Long-Term Degradation	-	1.0 feet
2.	Local Scour	-	0

3.	General Scour	-	0.5
4.	Bend Scour	-	0
5.	Low-Flow Incisement	-	1.0
6.	Antidune Troughs	-	2.0
	Subtotal:	-	4.5 feet
	Safety Factor:	-	x 1.5
	Total:	-	5.85 feet
	Rounded To:	-	6.0 feet

Using a constant 6-foot scour depth for cost estimating purposes is considered reasonably conservative, given the fact that with the exception of the Reata Pass channel, the remaining channels are presently sized with a constant flow depth of 4 feet. Toedown depths for the 8-point cross-section, used at Reata Pass, were increased to 8.5 feet to reflect potentially larger antidune trough depths. Any localized deviations from these scour depths could easily be accounted for with the 20 percent contingency factor applied to the total cost estimate.

It should be emphasized that these scour depths (6 feet and 8.5 feet) are not intended to be used for final design. A detailed, site-specific, scour analysis should be conducted for each channel reach as part of the final design effort for the channel system.

6.2 EQUILIBRIUM SLOPE ANALYSIS

Sediment transport analyses need to distinguish between short-term and long-term changes. Short-term changes are event specific and occur to some extent during each flood hydrograph. Referring to the preceding section, examples of short-term changes would be local scour, general scour, bend scour, bedform troughs, and to some extent, low-flow incisement. With the exception of low-flow incisement, any visible signs of these processes may be difficult to detect after the flow has subsided.

As discussed in the preceding section, short-term scour processes can usually be quantified with empirical and/or theoretical relationships. With the aid of the computer, general scour is frequently evaluated with mobile boundary, sediment routing models.

Precise prediction of long-term channel impacts can be much more elusive than their short-term counterparts because of the time-span involved and the numerous variables that impact long-term changes. Since lateral erosion of the channel banks will be controlled by the application of soil-cement (or a similar bank stabilization product), this study will only investigate the potential for long-term movement of the channel bed. This analysis will employ the concept of equilibrium slope to determine the long-term trend for aggradation or degradation of the channel bed.

Equilibrium slope is defined as the slope at which the sediment transport capacity of the channel is equal to incoming sediment supply. When these two quantities are equal, the channel bed will neither aggrade nor degrade. However, if the incoming sediment supply is greater than the transport capacity of the channel, aggradation will occur, as the channel attempts to steepen its slope to generate a higher flow velocity (and resulting sediment transport rate). Conversely, if the sediment supply is less than the channel transport rate, the channel bed will degrade in order to flatten its slope, resulting in a lower flow velocity and sediment transport rate. Ideally, flood control channels should be designed on the basis of this equilibrium slope.

A preliminary equilibrium slope analysis was considered important to this study to provide some confidence that the selected channel geometries, roughness values, and available ground slopes could combine to move the dominant sediment loads through the system, without causing any major long-term deposition. Should such deposition occur, the channel capacity might be jeopardized.

The first step in an equilibrium slope analysis is to determine the sediment supply to the channel. For ephemeral channels, the 5- to 10-year event is considered the dominant discharge most responsible for affecting long-term changes to the channel. The 10-year event was selected for use in this study. Accordingly, the sediment inflows quantified in this analysis are based on the 10-year discharge.

The sediment inflows computed for this analysis are based on a key assumption that the upstream washes are in a state of equilibrium. This is considered a reasonable assumption since the majority of the watershed is not presently developed to the extent that the natural supply of sediment is significantly disrupted. However, there may be some future, localized pockets of high density development that may effectively reduce the normal sediment supply to the natural washes. This possibility should be considered during final design.

A second basic assumption of equilibrium slope analysis is that the sediment supply used to determine equilibrium conditions for a specific channel reach be equal to the total supply of all sediment being delivered to that reach. This assumes that all sources of sediment inflow to a reach are at an equilibrium condition. This assumption simplifies the calculation of sediment inflows because it allows the use of the existing watercourse slopes to compute the hydraulic parameters and sediment transport rates of all incoming washes.

The equilibrium slope analysis utilized the following equation (Zeller-Fullerton, 1983) to compute both the sediment inflows (that is, sediment supply) and the sediment transport rate of the proposed channels.

$$q_s = 0.0064 \frac{n^{1.77} V^{4.32} G^{0.45}}{Y_h^{0.30} D_{50}^{0.61}} \tag{6.2}$$

where: q_s = Unit width sediment transport rate (cfs/ft)

n = Manning's roughness factor

V = Mean flow velocity (fps)

G = Gradation coefficient of bed material

Y_h = Hydraulic depth (ft)

D_{50} = Diameter of bed material particle for which 50 percent is greater by weight (mm)

This equation is based on a regression analysis of hydraulic and sediment transport data for sand bed channels in arid regions. The listed regression limits for D_{50} are 0.5 to 10.0 mm.

Bed-material data used for this study was taken from three sample locations upstream of the Outer Loop Highway alignment, north of the CAP. These samples were previously published in the report: *Concept Drainage Design, Outer Loop Highway North of the CAP Aqueduct*, Simons, Li & Associates, Inc., March 1989. Based on a review of the gradation curves for these three samples, a D_{50} of 2.0 mm, and gradation coefficient of 4.0, were selected for use in the equilibrium slope analysis. These values, which are typical of sand bed channels, were used for both the sediment supply calculations and the sediment transport rates through the proposed flood control channels.

The calculation sequence to determine equilibrium slope is an iterative procedure. The following steps describe how the procedure was applied to this study.

Step 1 - The approximate alignment of each channel was plotted on both a 1 inch = 1,000 feet, USGS 7.5 minute orthophoto, quadrangle, and on a supplementary 1 inch = 400 feet aerial photograph. Based on a visual review of these photos, the more prominent washes intercepted by each channel were identified and assigned an approximate bottomwidth (widths were roughly scaled from the photos).

Step 2 - Assuming a rectangular cross-sectional geometry, a discharge and associated sediment transport rate were computed for each intercepted wash at flow depths ranging from 0.25 to 5.0 feet. Discharges and velocities were computed with Manning's Equation at 0.25 to 0.5 foot intervals through the referenced depth range. An "n" value of 0.035 was used for the washes. The bed-slope was based on an average value computed from the quadrangle maps for all the washes intercepted by the flood control channel. Once the normal depth hydraulic parameters were determined, Equation 6.1 was used to compute the unit sediment transport rate. This unit value was then multiplied by the measured channel bottomwidth to obtain the total sediment transport rate for each depth increment.

Step 3 - The drainage boundaries for each sub-basin intercepted by the proposed channels were transferred from the HEC-1 model workmap to the aerial photographs. The natural washes identified in Step 1 were then referenced to their parent HEC-1 sub-basins. The HEC-1 models were then run for the 10-year event and the peak discharge from each sub-basin was identified. The depth/discharge tables (Step 2) were then scanned to find what constant depth of flow would be required in each wash to produce a cumulative discharge from all washes that would equal the peak discharge emanating from that specific sub-basin. The corresponding sediment transport rates associated with this flow depth were added together to get the total estimated sediment transport rate (sediment supply) being delivered to the channel from each intercepted sub-basin.

Step 4 - Each flood control channel was divided into several reaches, with the reach boundaries coinciding with the HEC-1 sub-basin boundaries. The equilibrium slope calculations were then initiated at the upstream end of the channel. The sediment supply to this first reach was equal to the total sediment transport rate computed for the corresponding HEC-1 sub-basin in Step 3.

The dimensions for the flood control channel were computed on the basis of conveying the 100-year event. The initial estimate for the channel slope was based on compatibility with the existing ground slope.

The ten-year discharge was then applied to the selected channel design and Manning's Equation was used to compute velocity, actual flow depth, hydraulic depth, and Froude Number. The appropriate hydraulic parameters were then applied to Equation 6.1 to compute the unit sediment transport rate for the channel. This unit value was then multiplied by the hydraulic width (measured at half the flow depth for a trapezoidal section) to get the total sediment transport rate for the channel. This transport rate was then compared to the computed sediment supply rate for this reach of channel. If the channel transport rate was lower than the supply rate, the channel slope was increased and the 100-year event channel dimensions were recomputed.

The ten-year event was then re-applied to this new channel section and the sediment transport rate was recomputed. For those cases where the channel transport rate was higher than the sediment supply rate, the same procedure is repeated, except the channel slope is decreased instead of being increased.

The above procedure is repeated until the sediment transport rate of the channel is equal to the computed sediment supply rate. The channel slope that makes these two values equal is the equilibrium slope for that channel reach.

Step 5 - The same approach is now applied to the next downstream reach of channel. The only difference becomes the change in the sediment supply to this downstream reach. The total sediment supply is now equal to that being supplied by the upstream reach of channel plus the additional sediment inflows emanating from the sub-basin intercepted by the present reach of channel. Accordingly, the sediment transport in the channel must now be increased to match this increased sediment supply.

This procedure is repeated for each segment of channel (being sure to make the correct adjustment to sediment supply from reach to reach) until the downstream end of the channel is reached.

As one proceeds downstream with the equilibrium slope calculations, it is important to be aware of a basic assumption that was used in this analysis. That assumption uses the dominant channel discharge (Q_{10}), from the total intercepted drainage area, to move the total intercepted sediment supply through a specific channel reach. No consideration is given to the possibility that intercepted, downstream sub-basins may deliver their dominant - discharge sediment supply to the main channel prior to the arrival of the peak dominant water discharge (from upstream areas intercepted by the channel) to that particular reach of channel. This is a complex problem that is unique to man-made interceptor channels that cut across the gradient of natural washes.

It is believed that these relatively small side inflows would lose substantial amounts of sediment transport capacity once they leave the small washes and enter the large cross-section of the man-made flood control channels. This theory is based on the supposition that these inflows would suffer a substantial loss of velocity (and hence sediment transport capacity) when exposed to the large cross-sectional area of the man-made channel.

To test this theory, two reaches of the Pima Road channel (Stations 250+40 to 237+50 and 126+90 to 78+00) were evaluated on the basis of side inflows occurring in the channel prior to the arrival of flows from upstream areas. The discharge from the side inflow was inserted into the flood control channel and Manning's Equation was used to compute the corresponding flow depth and velocities. These hydraulic parameters were then used to compute the channel's sediment transport rate. This transport rate was then compared to the rate of sediment supply from the side inflow sub-basin to see if the channel was capable of transporting this sediment supply without the benefit of additional upstream discharge. In both cases, this analysis revealed that the sediment inflow was three times greater than what the man-made channel could transport. Accordingly, it can be concluded that the use of the total intercepted upstream area water discharge to move the total intercepted sediment supply through a channel reach is not an unrealistic assumption. The possible conservative results provided by this assumption may cause the channel to have a slightly higher transport capacity than required. This slightly excessive transport capacity will help to minimize the potential for sediment deposition in the channel.

The final channel design will face another complication that could significantly influence equilibrium slope calculations. As discussed previously, low-flow culverts are recommended at periodic intervals along the channels in order to maintain a natural water supply to the vegetation community along downstream washes. These culverts will divert both water and sediment from the flood control channels. The exact

amount of such diversions is difficult to quantify. The loss of water and sediment from the main channels will reduce the sediment transport capacity of the channel and the sediment supply to downstream channel reaches. Such changes will obviously impact downstream equilibrium slope calculations. A quantitative assessment of this problem should be investigated during final channel design. All equilibrium slope calculations presented in this report assume the low-flow culverts are inoperable.

An objective of this study is to develop channel concepts that can be vegetated to blend with the natural desert environment. This has been pursued by using relatively high roughness values (0.045) for the preliminary channel analysis. However, the equilibrium slope calculations indicate that some reaches of channel are not capable of achieving equilibrium with these high "n" values. Accordingly, several reaches of channel will have to be designed with very sparse to no vegetation in order to keep the channel velocities high enough to transport the incoming sediment loads. Manning's "n" values as low as 0.018 were required in some channels to maintain an equilibrium slope.

A key assumption in the application of the equilibrium slope concept is the need for a stable pivot point around which the bed-slope may rotate. In natural river systems these pivot points might occur at bedrock outcrops. In man-made channels, such pivot points have to be constructed in the form of concrete or soil cement grade control structures. In the absence of such stable pivot points, there is no reference point to which the rotation of the bed-slope can be attached. As a result, there is no way to quantify the vertical bed movement that may occur as the channel seeks an equilibrium condition. In these cases, the equilibrium slope calculations become more of a qualitative tool than quantitative, that is, the calculations will predict trends towards aggradation or degradation but provide little insight on how the slope adjustment will be dimensioned within a specific reach of channel.

It is anticipated that there will be numerous grade control structures constructed as part of the recommended master drainage plan. Accordingly, stable pivot points should be available for use in designing the channel system around the equilibrium slope concept.

Tables 6.1 through 6.10 present the equilibrium slope calculations that were performed for the 35 miles of channel proposed in this report. These tables list the existing ground slope, the channel design slope, and the equilibrium slope for each reach of channel. All hydraulic and sediment transport parameters are also listed in these tables.

In reviewing these tables, the reader will note substantial variations in "n" values have been required to establish equilibrium in some channels. This occurs in those channel reaches that have natural ground slopes substantially less than the equilibrium slopes required for "n" values of 0.045. Since it is not practical to design the channel slope steeper than the existing ground, the only remaining mechanism available for increasing the transport rate was the channel roughness. Even with "n" = 0.018, equilibrium was not attainable for the upstream reach of the Deer Valley Road channel between Hayden and Scottsdale Roads.

Some concluding comments are warranted regarding the interpretation and use of the equilibrium slope analysis. First of all, it should be emphasized that an equilibrium slope develops over a long period of time, in response to a wide range of flow conditions which is simulated by the use of a single, dominant discharge. The actual bed-slope of the channel will probably oscillate around this theoretical equilibrium slope in response to the large variation in flood hydrographs that the channel will be exposed to over a long period of time. However, if the channel is designed at, or very near, the equilibrium slope, these oscillations should be minimized.

The numerous and complex assumptions required for an equilibrium slope analysis require that the computed slopes be viewed as possibly lying within a large confidence band, that is, reasonable changes in a few key assumptions might produce a dramatic impact on the computed equilibrium slope. As a result, equilibrium slope calculations should not be viewed as a precise design parameter. However, they do provide an important tool in helping the engineer to design a channel that should provide much more stable performance than one which totally ignores the importance of sediment transport.

6.3 DROP STRUCTURE REQUIREMENTS

Depending upon the differences between existing ground slope, channel design slope, and the equilibrium slope, drop structures may be required at periodic intervals along the proposed channels. In order to avoid excessive channel cuts, drop structures are utilized to raise the channel bed when the design slope (or equilibrium slope) is flatter than the existing ground slope. Figure 6.1 illustrates the typical soil-cement drop structure geometry that was used to generate cost estimates for this report.

Using the preliminary equilibrium slopes that were discussed in the preceding section, estimates can be made of the number of drop structures, and drop structure spacing, that might be required if the channels

were constructed to the equilibrium slopes listed in Tables 6.1 through 6.10. The equilibrium slopes listed in these tables are generally set to minimize the need for drop structures, that is, where possible, they are set near the existing ground slope.

The last three columns in Tables 6.1 through 6.10 list the following drop structure data.

1. **Total Vertical Drop Distance** - This is the total vertical elevation differential that will occur through a specific channel reach as a result of differences between the existing ground slope and the computed equilibrium slope. The distance is computed as:

   $(S_{EXIST} - S_{EQ})$ x (Channel Length)

2. **Number of Drop Structures** - This column lists the number of 3-foot high drop structures that will be required for a specified reach of channel. In some instances, this number is slightly high, since fractional numbers of drop structures are rounded up to the next higher whole number. This value is computed as:

   Total vertical drop ÷ 3, with fractional values rounded to the next higher number.

3. **Drop Structure Spacing** - This represents an average distance between drop structures, and is computed by dividing the channel reach length by the number of drop structures.

It should be understood that the drop structure data presented in Tables 6.1 through 6.10 is not intended for final design. This data was developed only to get a "feel" for possible drop requirements and to generate data for cost estimates.

7.0 IMPACTS TO CAP DIKES

An important consideration in the development of the regional drainage plan was the impact that such a plan might have on the inflow volumes to the CAP dikes. These dikes were originally designed by the Bureau of Reclamation on the basis of an undisturbed, natural, watershed drainage pattern. A Bureau memorandum, dated March 3, 1986, also states that the hydrology for the four CAP dikes (impacted by this regional drainage plan) was based on natural basin conditions, rather than forecast, future urban conditions.

In order to address the impact that the proposed regional drainage plan may have on the CAP dikes, a reservoir routing operation was developed for each dike and inserted in all of the HEC-1 models used for the hydrology analysis.

There are four CAP dikes that will serve as outfalls for the channel systems investigated in this study. Each of the dikes is labeled on Plates 2, 3, and 4. The dikes are located as follows.

- Dike 1 - Cave Creek Road to Tatum Boulevard
- Dike 2 - Tatum Boulevard to Scottsdale Road
- Dike 3 - Scottsdale Road to Pima Road
- Dike 4 - Pima Road to just west of 108th Street (Scottsdale)

Stage/storage relationships for each dike were taken from a Bureau memorandum dated September 29, 1987. These relationships were prepared in 1987 and reportedly reflect revisions due to golf course construction at the TPC and changes associated with the construction of Horse World (Horseman's Park).

As discussed in Section 3.0, a baseline HEC-1 model was developed to reflect existing watershed drainage patterns with future, forecast land-use densities. This model was used as a baseline condition for comparison with the models which include the proposed regional channelization plans. Such a comparison will identify any changes to the maximum water surface elevations, in the detention areas behind the four dikes, that may occur due to the proposed channelization system.

The baseline models used for this comparison reflect a drainage pattern delineation that is considered realistic of watershed conditions that existed in the spring of 1990. As previously discussed in Section 4.2.7, man-made alterations have disrupted natural drainage boundaries between CAP Dikes 2 and 3. These alterations are included in the baseline model.

Table 7.1 summarizes the detention basin performance of the four dikes when subjected to the 100-year, 24-hour storm. This table lists the existing baseline condition and the condition associated with installation of the proposed channelization system. The reservoir routing data is based on no releases from the dikes during the inflow hydrograph.

Due to a limitation in the HEC-1 program, the maximum stage and storage values are based on a total inflow period of 25 hours. The HEC-1 program is not capable of computing more than 300 hydrograph coordinates for any concentration point. When combined with the 5-minute computation selected for this analysis, the maximum traceable time base of the inflow hydrograph is: 5 x 300 = 1,500 minutes or 25 hours. With the SCS Type IIA rainfall distribution, the peak burst of rainfall occurs near hour 6 of the 24-hour storm. Accordingly, at the end of 25 hours, the inflow hydrographs to the detention basins are down to relatively small flows of a few hundred cfs or less. As a result, a very small volume of runoff is not accounted for in the data listed in Table 7.1. Also, pending changes to the Core North detention basins, and Core South channel system, may cause slight alterations to the data listed in Table 7.1.

The following comments summarize the data in Table 7.1.

- **Dike 1** - Although the peak discharge to this basin increases when the regional plan is in-place, the maximum stage and storage characteristics are essentially unchanged. The Bureau of Reclamation lists a maximum water surface elevation of 1,550.2 feet MSL for this basin, based on the Temporary Designers Operating Criteria (DOC).

- **Dike 2** - Again the peak discharge to the basin was found to increase when the regional plan is installed. However, the maximum stage and storage values show a slight decrease. This decrease is probably due to the Pima Road channel diverting some of the natural flow from CAP Dike 2 to CAP Dike 3.

  The DOC maximum water surface elevation for Dike 2 is 1,550.2 feet MSL.

- **Dike 3** - If the Reata Pass flows are all assumed to go to CAP Dike 4, there is an increase in both stage and storage when the regional plan is in place. As discussed for Dike 2, this occurs in response to a diversion of flow from Dike 2 to Dike 3 by the proposed Pima Road channel.

  The same trend exists for the condition where all of Reata Pass flows are assumed to go to the southwest to Dike 3. Again, the Pima Road channel is responsible for the increase in stage and storage when the regional plan is assumed to be in-place. The large decrease in peak inflow to CAP Dike 3 (compared to the existing drainage pattern condition) is due to the storage effects of detention basins 38R-2 and 53R (Core North).

  The DOC maximum water surface elevation for Dike 3 is 1,544.4 feet MSL.

- **Dike 4** - There is essentially no change to the reservoir performance, with and without the regional plan in-place, when the Reata Pass flows are assumed to go south to Dike 4. The same trend holds true for the case that assumes the Reata Pass flows are directed to Dike 3. However, there is approximately a 3.5 foot difference in reservoir stage when comparing the with and without Reata Pass diversion scenarios.

  The DOC maximum water surface elevation for Dike 4 is 1,539.0 feet MSL.

8.0 INTERIM SOLUTIONS

To the maximum extent practical, an important goal of this study is to identify drainage improvements that might be used on a temporary basis, until the complete regional plan can be installed to final specifications. These temporary, or interim solutions, would ideally be economical to install and have some salvage value that would allow them to be incorporated (to some degree) into the final regional plan. However, as interim solutions, they would not be built to repeatedly sustain exposure to flooding without requiring extensive repairs between flood events. As a result, the risk of damage to improved properties would be greater while the interim solutions are in-place than it would be after the regional plan is installed.

Identifying interim solutions for this project is a very difficult task. The disadvantages associated with such solutions nearly negate any benefits that they might generate. The following paragraphs discuss major components of the proposed regional plan and their potential use as interim solutions.

1. **Interim Channels** - Since the main component of the proposed regional drainage plan is a channelization system, interim channel concepts would be a logical objective to pursue. Substantial construction costs would be delayed if the channels were excavated along their proposed alignments, but not provided with bank lining until some future date during construction of the final regional plan. Although this may seem to be a tempting concept, the pursuit of such an approach would probably create more problems than would be solved. Construction of the proposed interceptor channels, without bank protection and cutoff walls along the top-of-bank, would undoubtedly lead to severe bank erosion and probable channel migration. An equally disastrous consequence would be the headcutting generated along the top of the channel banks. Headcuts would occur in response to the lateral channel inflow of surface runoff. Such headcutting could propagate substantial distances upstream and cause damage to private property and roads. The material that is eroded from the channel banks and headcut areas would cause serious sediment deposition in the channels, which could adversely effect the channel capacity.

   In summary, the temporary lack of bank protection and grade control structures would create a totally unstable channel system which could lead to substantial property damage,

due to unchecked bank erosion and headcutting action. The construction of the proposed channel system should only be done to its full specifications, which includes bank protection and any required grade control structures. An interim channel solution without these measures is not recommended.

2. **Detention Basins** - The three detention basins included in the proposed Core North drainage system are a critical element in controlling the peak discharges through portions of the Outer Loop Highway and into the Core South channel system. These basins must be in-place and be capable of providing the required detention storage and reduced outlet discharges before any upstream channelization takes place. It is difficult to envision any partial construction scenario of these basins that would allow them to operate at their design capacity. However, until the upstream channels are constructed, the basins will have no control over the capture of runoff from the existing upstream drainage pattern. Without such control, large amounts of runoff may bypass the basins and defeat their purpose of collecting and attenuating upstream runoff. Accordingly, until the upstream channel system is in-place, an interim solution might be to construct temporary earth training dikes across the desert to collect and funnel water into the detention basins. Such dikes would probably take the form of an earth berm 3 or 4 feet high, with a topwidth 10 to 12 feet to support vehicular traffic. As a cost saving feature, the berms would not be bank protected. As a result, they would be prone to bank erosion and probably require substantial maintenance.

   Disadvantages to such an approach would be possible severance of private property and the certain need for a temporary right-of-way permit, as well as the unpleasant visual scar that such a berm would create across the desert.

3. **Outer Loop Highway Crossings** - The proposed regional channelization system will have a major impact on the off-site drainage system required for the Outer Loop Highway. The proposed channel system will collect and direct concentrated flows to a very small number of box culvert or bridge crossings along the highway alignment. However, should the Outer Loop be constructed prior to completion of the regional drainage plan, some type of interim measure will have to be in-place to direct floodwaters to the specified highway crossings. In the absence of an upstream channel system, this interim

solution would most probably take the form of a parallel channel system along the north side of the Outer Loop. Failure to construct this channel, or the regional channel system, would cause floodwaters to simply pond along the Outer Loop roadway embankment until the water could discharge through the designated culvert locations proposed as part of the regional plan. Unless the highway embankment were designed to be high enough to prevent overtopping, and possible failure due to embankment saturation, the Outer Loop would be exposed to an almost certain unacceptable risk. Accordingly, the only probable interim measure would be to construct the parallel interceptor channel along the upstream (north) side of the Outer Loop. In order to protect against the 100-year event, such a channel would have to be much larger than its counterpart with the upstream regional channel system in-place. This will require additional highway right-of-way beyond that needed if the regional plan were in-place.

An awkward variation of this parallel interceptor channel concept might be to construct long training dikes across the desert to intercept overland flows and funnel them to the designated highway culverts. These V-shaped interceptor dikes would probably create numerous right-of-way problems and visual scars, such as those previously referenced for the training dikes leading to the Core North detention basins.

The preferable interim solution for the Outer Loop would seem to be the parallel interceptor channel along the north side of the highway.

4. **Partial Construction of Reata Pass Channel** - Construction of the Reata Pass channel to CAP Detention Basin 4 is a very critical element in the design of the lower portion of the Pima Road channel and Core North Detention Basin 53R. Until the Reata Pass channel is in-place, the Pima Road channel, and basin 53R, will have to be designed on the assumption that 100 percent of the Reata Pass flow will travel southwest and be intercepted by the Pima Road channel.

However, an interim solution might be to construct the upper portion of the Reata Pass channel far enough to the south so that the channel would discharge to a natural swale leading to CAP Dike 4. In the absence of temporarily outletting to a natural swale, the channel could be continued far enough south to physically prevent any discharge from the

channel from having any possible path to Pima Road, prior to being captured by CAP Dike 4.

9.0 COST ESTIMATE

The estimated installation cost of the proposed regional drainage and flood control plan has been developed by subdividing the construction effort into several different categories. Costs were then generated for each category.

As emphasized in previous sections of this report, there are still major decisions to be made that will have a significant impact on the construction cost. Most notable of these decisions is the treatment of the Reata Pass channel, that is, will or will not, the Pima Road channel and Detention Basin 53R be constructed to accept Reata Pass flows. A second important decision relates to the treatment of low-flow culverts, that is, how many will actually be needed and will the channel design (and subsequent costs) reflect a loss of water through such culverts.

Channel costs, including right-of-way acquisitions, could be significantly influenced by the final sediment transport analysis. This analysis will be influential in establishing the channel slope, which will have a direct bearing on the channel width and associated excavation costs and right-of-way requirements. The sediment transport analysis will also determine the location and number of required drop structures in each channel. Also, as indicated in Section 4.2.10, the final alignment of the Reata Pass channel has not yet been determined, although a recent, independent planning study completed by the City of Scottsdale has identified a tentative alignment that follows natural drainage channels west of the proposed Thompson Peak Parkway. Accordingly, the costs for this channel could change substantially from those listed in this report.

In consideration of all these variables, multiple cost estimates have been developed to establish probable envelopes that hopefully approximate the maximum costs that would reasonably be required for the regional drainage system. Table 9.1 is a matrix that summarizes the different assumptions used for each cost estimate scenario.

Standard unit cost assumptions, that are common to all the scenarios listed in Table 9.1, are summarized as:

1. Channel Excavation - $2.00/cy
2. Soil-Cement Bank Protection (in-place) - $24/cy
3. Soil-Cement Drop Structures (in-place) - $27/cy
4. Bridges - $45/sf of Deck Area
5. Low-Flow Culverts - 36-inch RCP @ $78.57/lf (in-place), two - concrete headwalls (inlet and outlet) @ $1,950 each, based on ADOT structural detail C-14.20, with in-place concrete at $300/cy
6. Channel access ramps - $2,500 each (placed at 0.25-mile intervals)
7. Channel right-of-way @ $50,000/acre
8. Channel landscaping @ $7,500/acre

Since the proposed regional drainage and flood control plan incorporates, or relies on, certain elements of drainage plans being developed by other parties, it is important to define what drainage features are, or are not, included in the cost estimates presented in this report. Accordingly, the following assumptions were used for estimating the cost of the regional plan.

1. Costs <u>are not</u> included for any culvert or bridge crossings of the Outer Loop Highway. These costs are assumed to be part of the highway construction cost.

2. Costs <u>are not</u> included for any of the <u>interior</u> channels in Core North. Costs <u>are</u> included for all of the perimeter channels around Core North, that is, Pima Road, Deer Valley Road, and Scottsdale Road.

3. Costs <u>are not</u> included for the three detention basins located within Core North.

4. Costs <u>are not</u> included for any special outlet structure requirements to the CAP detention basins. These costs will be absorbed as part of the 20 percent contingency factor.

5. Costs <u>are not</u> included for training dikes at the upstream end of the Rawhide Wash channel. These costs will also be absorbed by the contingency allotment.

6. Costs <u>are not</u> included for any of the small drainage channels that may be required along the upstream side of the Outer Loop Highway embankment.

Summary cost tables for all the construction scenarios are presented in Tables 9.1, 9.2, 9.3, and 9.4. These four tables are based on all the unit cost assumptions and exclusions listed in the preceding paragraphs.

In reviewing these tables, it should be noted that Tables 9.4 and 9.5 do not include any costs for the Reata Pass channel. These two scenarios assume that Reata Pass will be left in a natural condition and not channeled to CAP Dike 4. Accordingly, no costs are shown for the Reata Pass system in these two tables. However, these two scenarios do assume that 100 percent of the Reata Pass flows will be intercepted by the Pima Road channel. As a result, Tables 9.4 and 9.5 do reflect the additional costs for the Pima Road channel to convey the Reata Pass flows.

10.0 DISTRIBUTION OF INSTALLATION COSTS

The large cost associated with the regional drainage and flood control plan will be difficult to absorb by any single entity. Some type of cost sharing arrangement that would distribute the costs among multiple agencies (both government and private) would be a preferred alternative. Such an arrangement would not only lessen the financial burden on any single agency, but would also expedite the availability of funds to the project, which would hasten construction of the drainage system.

The most probable governmental participants in a cost sharing agreement would be:

- City of Scottsdale
- City of Phoenix
- Flood Control District of Maricopa County (FCDMC)
- Arizona Department of Water Resources (ADWR)
- Arizona Department of Transportation (ADOT)

All five of these agencies are members of the Task Force for the development of the *Upper Indian Bend Wash Regional Drainage and Flood Control Plan.*

Participation by the cities of Scottsdale and Phoenix would essentially be mandatory, since the project lies within the boundaries of these two municipalities. Both cities would probably seek some type of equitable distribution of taxes to help fund their portion of the project cost. This may include the formation of improvement districts and possible stipulations on future development to incorporate portions of the regional plan into their drainage system.

Several options are available to the cities to achieve an equitable distribution of taxes to fund the cities portion of the installation cost. Possible options would include:

1. Tax assessment based on the volume of runoff generated by individually owned parcels of land. This would appear to be an equitable approach that would prorate the cost on the basis of how much each landowner contributes to the flooding problem.

2. Tax assessment based on property evaluations. A disadvantage to this approach is that there is no correlation with the source of the flooding problem, and not necessarily any correlation with flood control benefits to specific parcels, that is, some very expensive property might pay a high tax assessment yet not be located in a flood prone area.

3. Tax assessment based on receipt of flood control benefits. This could be a very difficult option to apply. Detailed floodplain delineations would have to be prepared and then property damage estimates would have to be compiled for each parcel of land. These damage estimates would then have to be compared to post-project flooding potential to determine the damage reduction benefit. Some type of formula would then have to be devised to prorate the project costs according to the computed damage reduction.

   The computation of flood control benefits could be complicated further by the fact that flood control benefits would not necessarily be limited to property damage, but might also include such issues as an increase in developable land and reductions in flood insurance rates.

4. Tax assessment based on the area of land ownership. This is somewhat similar to Option 1, but does not address the amount of water contributed from each parcel to the overall flooding problem. Under this scenario, each landowner would contribute the same amount of money per acre without consideration of the runoff characteristics of the land.

Cost sharing by ADOT would most probably be based on the amount of money saved on the off-site drainage system for the Outer Loop. This savings would occur as a result of eliminating numerous culvert installations that would be required if an upstream channelization system were not in-place. Such a cost savings (and project cost contribution) could be quantified by comparing the cost of a pass-through drainage system, for existing watershed conditions, to the system required with the regional drainage and flood control plan in-place.

ADWR has historically administered flood control assistance programs for projects throughout Arizona. The state legislature has previously appropriated funds to ADWR for cost sharing on specific flood control projects. Such an appropriation would almost certainly be required for ADWR to participate in the construction funding of the Upper Indian Bend Wash Regional Drainage and Flood Control Project.

The amount of such an appropriation would have be negotiated with ADWR and legislative representatives.

The Flood Control District of Maricopa County has a primary purpose of sponsoring and coordinating flood control projects within Maricopa County. The District has historically contributed millions of dollars towards the installation of federal and local projects in the county. Accordingly, they would be a logical co-sponsor and funding contributor to the proposed regional drainage plan.

A proposed distribution of costs is beyond the scope of this report. However, a cost-sharing arrangement will have to be developed if the regional plan is to be constructed in a reasonable time frame.

The preceding comments identify potential funding sources and possible ways to distribute costs among area landowners. These suggestions should be expanded into meaningful discussions with all interested parties in an effort to develop a cost-sharing agreement.

11.0 INSTALLATION SCHEDULE

Until a reliable source of funding is identified, it is difficult to establish an installation schedule that can be referenced to a specific starting and ending time. However, the proposed construction schedule for the Outer Loop Highway should serve as a catalyst for initiating work on the proposed regional drainage and flood control plan.

Current ADOT forecasts call for construction to begin on the interim roadway section of the Outer Loop, from Scottsdale Road to Bell Road, in July 1993. This section of roadway should be completed in December 1994. Since the cross-drainage system for this reach of the Outer Loop is dependent upon certain portions of the regional drainage plan (or some type of interim measure) being in place, it would seem reasonable to develop a schedule that would prioritize construction of the three Core North detention basins, the Pima Road channel and the Deer Valley Road channel. Due to the dependence of the Pima Road channel and detention basin 53R on the construction of the Reata Pass channel, equal priority will have to be placed on installation of the Reata Pass channel. If Reata Pass is not channelized, then basin 53R and the lower portion of the Pima Road channel would have to be over-designed to receive possible inflows from Reata Pass.

The remaining elements of the regional plan, that is, the Squaw Peak-Jomax Road-Pinnacle Peak channel system, and the Rawhide Wash - Scottsdale Road system, each drain separate watersheds and could be constructed at anytime prior to the Outer Loop being extended through their respective watersheds. Actually, the Outer Loop is not dependent upon prior construction of the Squaw Peak-Jomax-Pinnacle Peak Road system, because the inflow points to this western section of the Outer Loop are being controlled by drainage improvements to be built as part of the development of Desert Ridge.

Figure 11.1 presents a possible installation sequence for the regional drainage and flood control plan. Certainly this schedule should only be considered as a rough approximation of what may ultimately occur. If adequate funding is not forthcoming, the schedule would be subject to considerable change. Right-of-way negotiations may also have a significant impact on the timing of construction.

12.0 RECOMMENDATIONS FOR FINAL DESIGN

As emphasized throughout the previous technical discussions, the information in this report is not intended to represent a final engineering analysis from which construction drawings would be initiated. This report presents a concept drainage plan with a limited engineering evaluation designed to identify approximate structure locations, sizes, costs, and technical problems expected to be encountered during final design.

It is recommended that the proposed channels be broken into finite segments and that separate design contracts be awarded for each segment. These design contracts would include a final engineering analysis as well as preparation of construction drawings and specifications.

In order to avoid confusion, design conflicts, and technical review problems, it is highly recommended that a standardized set of engineering guidelines be prepared as part of the scope of work for each channel segment. This will be especially critical for the sediment transport and scour portion of the final engineering analysis. Failure to specify standard procedures to use for such complex analyses will lead to a potpourri of methodologies and design assumptions that will undoubtedly lead to vastly different design conclusions from one consultant to the next. Designing an integrated channel system under such a non-standard approach would almost certainly be an invitation to poor hydraulic performance.

In order to provide some assistance for coping with this problem, an extensive list of critical design issues is outlined in the following subsections of this report. This list is not intended to be all-inclusive, that is, many refinements and additions will most probably be added to the list prior to arriving at a final set of engineering design guidelines for this project.

12.1 HYDROLOGY ANALYSIS

The HEC-1 models used to establish design discharges and hydrographs for this project are considered to be suitable for final design of the regional drainage system. These models have evolved from a series of three previous drainage studies prepared for this watershed. Several revisions and improvements have been made to the models from one study to the next. However, depending on final design requirements, there may be a need for minor adjustments of concentration points, or drainage area boundaries, to provide estimates of inflows at different locations along the proposed channel alignments.

The City of Scottsdale has also expressed an intent to update these models to reflect the recent hydrologic assumptions that were submitted to FEMA as part of the appeal process for the Flood Insurance Studies for this region. Detention basin and channel alignment revisions will also be required to reflect pending changes to the Core South and Core North drainage systems.

The divert routines used for the low-flow culverts should also be refined during final design. If future land-use changes occur that are different from the forecasts provided by the cities of Phoenix and Scottsdale, the HEC-1 models may require adjustments to the percents of impervious cover used to simulate future development. It should be reiterated that all hydrologic modeling used for the analysis of the regional drainage plan is based on future land-use conditions.

12.2 HYDRAULIC AND SEDIMENT TRANSPORT ANALYSIS

This phase of the final engineering analysis will require numerous refinements to the work presented in this report. Important issues are outlined as follows.

- Current, detailed topographic mapping must be obtained along each channel alignment. The only continuous mapping available for this concept study was 1 inch = 2,000 feet (10 feet and 20 feet CI) USGS quadrangle maps.

- Computerized backwater profiles must be developed for each channel. HEC-2 models should be required for this purpose. Only normal depth calculations, using Manning's Equation, were used for this concept study.

- Sediment transport studies should be conducted for both long-term and single-event conditions for all channel reaches. Long-term analyses should be conducted to determine an equilibrium slope for each channel segment. This analysis will also identify the need for, and location of, drop and/or grade control structures. Equilibrium slope analyses should utilize a dominant discharge, such as the 10-year event, to compute a stable channel slope for use in the final channel design. A single-event sediment routing model should be run for each channel in order to identify the sediment transport and hydraulic characteristics during passage of the hydrograph for the 100-year event. Any problems, such as substantial sediment deposition, should be noted and corrected.

All sediment transport studies, both long-term and single-event, should be based on site-specific sediment data. Detailed field inspections should be made to identify all intercepted washes and their sediment transport characteristics.

Sediment transport analyses should also examine potential problems associated with partial channel flows that may occur if a storm only occurs over part of the intercepted watershed. The analysis should focus on the channel's ability to carry the incoming sediment loads under such conditions.

- A site-specific scour analysis should be performed for each reach of channel. This analysis, which must be closely coordinated with the hydraulic and sediment transport studies, should quantify the following phenomena.

    1. Long-term aggradation/degradation
    2. Local scour
    3. General scour
    4. Bend scour
    5. Low-flow incisement
    6. Bed-form troughs

- The design of the three detention basins in Core North should include a sediment inflow analysis in order to establish a dead storage allotment for sediment accumulation.

- Detailed criteria needs to be established for the design of low-flow culverts in the proposed interceptor channels. This criteria should address the following issues.

    1. The size of washes that will receive low-flow releases from the channels.

    2. What maximum discharge will be allowed through each low-flow outlet?

    3. How will the low-flow outlets be stabilized to ensure a reliable stage/discharge rating?

4. What low-flow outlet discharge, if any, will be used for the hydraulic and sediment transport analysis of the channels?

- In order to preserve the esthetics of the watershed, natural landscaping has been recommended for incorporation into the interceptor channel designs. The density of vegetation within the channel cross-sections will be dependent upon the sediment transport capacity required to move the sediment inflows through the channels. As was demonstrated in Section 6.2 of this report, some channel sections may have to have very little to no vegetation in order to maintain sufficiently high velocities to transport the incoming sediment load. Accordingly, the channel landscape design will have to be closely coordinated with the hydraulic and sediment transport analysis of each channel.

12.3 MISCELLANEOUS DESIGN RECOMMENDATIONS

- Bridge locations used for this study are very preliminary. Final locations will have to be determined during final channel design.

- Soil-cement has been recommended as a preferred lining material for the channel banks. The cost estimates presented in this report assume each channel bank would have a horizontal lining thickness of 8 feet. Recent advances in the placement of soil-cement bank lining suggest that narrower horizontal thickness may be possible. The physical capability to place soil-cement, on 2:1 sideslopes, at less than 8-foot widths should be investigated. The durability of soil-cement bank lining at any reduced width (less than 8 feet) should also be verified prior to using such a width in the final channel design. Mr. Paul Mueller, Materials Laboratory, Civil Engineering Department, Arizona State University, should be contacted for further information on this issue.

- At the time this report was published, details on the design of the three Core North detention basins had not been completed. The design of these basins is not part of the scope of work for this study, although some technical analyses for basin 53R was performed as part of this study.

Some of these detention basins may require embankment heights which will classify the structures as dams. Should this occur, a dam safety analysis will be required by ADWR.

Also, since it appears imminent that the outlet structures for basin 53R, and possibly basin 38R-2, will be changed from that previously used to analyze the Core South channel system, it is recommended that the hydrologic model for the Core South channel system be re-evaluated to insure that timing differences in the Core North detention basin outflow hydrographs will not cause any increase in the peak design discharges for the Core South channels. This re-evaluation of the Core South Channel system should also quantify changes in the design discharges that will almost certainly occur as a result of some recent channel alignment changes through the Core South system. The most notable of these changes is the shifting of the Hayden Road outlet from detention basin 53R to a point located a substantial distance east of Hayden Road.

- All final design studies should include close coordination with ADOT's design efforts for the Outer Loop Highway, and with planning efforts that may be underway for the numerous state trust lands that are located in the study area.

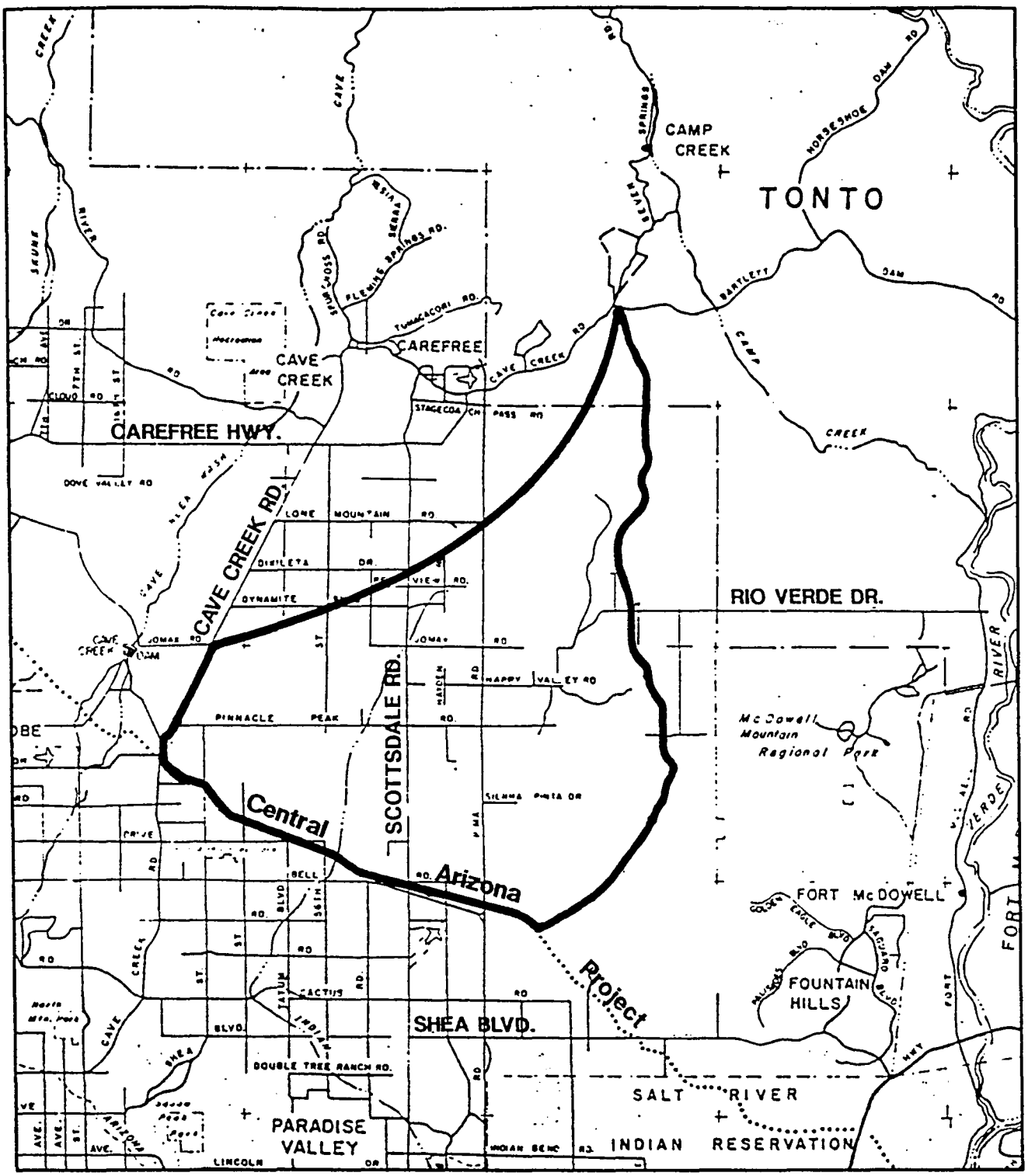
- Close coordination should be maintained with the Bureau of Reclamation to insure that the channel outlets into the CAP detention basins are placed at acceptable locations. The channel outlets should also be designed to insure that adverse backwater and scour conditions are not initiated at the outlet points. Any un-natural transfers of runoff between CAP basins should be avoided. If such transfers cannot be avoided, they should be referred to the Bureau for review and approval.

- Final design of all channel segments should include an updated estimate of the channel installation cost.

- It is not known what hydrology data was used by BRW, Inc. for the design of the Desert Ridge culvert and bridge crossings of the Outer Loop Highway. There is a high probability that those structures were not designed on the basis of the HEC-1 models developed for this study. It is recommended that this issue be investigated so that any

possible inconsistencies with this study can be rectified, if deemed appropriate by ADOT or the City of Phoenix.

FIGURES



4 0 4 MILES

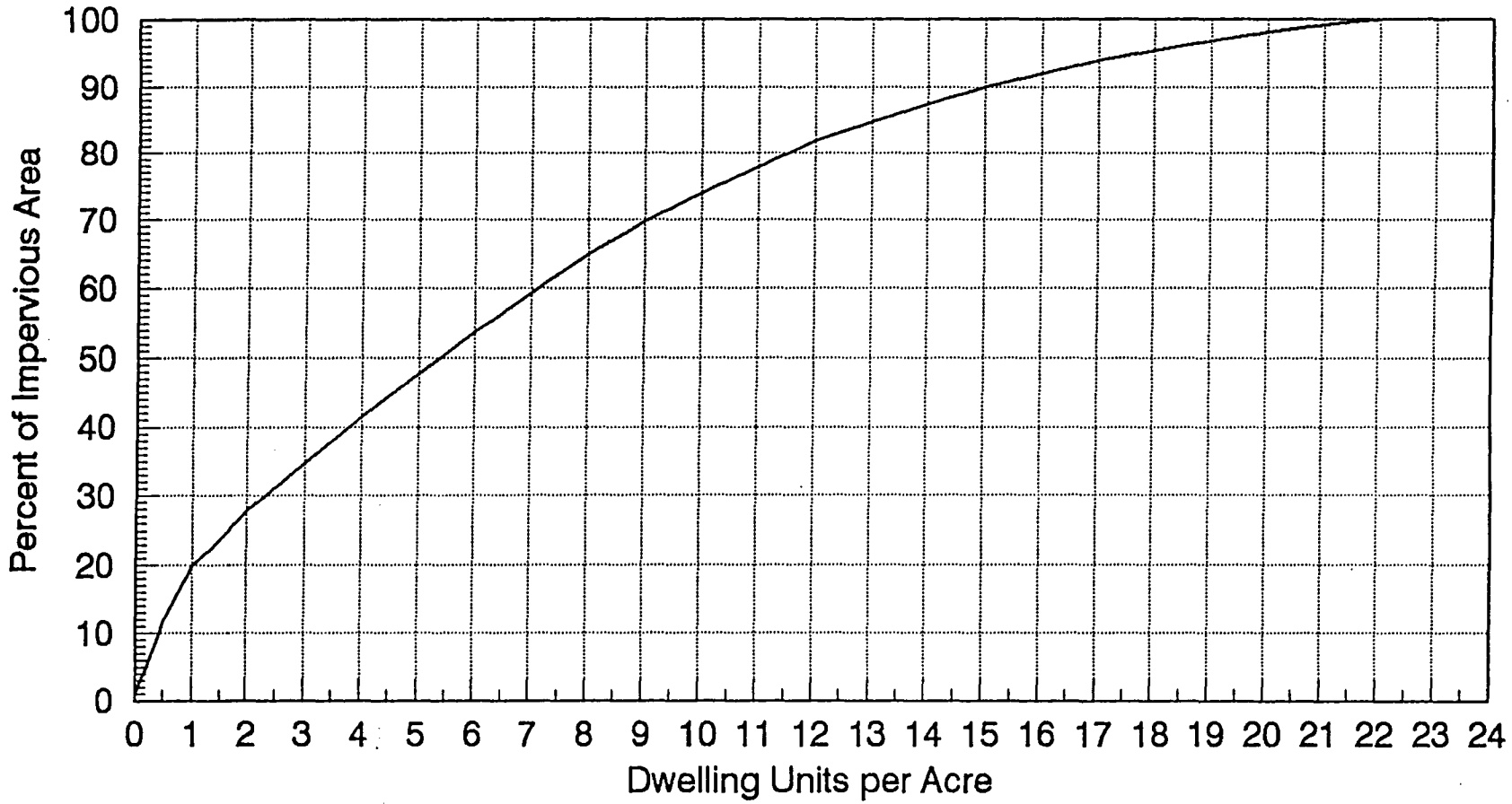
LEGEND

Study Boundary

BASE MAP : Maricopa Co. Dept. Of Planning And Development

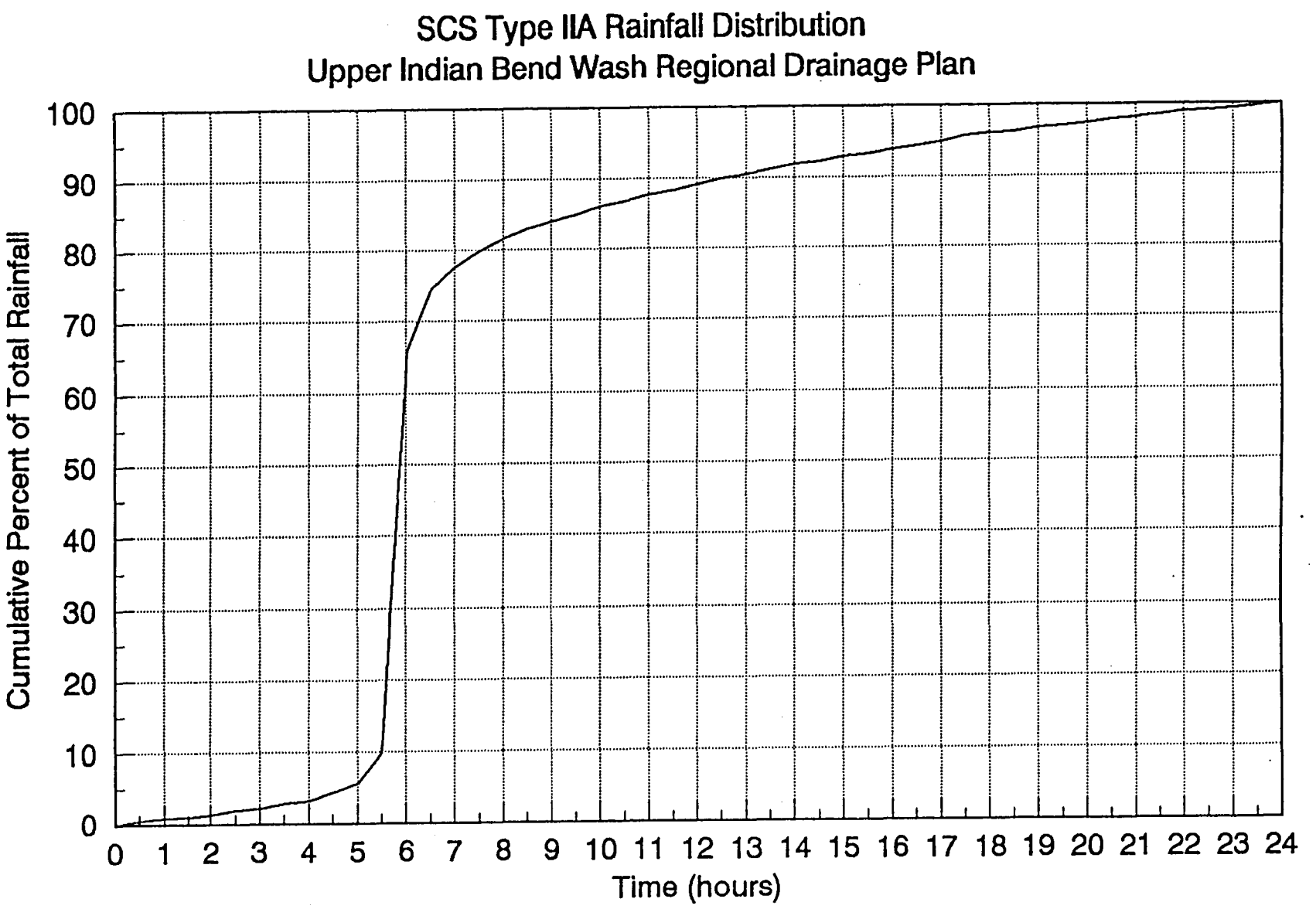
Water Resources Associates, Inc.	STUDY BOUNDARY	FIGURE 2.1
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists		
PROJECT NO. AC02670 / DRAWN BY KMA		
DATE 8/31/90 / CHECKED BY JJH		

Percent of Impervious Area vs Dwelling Density
Upper Indian Bend Wash Regional Drainage Plan



This curve was developed from data in Table 2.2a of TR-55, Urban Hydrology For Small Watersheds, and from discussions with Scottsdale city staff.

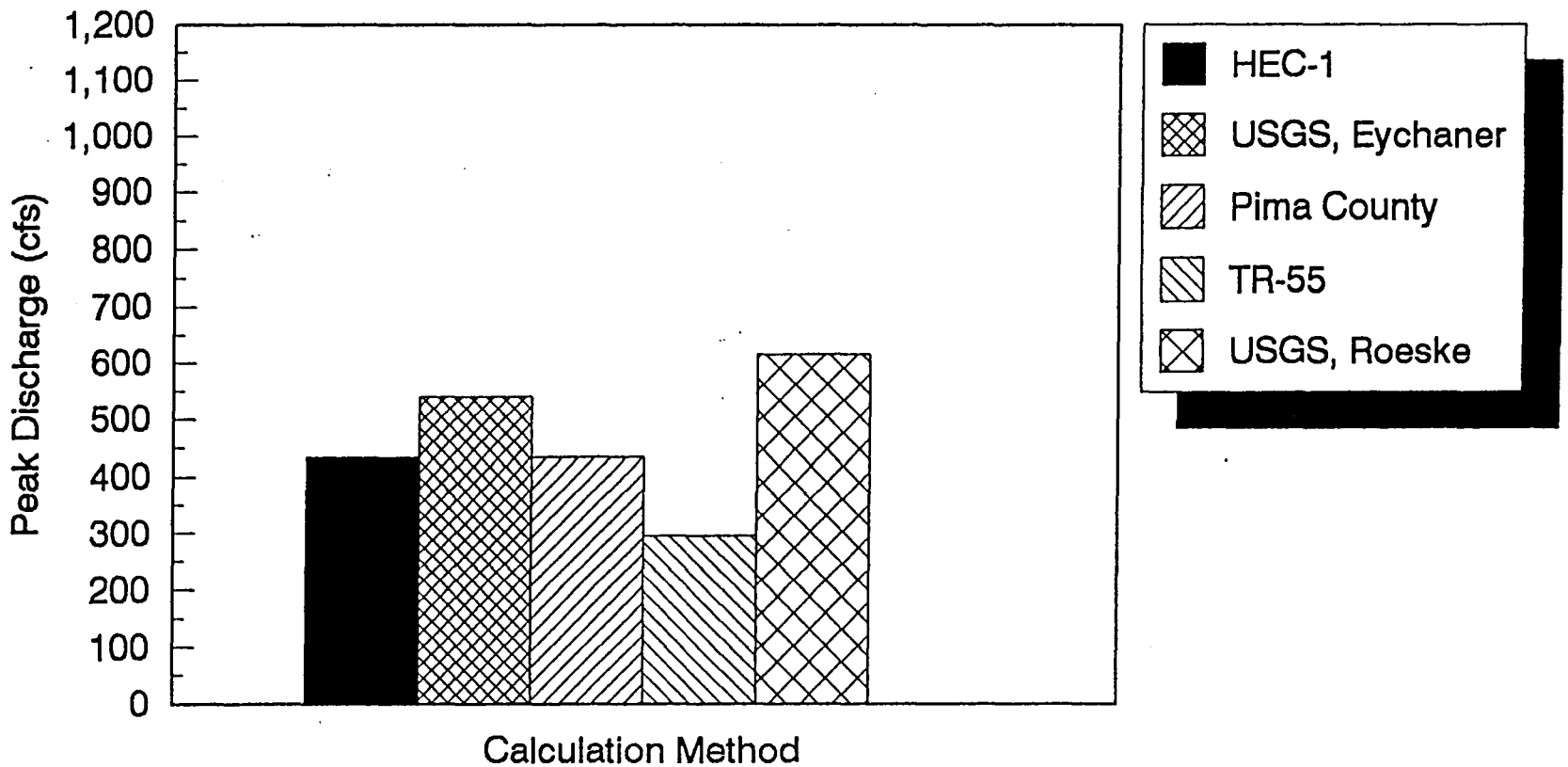
Water Resources Associates, Inc.	PERCENT OF IMPERVIOUS AREA VS. DWELLING UNITS/ACRE	FIGURE 3.1
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists		
PROJECT NO. AC02670 / DRAWN BY KMA		
DATE 8/31/90 / CHECKED BY JJH		



Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF SCS TYPE IIA RAINFALL DISTRIBUTION	FIGURE 3.2

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3020

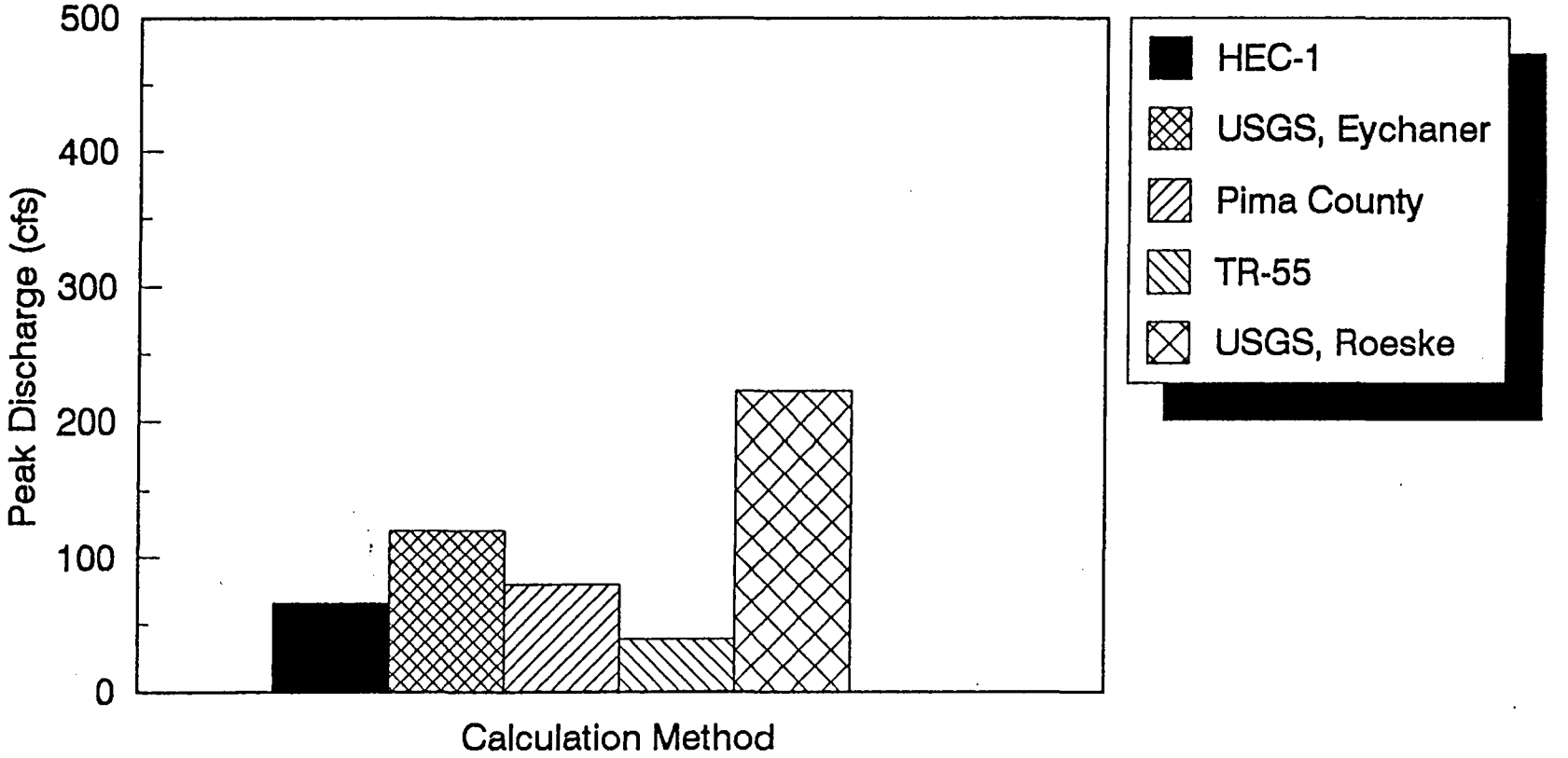


Drainage Area = 0.31 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3020	FIGURE 3.3

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3135

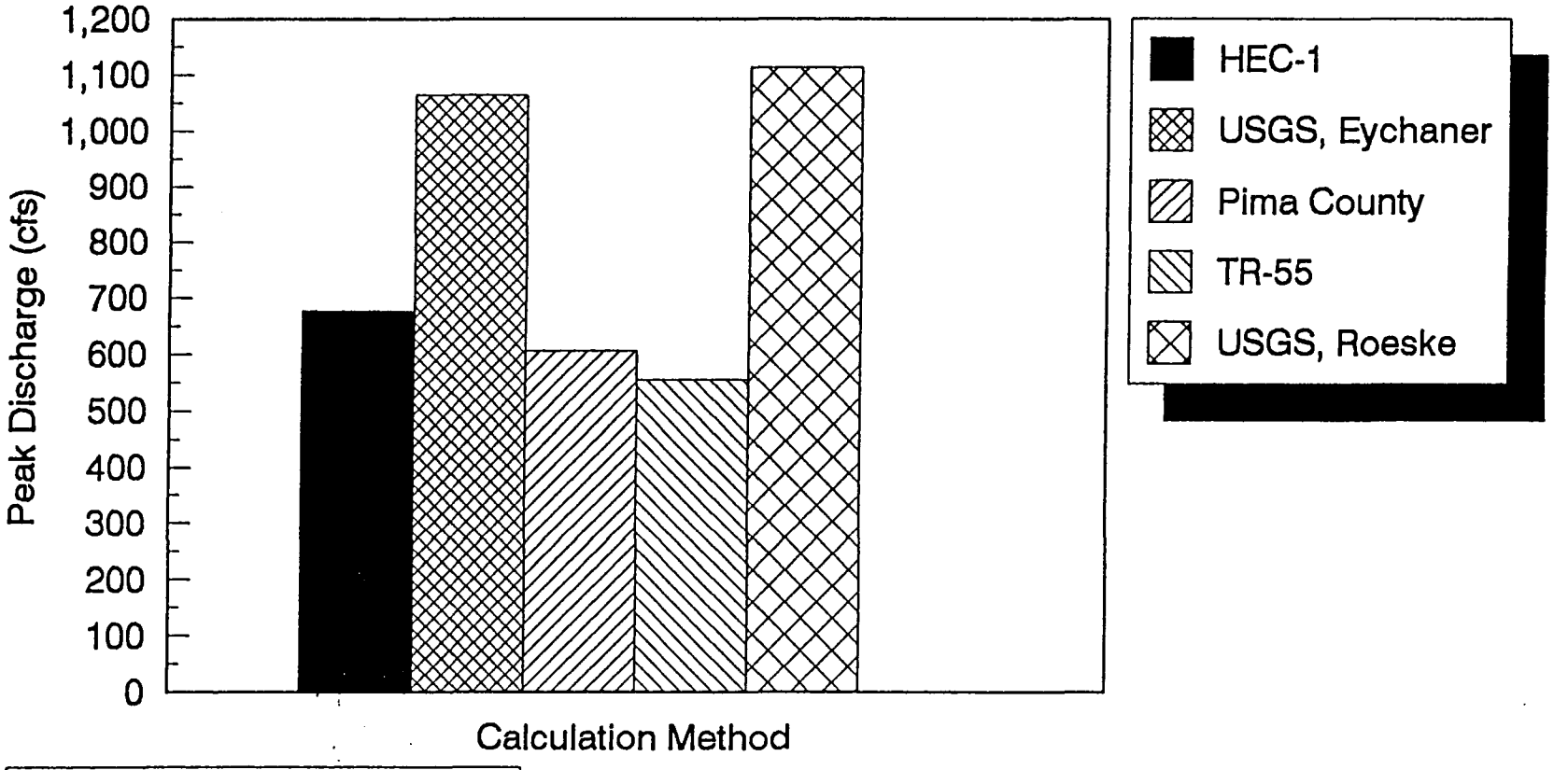


Drainage Area = 0.04 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3135	FIGURE 3.4

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3150

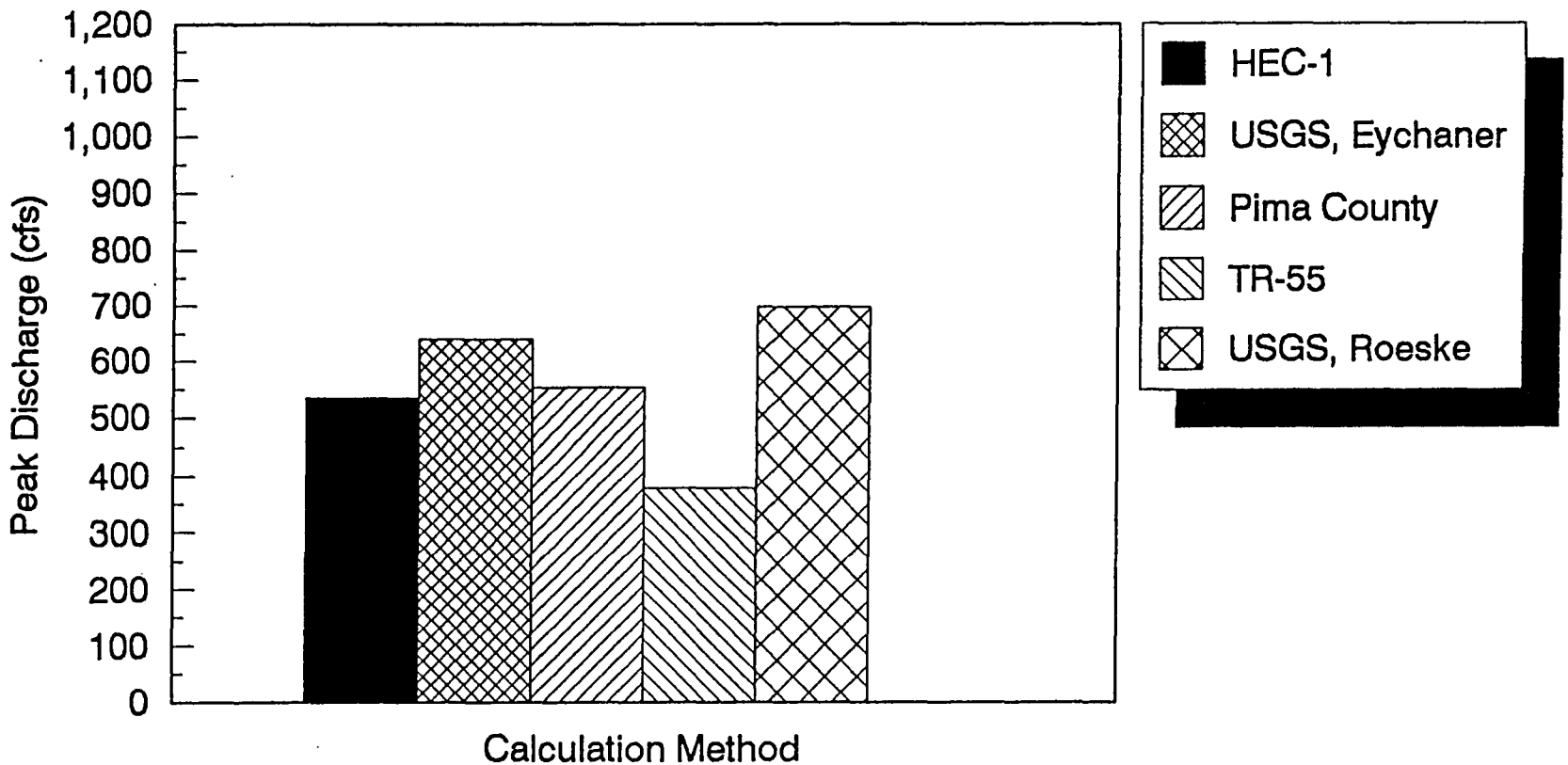


Drainage Area = 1.02 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc.	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3150	FIGURE 3.5
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists		
PROJECT NO. AC02670 / DRAWN BY KMA		
DATE 8/31/90 / CHECKED BY JJH		

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3160

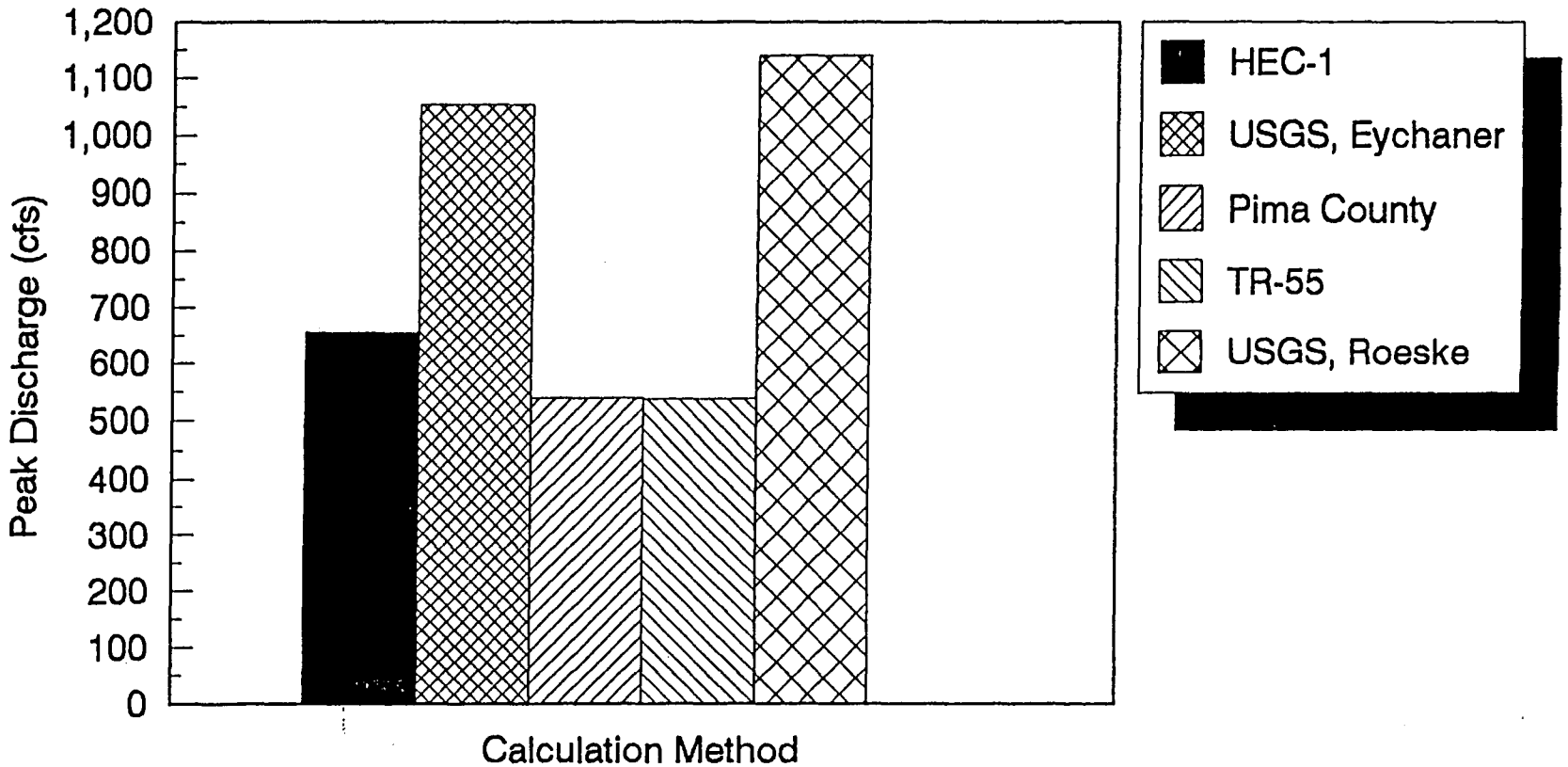


Drainage Area = 0.40 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists		GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3160	FIGURE 3.6
PROJECT NO. AC02670	DRAWN BY KMA		
DATE 8/31/90	CHECKED BY JJH		

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3400

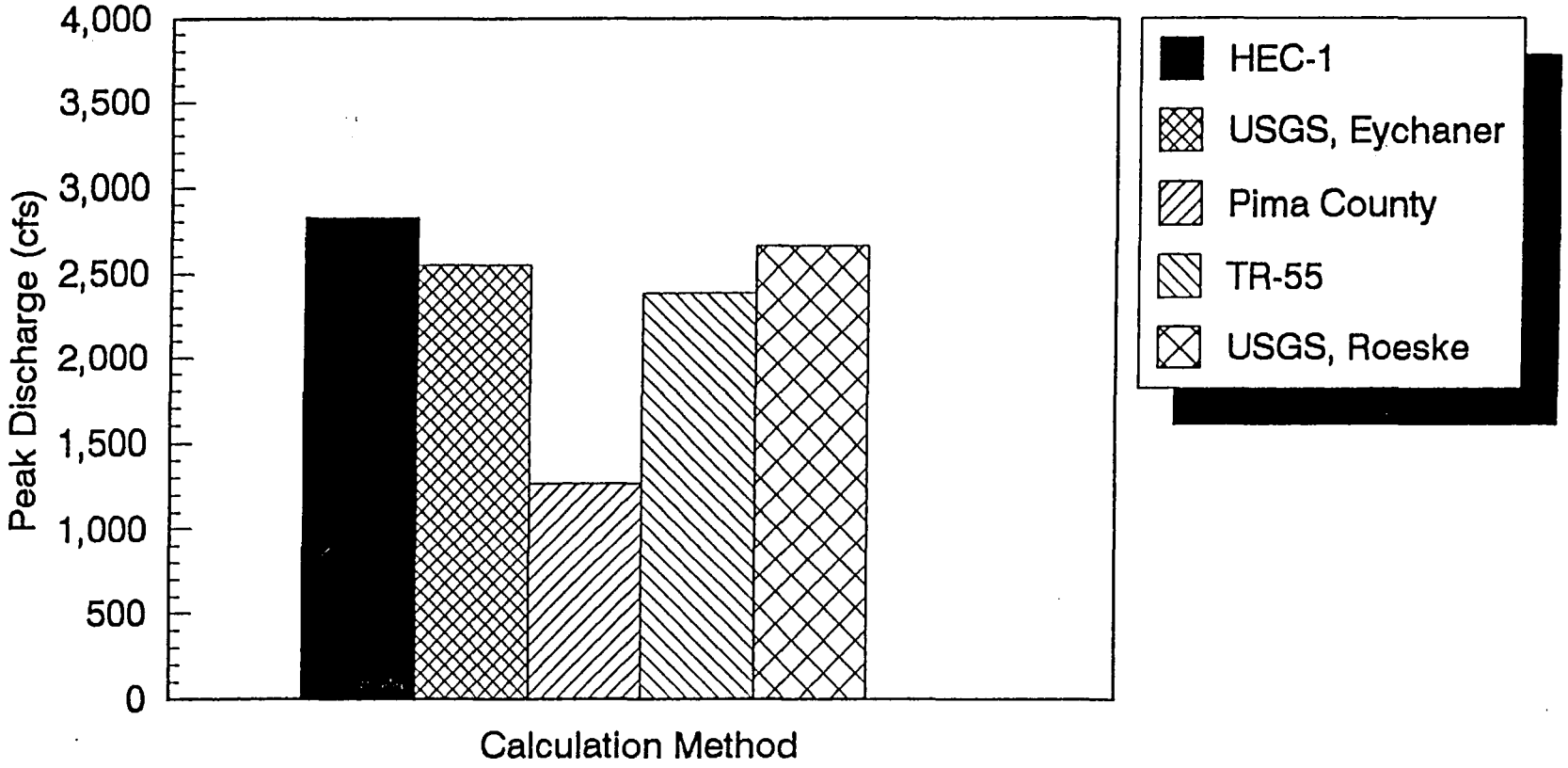


Drainage Area = 1.07 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3400	FIGURE 3.7

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3490

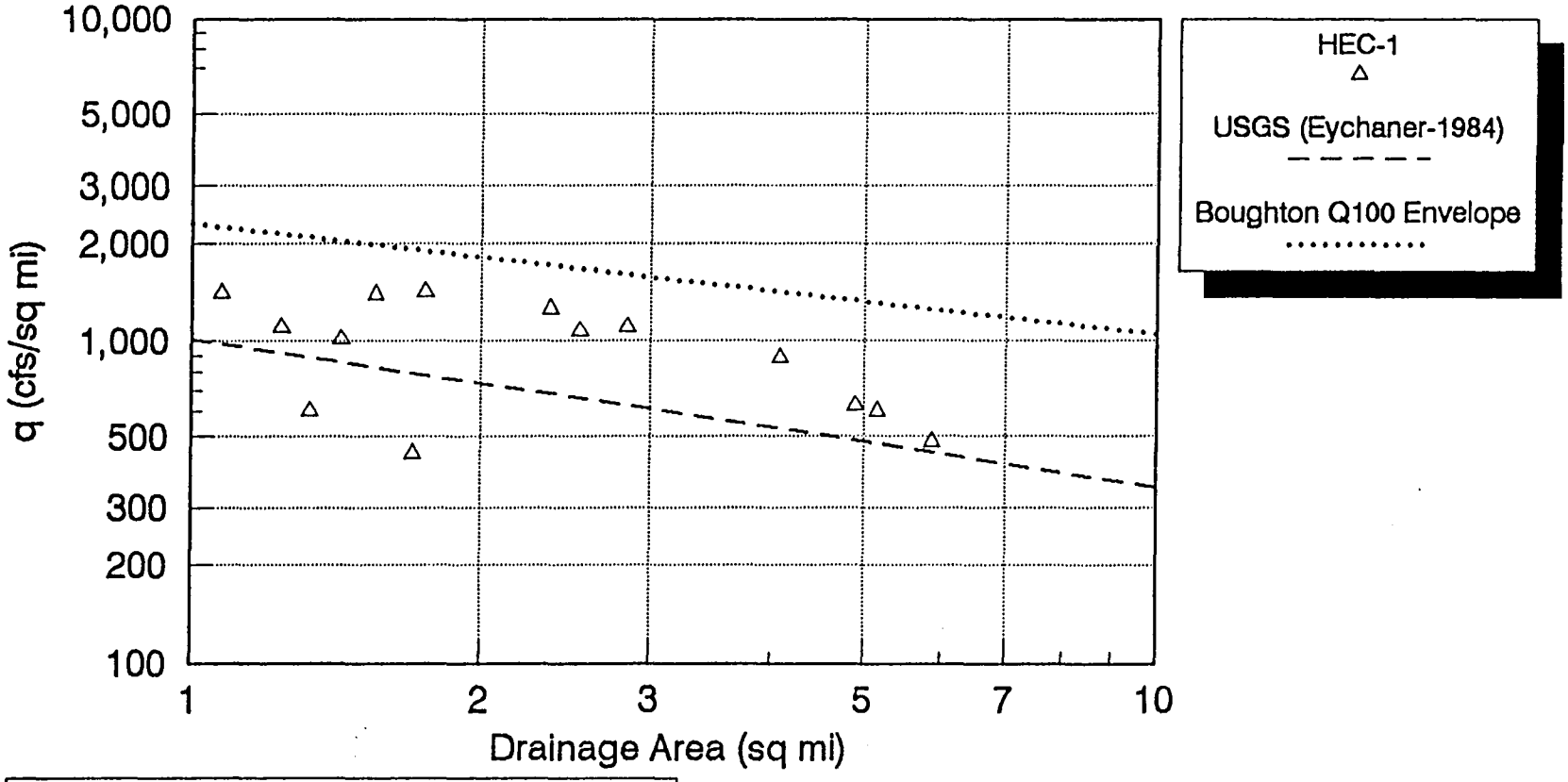


Drainage Area = 5.87 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3490	FIGURE 3.8

Unit Discharge vs Drainage Area

100-Year Event, Non-Sheetflow Areas
City of Phoenix - Areas C & D



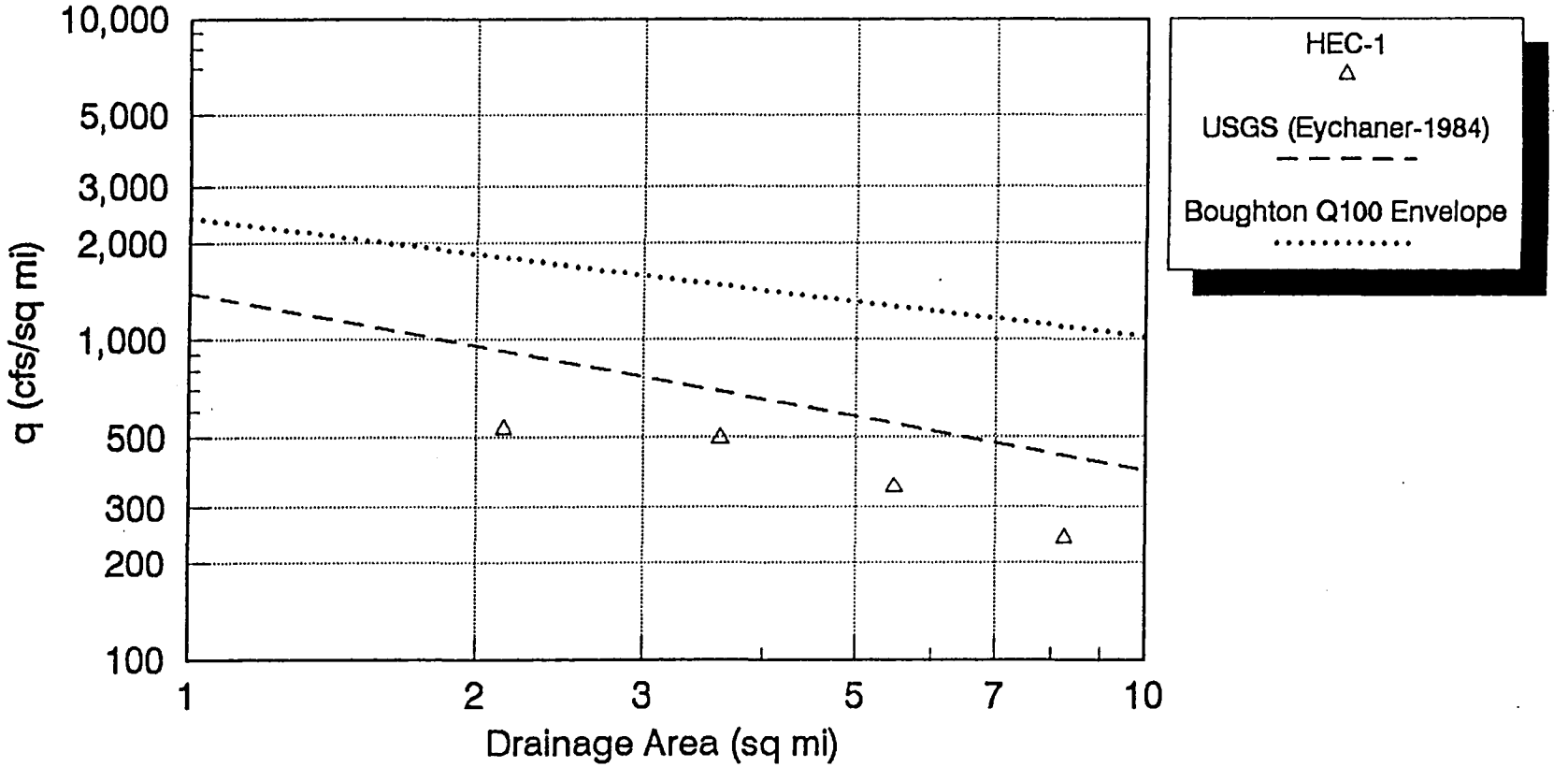
USGS Data from Primary Equations (Table 1)
HEC-1 data based on 24-hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc.	UNIT DISCHARGE VS. DRAINAGE AREA NON-SHEETFLOW AREAS CITY OF PHOENIX - AREAS C & D	FIGURE 3.9
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists		
PROJECT NO. AC02670 / DRAWN BY KMA		
DATE 8/31/90 / CHECKED BY JJH		

Unit Discharge vs Drainage Area

100-Year Event, Sheet-Flow Areas

City of Phoenix - Areas C & D



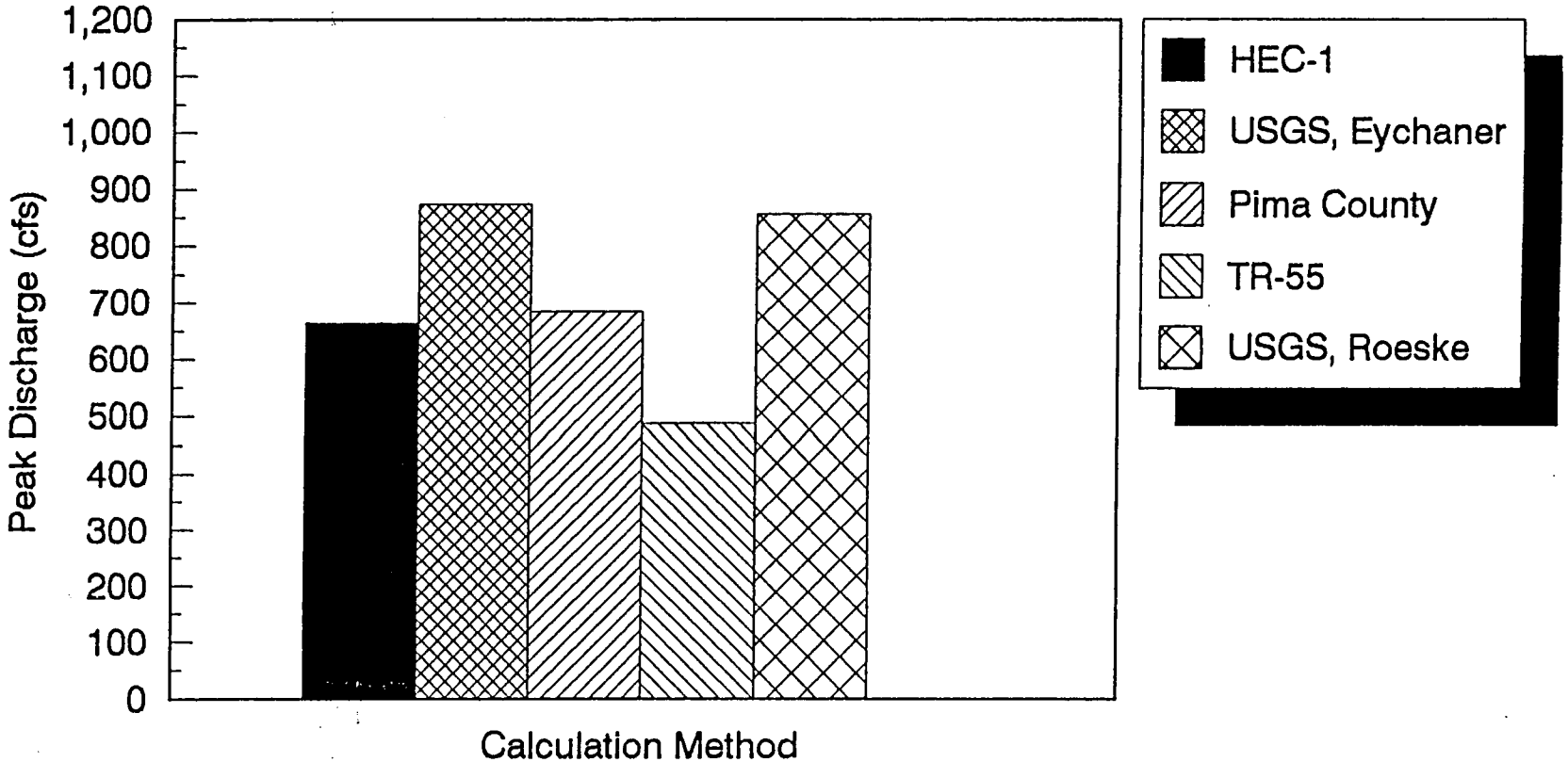
USGS Data from Primary Equations (Table 1)
HEC-1 data based on 24-hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	UNIT DISCHARGE VS. DRAINAGE AREA SHEETFLOW AREAS CITY OF PHOENIX - AREAS C & D	FIGURE 3.10

HEC-1 Peak Discharge Verification

North Scottsdale Area

Concentration Point 2240



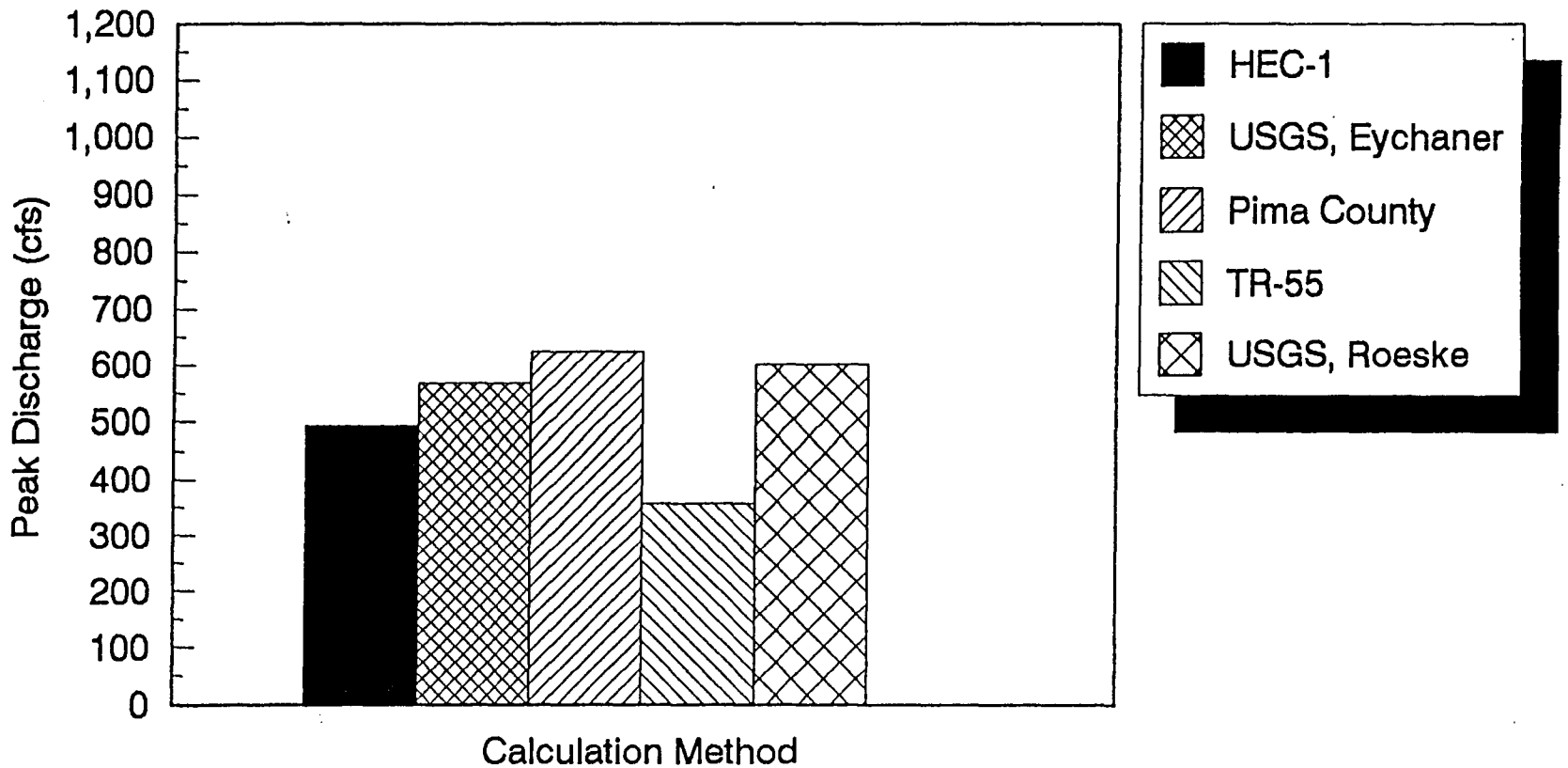
Drainage Area = 0.6033 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 2240	FIGURE 3.11

HEC-1 Peak Discharge Verification

North Scottsdale Area

Concentration Point 2255



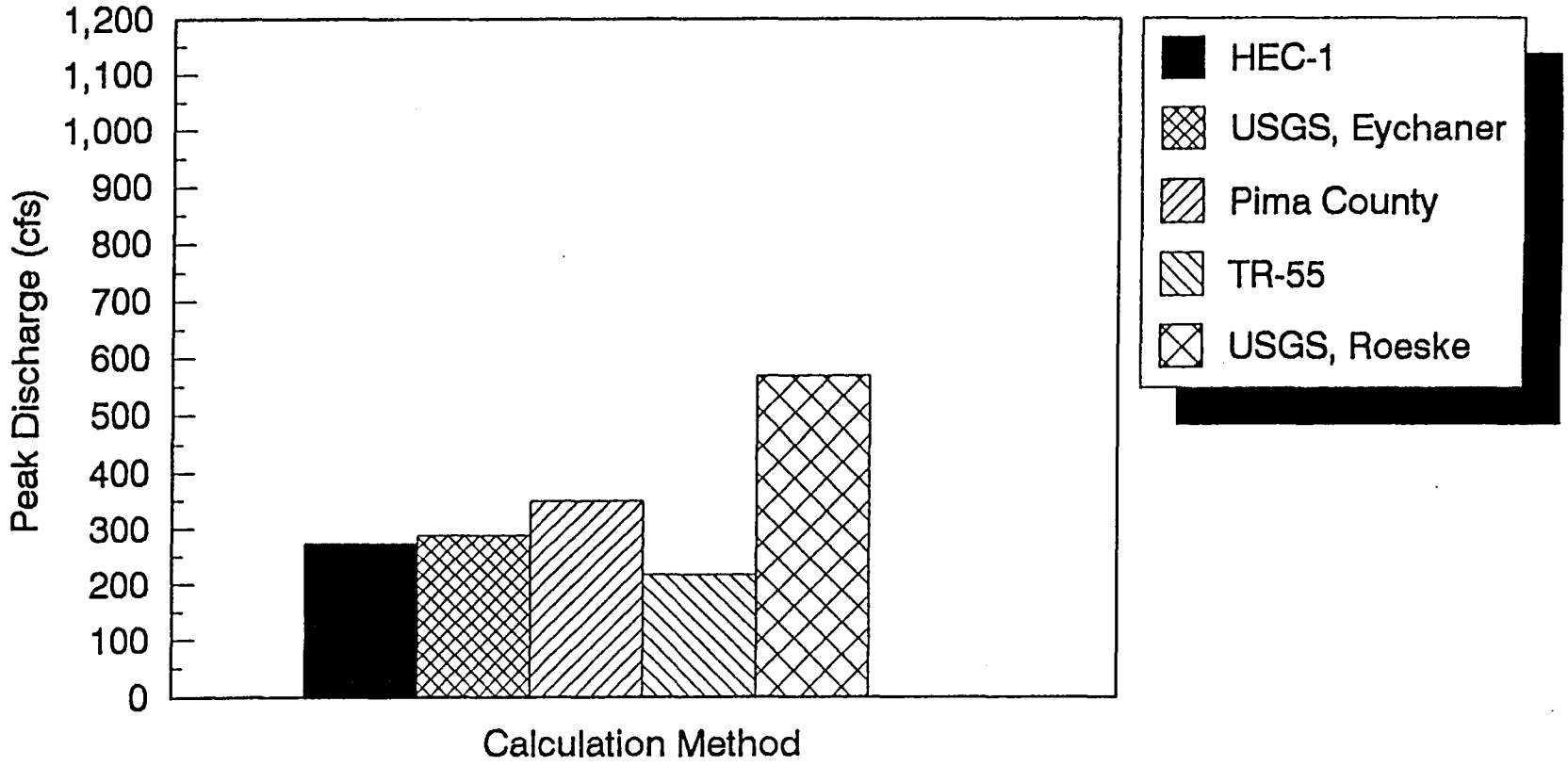
Drainage Area = 0.2955 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 2255	FIGURE 3.12

HEC-1 Peak Discharge Verification

North Scottsdale Area

Concentration Point 2260



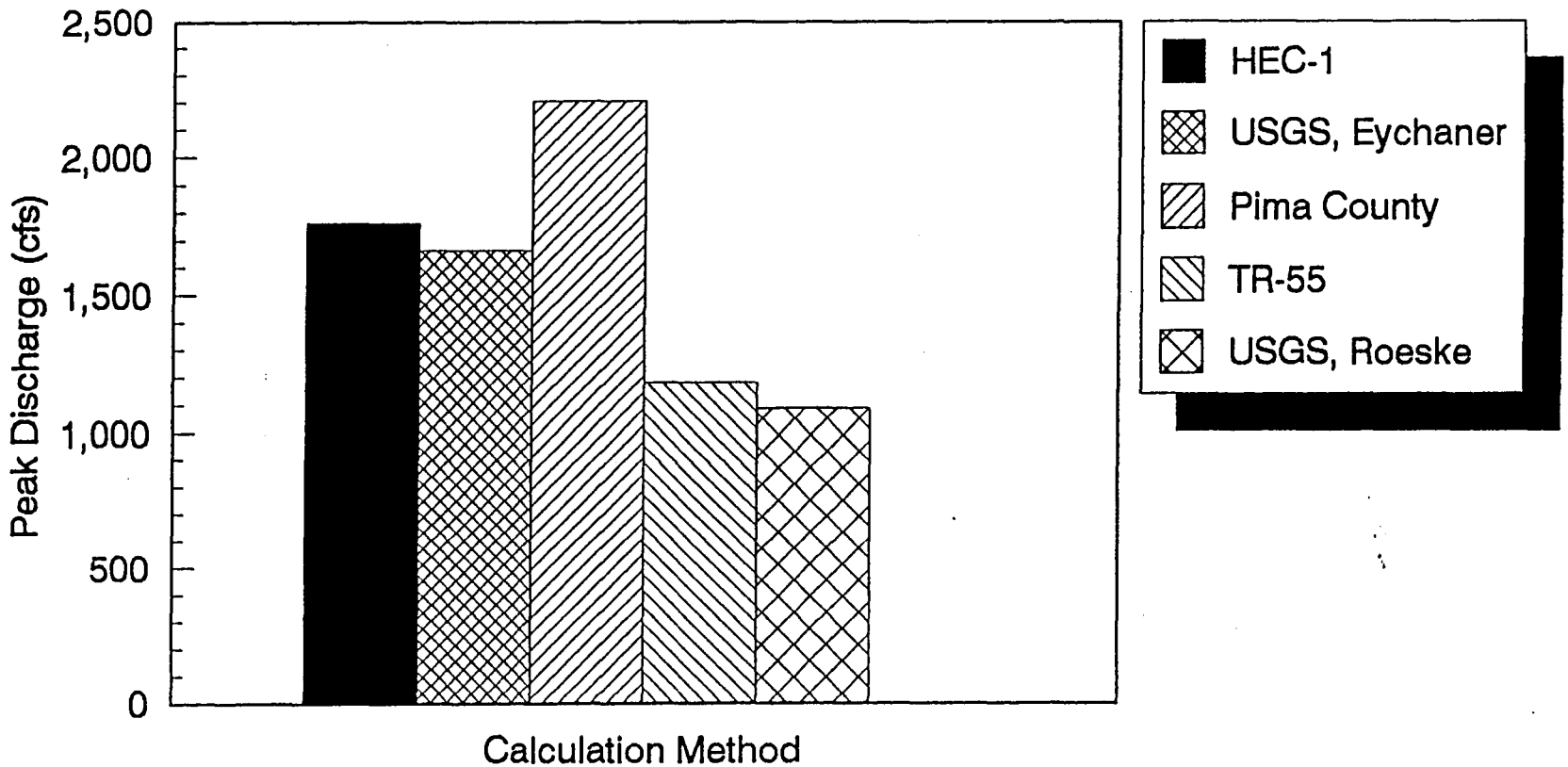
Drainage Area = 0.2647 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 2260	FIGURE 3.13

HEC-1 Peak Discharge Verification

North Scottsdale Area

Concentration Point 2270



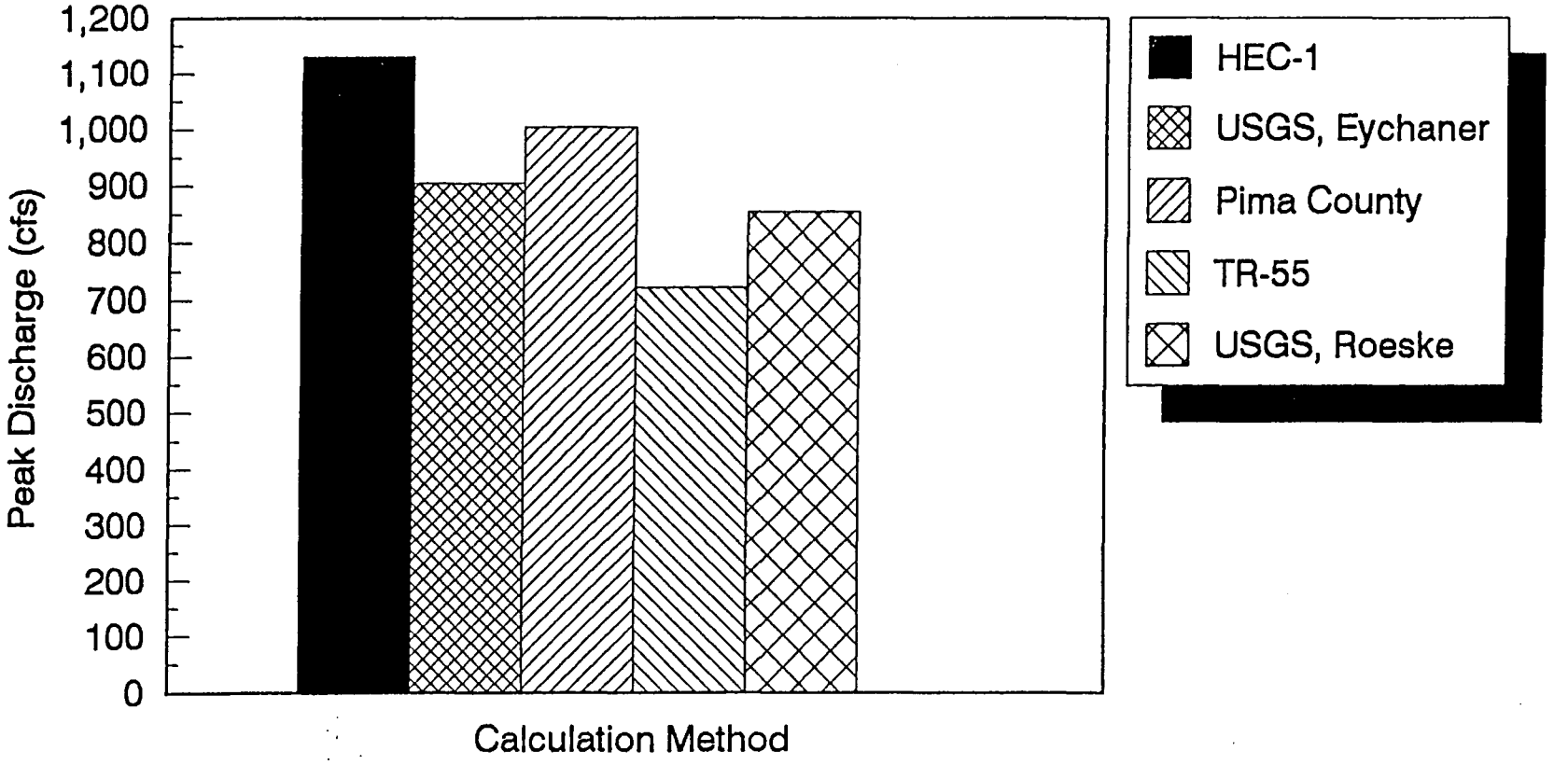
Drainage Area = 0.9714 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 2270	FIGURE 3.14

HEC-1 Peak Discharge Verification

North Scottsdale Area

Concentration Point 2290



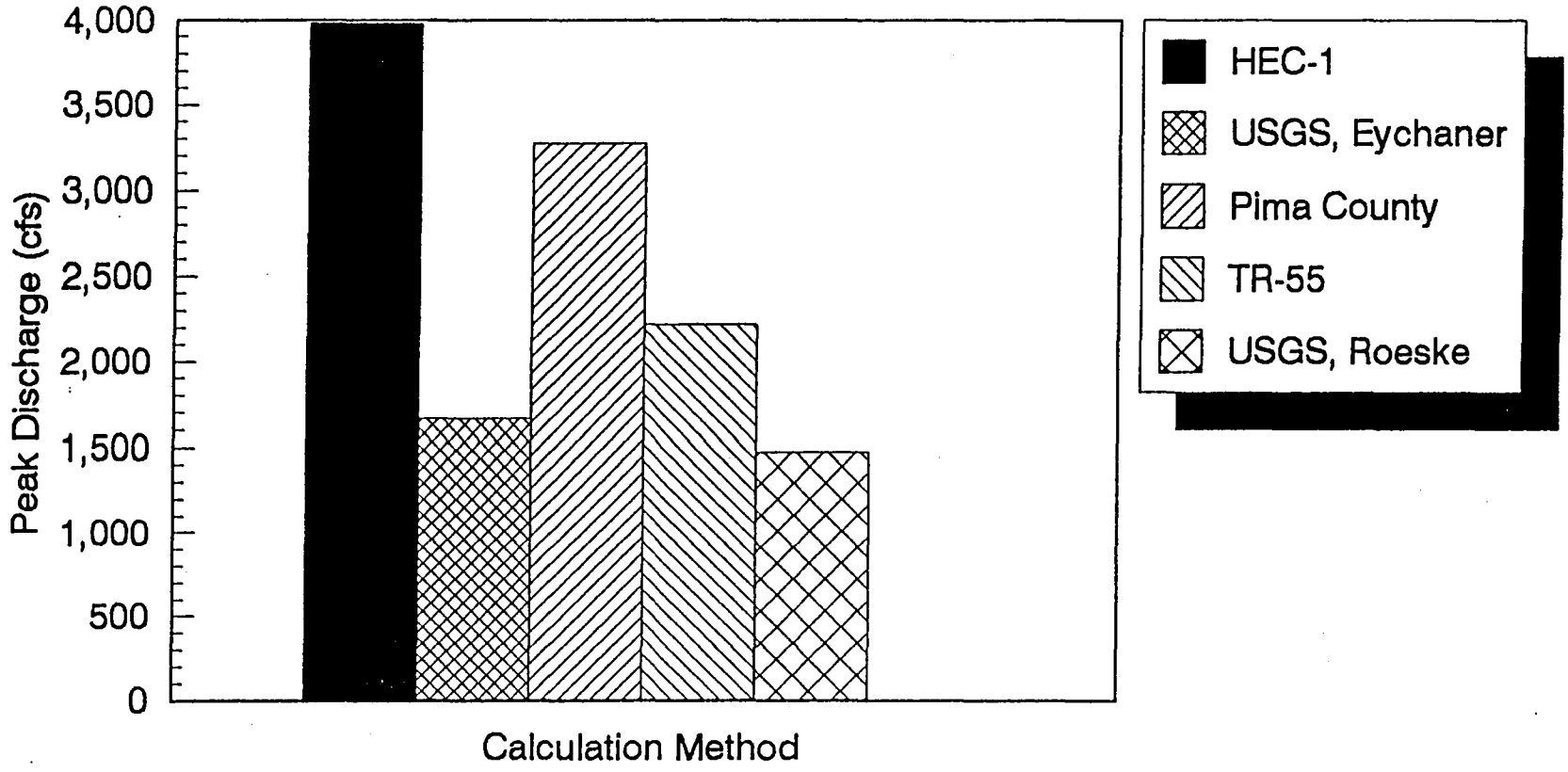
Drainage Area = 0.6005 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc.	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 2290	FIGURE 3.15
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists		
PROJECT NO. AC02670 / DRAWN BY KMA		
DATE 8/31/90 / CHECKED BY JJH		

HEC-1 Peak Discharge Verification

North Scottsdale Area

Concentration Point 2300

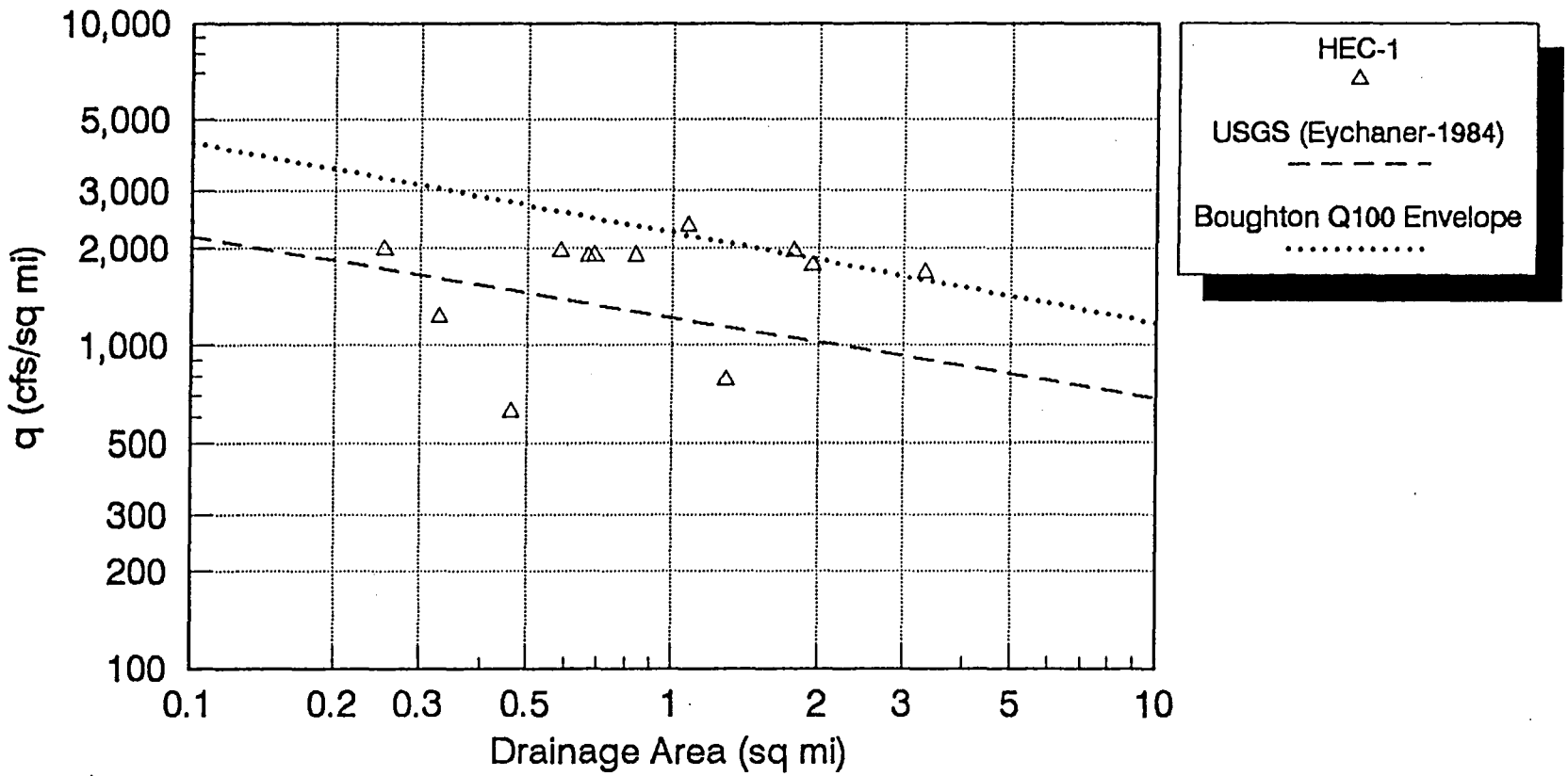


Drainage Area = 1.8007 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 2300	FIGURE 3.16

Unit Discharge vs Drainage Area

100-Year Event, Non-Sheetflow Areas
North Scottsdale Area

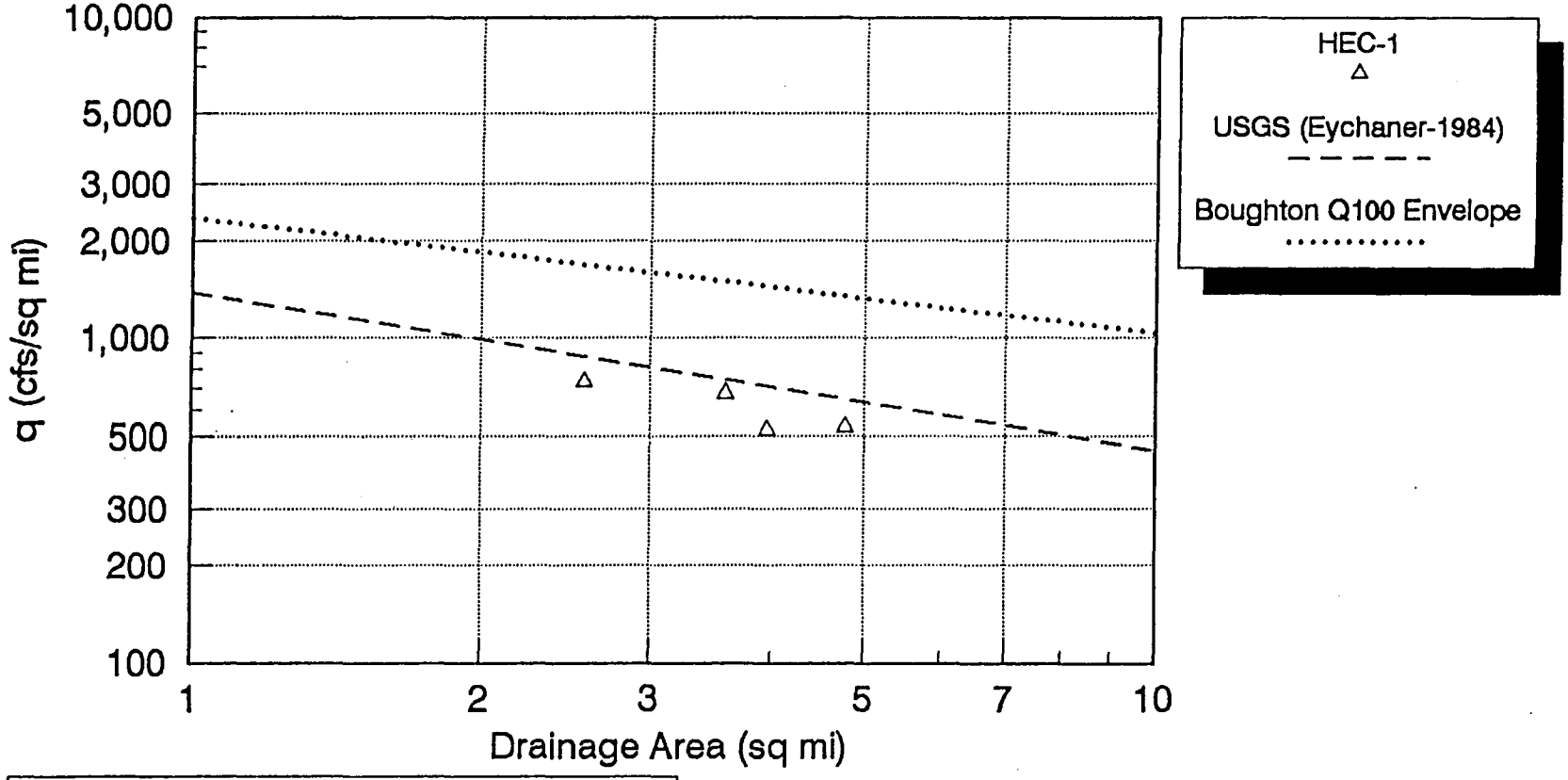


USGS Data from Primary Equations (Table 1)
HEC-1 data based on 24-hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc. Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists PROJECT NO. AC02670 DRAWN BY KMA DATE 8/31/90 CHECKED BY JJH	UNIT DISCHARGE VS. DRAINAGE AREA NON-SHEETFLOW AREAS NORTH SCOTTSDALE	FIGURE 3.17

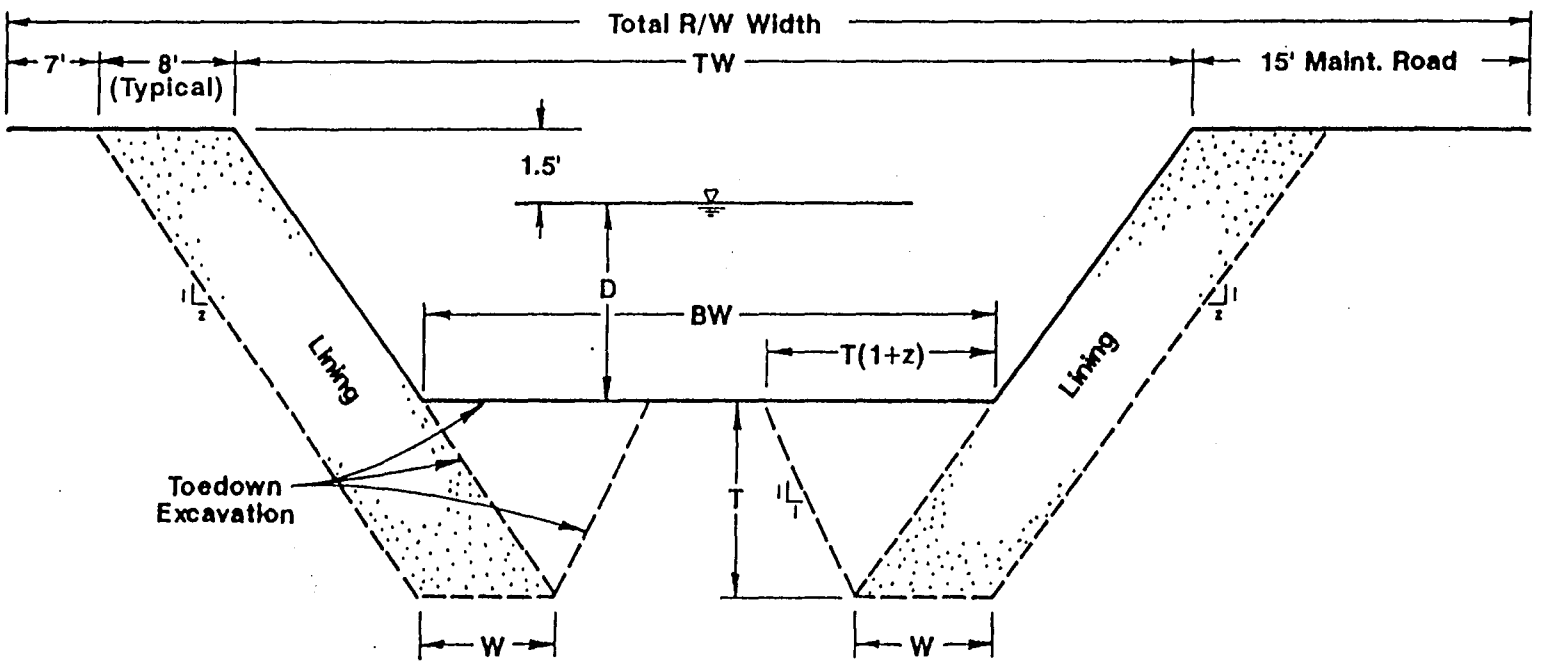
Unit Discharge vs Drainage Area

100-Year Event, Sheet-Flow Areas
North Scottsdale Area



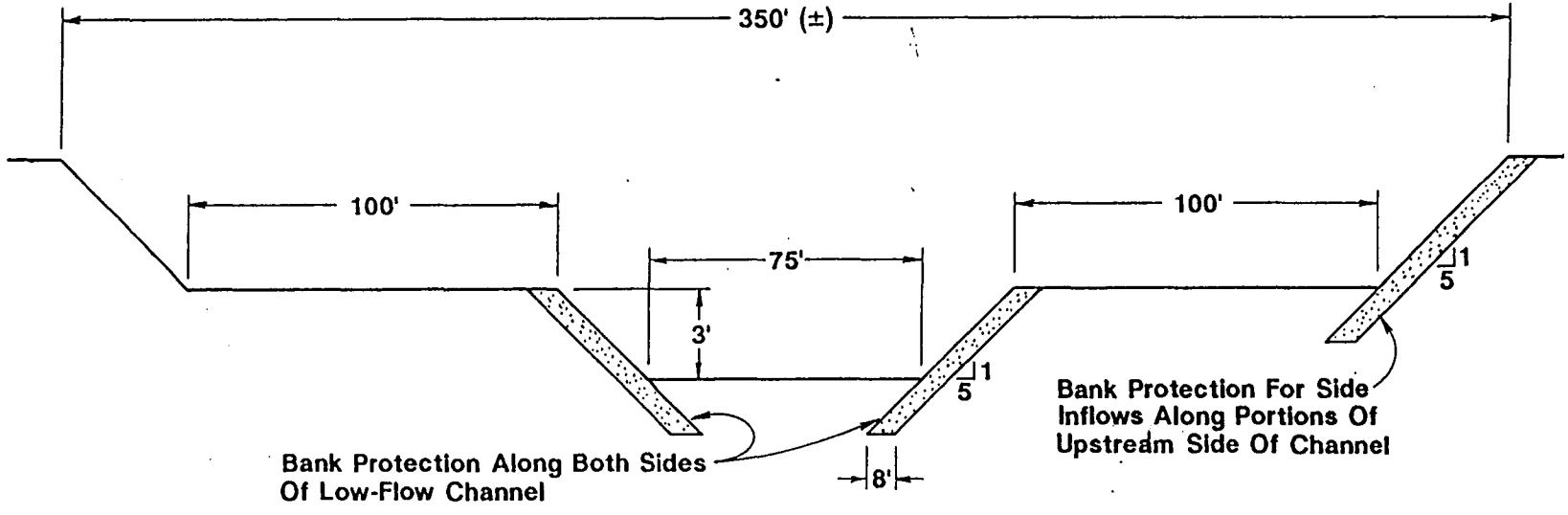
USGS Data from Primary Equations (Table 1)
HEC-1 data based on 24-hour, SCS Type IIA Storm
AMC II, Existing Land-Use Conditions

Water Resources Associates, Inc.	UNIT DISCHARGE VS. DRAINAGE AREA SHEETFLOW AREAS NORTH SCOTTSDALE	FIGURE 3.18
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists		
PROJECT NO. AC02670 / DRAWN BY KMA		
DATE 8/31/90 / CHECKED BY JJH		



NOT TO SCALE

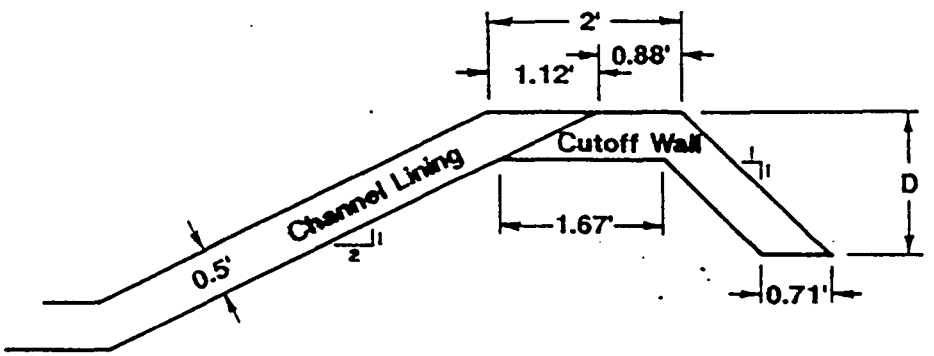
Water Resources Associates, Inc.	GEOMETRY OF 4-POINT CHANNEL CROSS-SECTION	FIGURE 4.1
PROJECT NO AC02670 DRAWN BY KMA		
DATE 8/31/90 CHECKED BY RLW		



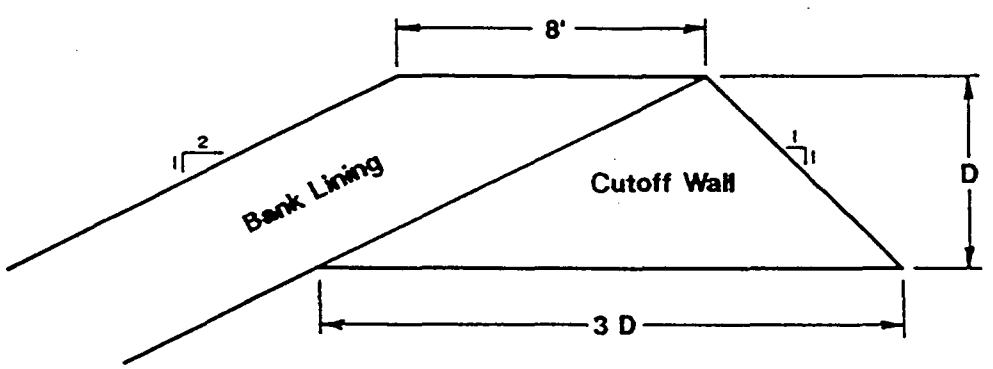
All Side Slopes Are 5:1

NOT TO SCALE

Water Resources Associates, Inc.	GEOMETRY OF 8-POINT CHANNEL CROSS-SECTION	FIGURE 4.2
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists		
PROJECT NO. AC02670 / DRAWN BY KMA		
DATE 8/31/90 / CHECKED BY JJH		

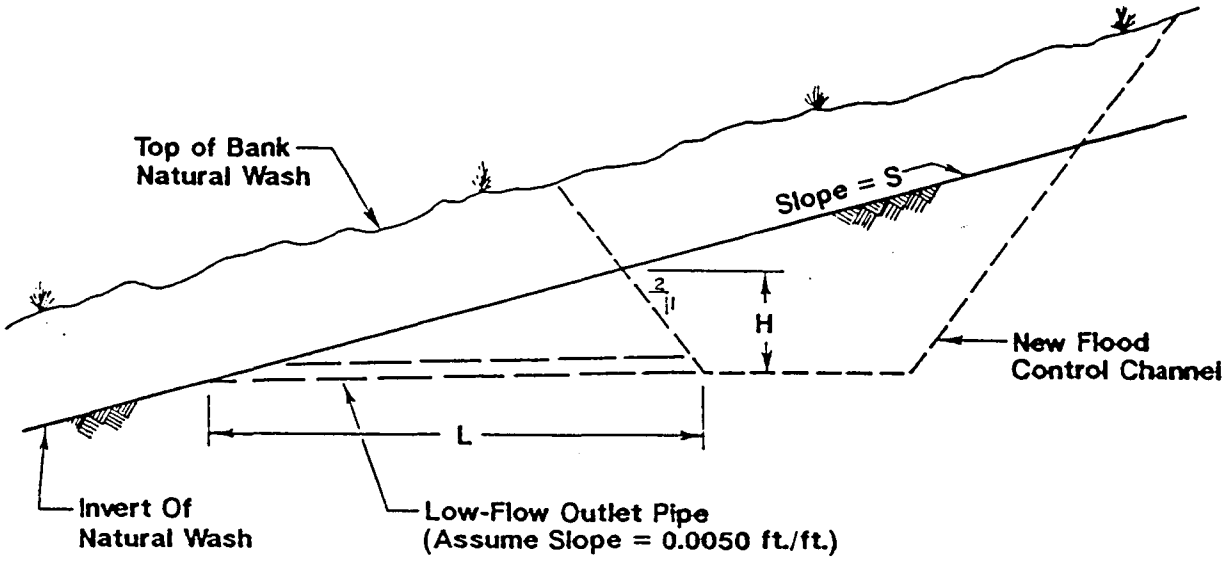


CONCRETE LINING
Not To Scale



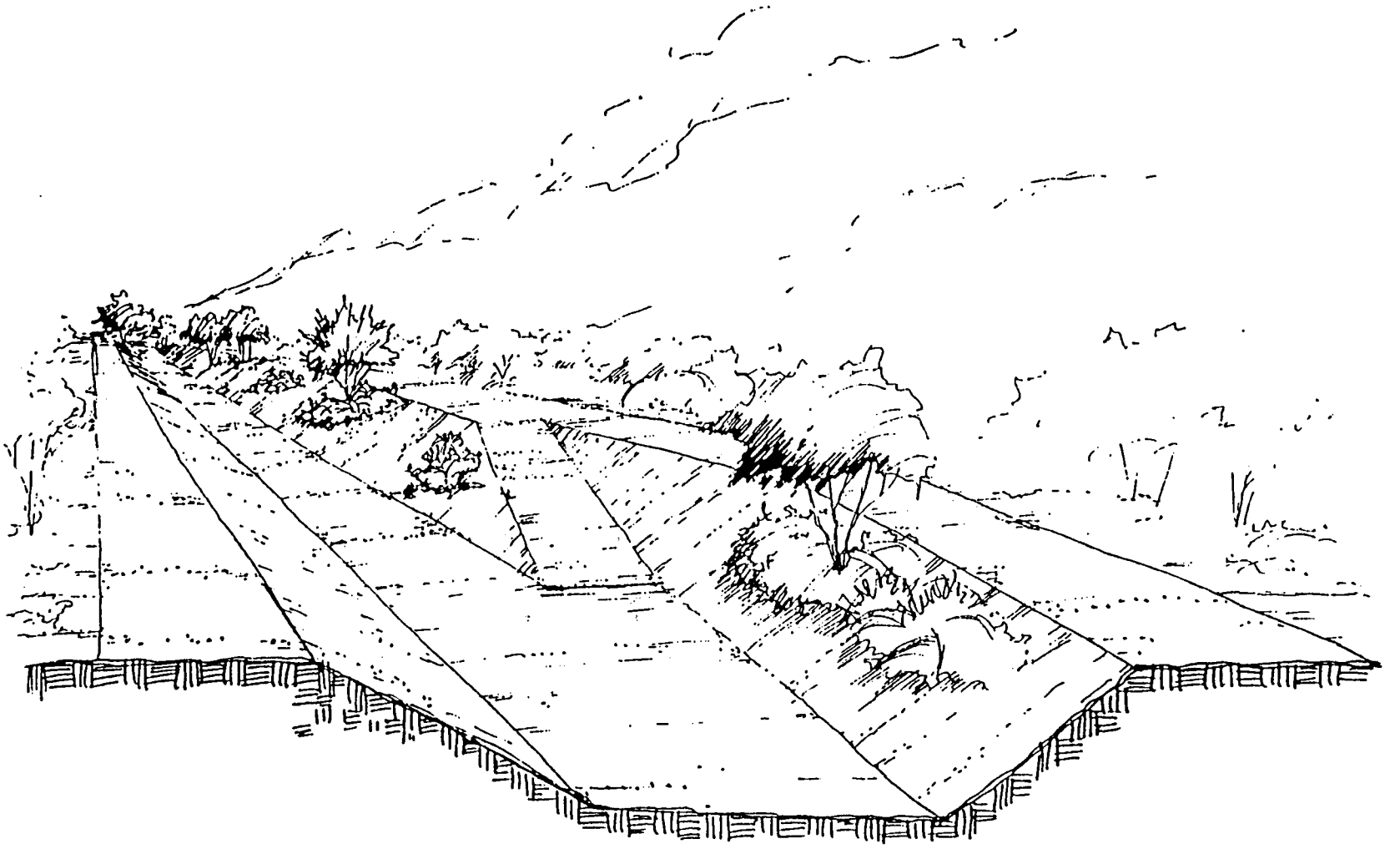
SOIL-CEMENT
Not To Scale

Water Resources Associates, Inc.	CHANNEL LINING CUT-OFF WALL	FIGURE 4.3
PROJECT NO. AC02670 DRAWN BY KMA		
DATE 8/31/90 CHECKED BY BLW		

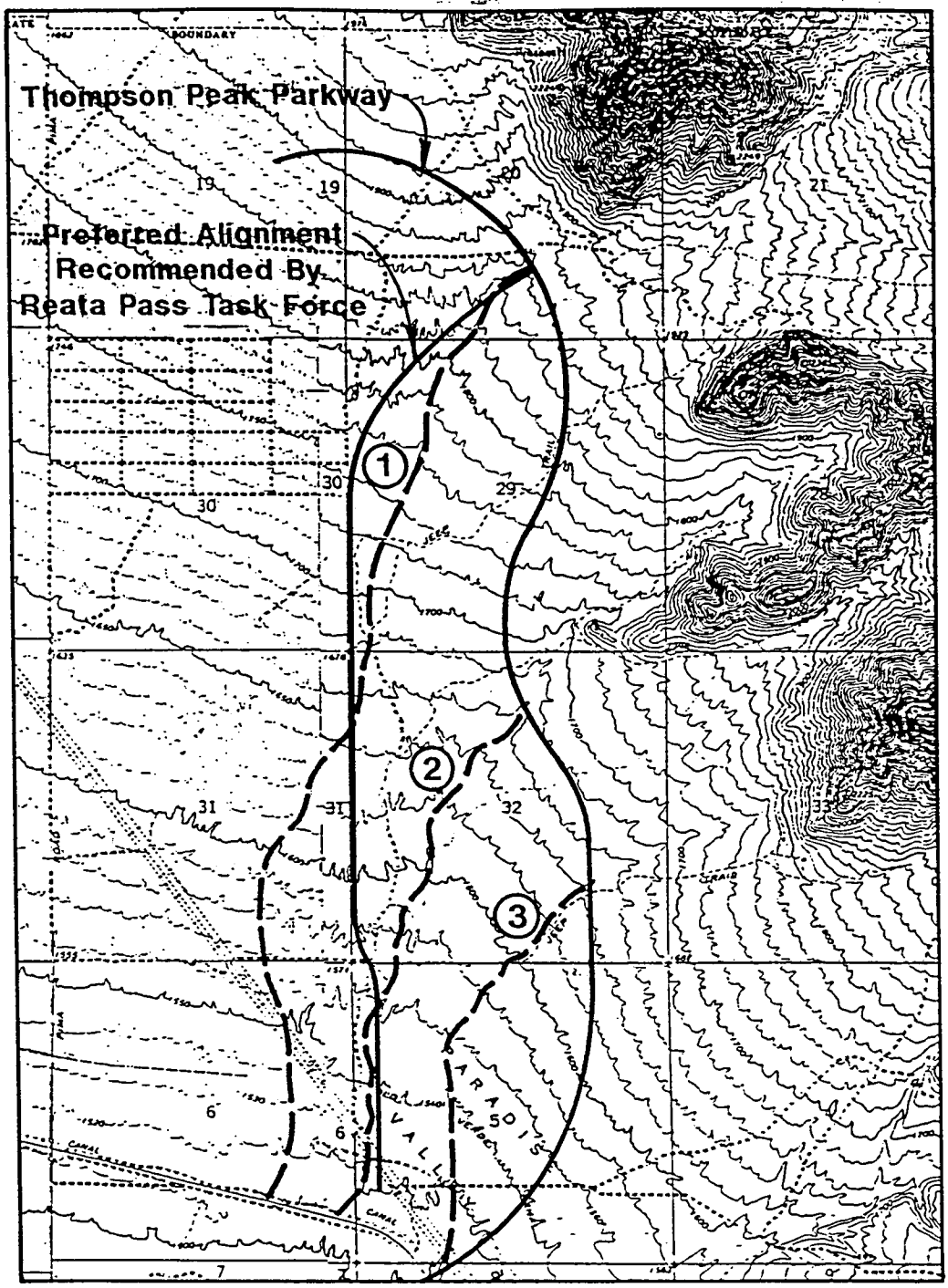


NOT TO SCALE

Water Resources Associates, Inc.	SCHEMATIC OF LOW FLOW CULVERT INSTALLATION	FIGURE 4.4
PROJECT NO AC02670 DRAWN BY KMA		
DATE 8/31/90 CHECKED BY JH		



Water Resources Associates, Inc.		SCHEMATIC OF CHANNEL ACCESS RAMPS	FIGURE 4.5
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists			
PROJECT NO. AC02670	DRAWN BY RM		
DATE 8/31/90	CHECKED BY JJH		

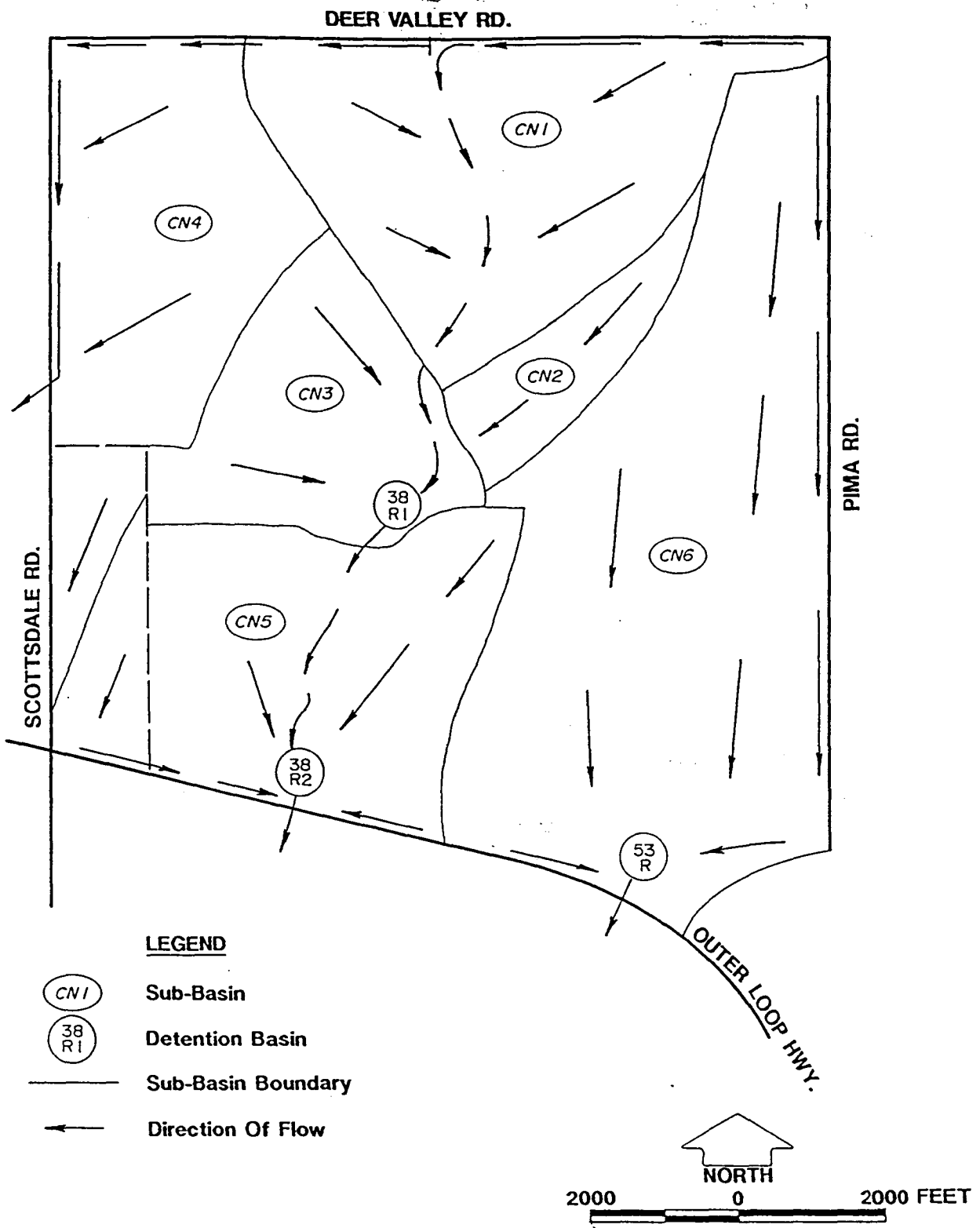


NORTH

3000 0 3000 MILES

BASE MAP : USGS 7.5 Minute Quadrangles - McDowell Peak and Curry's Corner, Az.

Water Resources Associates, Inc.	ALTERNATE CHANNEL ALIGNMENTS FOR REATA PASS	FIGURE 4.6
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists		
PROJECT NO. AC02670 / DRAWN BY KMA		
DATE 8/31/90 / CHECKED BY JJH		



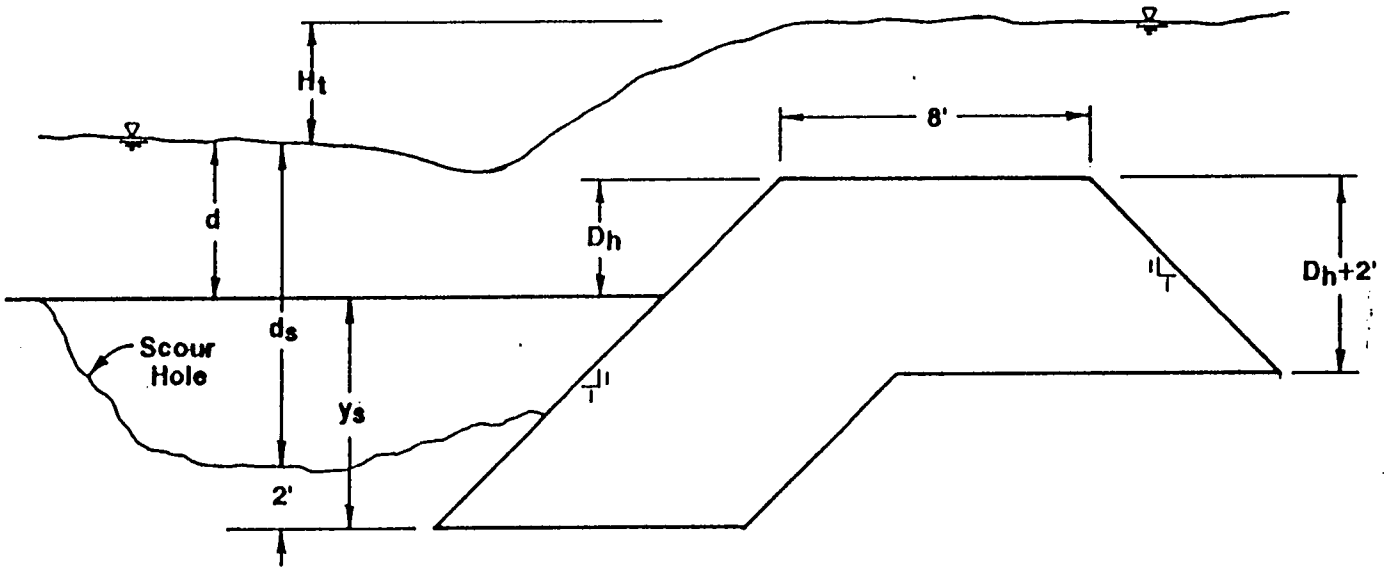
Water Resources Associates, Inc.

Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists

PROJECT NO. AC02670	DRAWN BY KMA
DATE 8/31/90	CHECKED BY JJH

SCHEMATIC OF CORE NORTH DRAINAGE SYSTEM

FIGURE 4.7



NOT TO SCALE

Water Resources Associates, Inc.	TYPICAL SOIL-CEMENT DROP STRUCTURE GEOMETRY	FIGURE 6.1
PROJECT NO AC02670 DRAWN BY KMA		
DATE 8/31/90 CHECKED BY RLW		

Years

Channel Segment	1993	1994	1995	1996	1997	1998
A. Phoenix System						
1. Squaw Peak Parkway Channel						
2. Jomax Road Channel						
3. Pinnacle Peak Road, West						
4. Pinnacle Peak Road, East						
B. Scottsdale Road - Rawhide System						
1. Rawhide Wash						
a. CAP to OLH						
b. OLH to Scottsdale Road						
c. Scottsdale Road to Jomax Road, along Rawhide Wash						
d. Scottsdale Road to 1/2 mile north of Jomax Road						
C. Core North/South System						
*1. Core South Channel System						
*2. Detention Basins 38R-1, 38R-2, 53R						
3. Pima Road Channel						
4. OLH Crossing for Sub 21A/21B						
5. OLH Channel for Sub 2125						
*6. Interior Core North Channels						
7. Scottsdale Road/Deer Valley Road Channel						
D. Reata Pass Channel						
1. Will impact Core North detention basin 53R & Pima Road channel below Deer Valley Road						
Outer Loop Highway (Scottsdale Road to Bell Road)						

* To be built by private interests

Water Resources Associates, Inc.

Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists

PROJECT NO. AC02670 | DRAWN BY KMA

DATE 8/31/90 | CHECKED BY JJH

PROJECT INSTALLATION SCHEDULE

FIGURE 11.1

TABLES

TABLE 2.1

SUMMARY OF HYDROLOGIC SOIL GROUP ASSIGNED TO SOIL COMPLEX/ASSOCIATIONS

MAP SYMBOL	SOIL COMPLEX/ ASSOCIATION	PERCENTAGE	DESCRIPTION	HYDROLOGIC SOIL GROUP
3 & 4	Antho-Carrizo-Maripo Complex	35	Antho	B
		30	Carrizo	A
		20	Maripo	B
		15	Brios	A
			Gilman	B
			Vint	B
			Denure	B
			Momoli	B
			Carrizo	A
Use HSG B for Map Symbol 3 & 4				
6 & 7	Anthony Arizo Complex	40	Anthony	B
		40	Arizo	A
		20	of the group is sandy soils similar to Arizo	A
Use 50% HSG B & 50% HSG A for Map Symbols 6 & 7				
39	Eba-Nickel-Cave Association	30	Eba	C
		25	Nickel	B
		25	Cave	D
		20	Rock Outcrop	D
Use HSG C for Map Symbol 39				

TABLE 2.1 - Continued

SUMMARY OF HYDROLOGIC SOIL GROUP
ASSIGNED TO SOIL COMPLEX/ASSOCIATIONS

MAP SYMBOL	SOIL COMPLEX/ ASSOCIATION	PERCENTAGE	DESCRIPTION	HYDROLOGIC SOIL GROUP
40, 41 42, 43	Eba-Pinaleno Complex	45	Eba	C
		35	Pinaleno	B
		20	Arizo	A
			Anthony	B
			Continental	C
			Ohacco	C
			Greyeagle	D
			Nickel	B
			Vado	B
			Tres Hermanos	B
Use 50% HSG B & 50% HSG C for Map Symbols 40, 41, 42, 43				
91 & 92	Momoli-Carrizo Complex	45	Momoli	B
		35	Carrizo	A
		20	Mohall	B
			Tremant	B
			Gunsight	B
			Chuckawalla	B
			Denure	B
			Gilman	B
			Maripo	B
			Carrizo	A
Use HSG B for Map Symbols 91 & 92				

TABLE 2.1 - Continued

SUMMARY OF HYDROLOGIC SOIL GROUP
ASSIGNED TO SOIL COMPLEX/ASSOCIATIONS

MAP SYMBOL	SOIL COMPLEX/ ASSOCIATION	PERCENTAGE	DESCRIPTION	HYDROLOGIC SOIL GROUP
93 & 94	Nickel-Cave Complex	50	Nickel	B
		35	Cave	D
		15	Arizo	A
			Anthony	B
			Pinaleno	B
			Greyeagle	D
Use HSG C for Map Symbols 93 & 94				

TABLE 3.1

SUMMARY OF SCS CURVE NUMBERS AS A FUNCTION OF STORM DURATION AND HYDROLOGIC SOIL GROUP

HYDROLOGIC SOIL GROUP	CURVE NUMBER BY STORM DURATION[1]					
	1-HR	2-HR	3-HR	6-HR	12-HR	24-HR
A	74	71	69	66	63	60
B	83	81	80	78	76	74
C	89	87	86	85	83	82
D	92	91	90	88	87	86

NOTES:

1. Adjustments made in accordance with Runoff Curve Numbers for Semiarid Range and Forest Conditions, Woodward, 1973. Values are rounded to the nearest integer.

   All curve numbers are based on "desert brush" with 15% cover density. Antecedent Moisture Condition II is assumed.

Table 3.2
Percent Of Impervious Cover For Residential Land Use
Adjusted For Natural Area Open Space (NAOS) Requirements
Upper Indian Bend Wash Regional Drainage Plan

Listed Residential Density (DU/Ac.)	Average Density For This Range (DU/Ac.)	Percent Impervious Cover With No Adjustment For NAOS	Effective Land-Use Density, With Adjustment For NAOS Requirements, DU/Ac.					
			0-2% Slope & Under 2000' Elevation NAOS=15%	Over 2% Slope & Under 2000' Elevation NAOS=20%	Over 5% Slope Or Over 2000' Elevation NAOS=25%	Over 10% Slope Or Over 2500' Elevation NAOS=30%	Over 15% Slope Or Over 3500' Elevation NAOS=35%	Over 25% Slope Or Over 4000' Elevation NAOS=80%
1/5 - 1/3	0.267	6.9	0.314	0.334	0.356	0.381	0.411	1.335
1/3 - 1/2	0.417	10.2	0.491	0.521	0.556	0.596	0.642	2.085
1/2 - 1	0.750	16.0	0.882	0.938	1.000	1.071	1.154	3.750
1 - 2	1.500	24.1	1.765	1.875	2.000	2.143	2.308	7.500
2 - 4	3.000	34.9	3.529	3.750	4.000	4.286	4.615	15.000
4 - 8	6.000	54.0	7.059	7.500	8.000	8.571	9.231	30.000
8 - 12	10.000	74.0	11.765	12.500	13.333	14.286	15.385	50.000
12 - 22	17.000	94.0	20.000	21.250	22.667	24.286	26.154	85.000

Listed Residential Density (DU/Ac.)	Average Density For This Range (DU/Ac.)	Percent Impervious Cover With No Adjustment For NAOS	Average Percent Of Impervious Cover, With Adjustment For NAOS Requirements					
			0-2% Slope & Under 2000' Elevation NAOS=15%	Over 2% Slope & Under 2000' Elevation NAOS=20%	Over 5% Slope Or Over 2000' Elevation NAOS=25%	Over 10% Slope Or Over 2500' Elevation NAOS=30%	Over 15% Slope Or Over 3500' Elevation NAOS=35%	Over 25% Slope Or Over 4000' Elevation NAOS=80%
1/5 - 1/3	0.267	6.9	6.9	6.9	6.9	6.9	6.9	5.3
1/3 - 1/2	0.417	10.2	10.2	10.1	9.9	9.8	9.6	6.5
1/2 - 1	0.750	16.0	15.5	15.4	15.3	14.7	14.2	8.8
1 - 2	1.500	24.1	22.4	21.9	21.3	20.7	20.0	13.2
2 - 4	3.000	34.9	32.8	32.1	31.5	30.7	29.9	18.8
4 - 8	6.000	54.0	51.0	50.0	49.0	47.7	46.4	20.8
8 - 12	10.000	74.0	69.1	66.9	64.5	62.1	59.4	20.8
12 - 22	17.000	94.0	83.6	79.7	75.3	70.3	65.4	20.8

Note: Percents of impervious cover are taken from Figure 3.1, General Drainage Plan For North Scottsdale, Arizona, 1989.
NAOS requirements are taken from a rough draft of ESL Section 7.850, Development Standards, 4/13/90, City of Scottsdale.

File: NAOS.WK1

TABLE 3.3

SUMMARY OF OVERLAND FLOW ROUGHNESS VALUES

TOPOGRAPHIC AREA	OVERLAND FLOW ROUGHNESS VALUE
1. Alluvial Fan Terrace	0.10
2. Rolling Fan Terrace	0.15
3. Northern portions of watershed with well-defined channel geometry and coarse-grained surface conditions	0.20
4. Transition areas from steep mountain slopes to pediments and fan terraces	0.25
5. Steep mountain slopes with rugged rock outcrops	0.30

TABLE 3.4

SUMMARY OF RAINFALL VALUES
UPPER INDIAN BEND WASH REGIONAL DRAINAGE AND FLOOD CONTROL PLAN
100 - YEAR, 24 - HOUR STORM

SUB-BASIN SERIES	RAINFALL DEPTH (inches) For Specified Areal Reduction			
	O Square Miles	10 Square Miles	25 Square Miles	50 Square Miles
1-53	4.39	4.32	4.25	4.18
1000 Series	4.78	4.71	4.63	4.55
2000 Series	4.29	4.23	4.15	4.08
3000 Series	4.28	4.22	4.14	4.07

NOTE: Areal reduction factors taken from Figure 14, NOAA Atlas 2, Volume VIII, Arizona

10 square miles = 0.985
25 square miles = 0.968
50 square miles = 0.952

TABLE 3.5

SUMMARY OF INDEPENDENT PEAK DISCHARGE CALCULATION
FOR VERIFICATION OF HEC-1 MODEL RESULTS
NORTH PHOENIX AREA

CONCENTRATION POINT	Q_{100} (cfs)				
	HEC-1[1]	USGS EYCHANER[2]	PIMA COUNTY PEAK[1]	TR-55 TYPE II[1]	USGS ROESKE[3]
3020 D.A. = 0.31 mi.^{2}	433	538	432	295	613/1054
3135 D.A. = 0.04 mi.^{2}	66	120	79	39	221/324
3150 D.A. = 1.02 mi.^{2}	675	1064	604	552	1111/2629
3160 D.A. = 0.40 mi.^{2}	532	639	550	378	696/1201
3400 D.A. = 1.07 mi.^{2}	655	1053	537	534	1138/2423
3490 D.A. = 5.87 mi.^{2}	2824	2543	1264	2382	2660/5094

NOTES:

1) Based on 10 Square Mile Area Reduction for Rainfall and Existing Land Use

2) Uses Primary Estimating Equations in Table 1 (Pg. 6 of 1984 USGS Reference)

3) Region 2 vs. Region 3
Mean Annual Precipitation Approximately 12.5 Inches
Average Basin Elevation from Eychaner Calculations

TABLE 3.6

SUMMARY OF INDEPENDENT PEAK DISCHARGE CALCULATIONS
FOR VERIFICATION OF HEC-1 MODEL RESULTS,
NORTH SCOTTSDALE AREA

CONCENTRATION POINT	Q_{100} (cfs)				
	HEC-1[1]	USGS EYCHANER[2]	PIMA COUNTY PEAK[1]	TR-55 TYPE II[1]	USGS ROESKE[3]
2240 D.A. = .6033 mi.^{2}	664	872	683	485	855/2154
2255 D.A. = .2955 mi.^{2}	491	566	623	356	599/1200
2260 D.A. = .2647 mi.^{2}	273	288	349	216	567/1171
2270 D.A. = .9714 mi.^{2}	1763	1662	2203	1179	1084/2379
2290 D.A. = .6005 mi.^{2}	1130	903	1002	721	853/2114
2300 D.A. = 1.8007 mi.^{2}	3975	1663	3273	2216	1475/2000

NOTES:

1) Based on 10 Square Mile Areal Reduction for Rainfall and Existing Land Use

2) Uses Primary Estimating Equations in Table I (pg. 6 of 1984 USGS reference).

3) Region 2 vs. Region 3,
Mean Annual Precipitation Approximately 12.5 Inches
Average Basin Elevation from Eychaner Calculations

Table 4.1
Summary Of Low-Flow Culvert Costs
Upper Indian Bend Wash Regional Drainage Plan
100-Year, 24-Hour Event

Horizontal Component of Channel Side-Slope: 2

Channel System	Headwall Cost (2)	Unit Cost, Installed Pipe (36" RCP, $/L.F.)	Differential Channel Depth (ft)	Slope of Intercepted Wash (ft/ft)	Culvert Slope (ft/ft)	Culvert Length (ft)	Culvert Cost, With Headwalls (each)	No. of Culverts	Total Culvert Cost, With Headwalls
Pima Road	$3,900	$78.57	4.0	0.0125	0.0050	547	$46,852	114	$5,341,144
Rawhide Wash	$3,900	$78.57	4.0	0.0125	0.0050	547	$46,852	0	$0
Scottsdale Road	$3,900	$78.57	4.0	0.0125	0.0050	547	$46,852	26	$1,218,156
Deer Valley, Hayden to Pima	$3,900	$78.57	4.0	0.0125	0.0050	547	$46,852	0	$0
Deer Valley, Hayden to Scottsdale to Beardsley	$3,900	$78.57	4.0	0.0125	0.0050	547	$46,852	0	$0
Jomax Road, to Pinnacle Peak Rd	$3,900	$78.57	4.0	0.0125	0.0050	547	$46,852	79	$3,701,319
Pinnacle Peak Road, East of Squaw Peak Parkway	$3,900	$78.57	4.0	0.0125	0.0050	547	$46,852	29	$1,358,712
Pinnacle Peak Road, West of Squaw Peak Parkway	$3,900	$78.57	4.0	0.0125	0.0050	547	$46,852	30	$1,405,564
Reata Pass, East of Thompson Peak Parkway	$3,900	$78.57	4.0	0.0125	0.0050	547	$46,852	31	$1,452,416
TOTALS:								309	$14,477,310

File: LFCULV.WK1

Note: Differential channel depth is based on a 2 foot deep natural wash being intercepted by a 6 foot deep manmade channel

Table 4.2
Summary Of Bridge Costs, No Low-Flow Culverts
Upper Indian Bend Wash Regional Drainage Plan
100-Year, 24-Hour Event

Horizontal Component of Channel Side-Slope: 2
Unit Cost of Bridge Deck Area ($/sq ft): $45.00
Channel n=.045, FB=2.0 ft

Channel System	Bridge Location	Number of Lanes	Channel Bottomwidth (ft)	Channel Depth With Freeboard (ft)	Channel Topwidth (ft)	Bridge Width (ft)	Bridge Cost	Bridge Cost Per System
Pima Road (Reata Pass to CAP Dike 4)	Jomax Road	4	11.2	6.0	35.2	62.0	$142,848	$1,471,483
	Happy Valley Road	4	66.0	6.0	90.0	62.0	$295,740	
	Pinnacle Peak Road	4	85.7	6.0	109.7	62.0	$350,703	
	Pima/Deer Valley Rd Intxn	6	86.5	6.0	156.3	88.0	$682,192	
Rawhide Wash	Happy Valley Road	4	218.0	6.0	242.0	62.0	$719,820	$6,095,178
	Pinnacle Peak Road	4	184.6	6.0	208.6	62.0	$626,634	
	Scottsdale Road	6	290.6	6.0	314.6	88.0	$1,309,176	
	Deer Valley Road	6	296.5	6.0	320.5	88.0	$1,332,540	
	Beardsley Road	4	310.2	6.0	334.2	62.0	$977,058	
	Union Hills Drive	4	365.0	6.0	389.0	62.0	$1,129,950	
Scottsdale Road	Jomax Road	4	15.9	6.0	39.9	62.0	$155,961	$619,938
	Happy Valley Road	4	42.8	6.0	66.8	62.0	$231,012	
	Pinnacle Peak Road	4	43.5	6.0	67.5	62.0	$232,965	
Deer Valley, Hayden to Pima	(none)		0.0	6.0	24.0	0.0	$0	$0
Deer Valley, Hayden to Scottsdale to Beardsley	Scottsdale Road	6	46.6	6.0	70.6	88.0	$342,936	$342,936
Jomax Road to Squaw Peak Parkway to Pinnacle Peak Rd	64th Street	4	28.6	6.0	52.6	62.0	$191,394	$1,731,960
	56th Street	4	73.8	6.0	97.8	62.0	$317,502	
	Tatum Blvd	6	114.7	6.0	138.7	88.0	$612,612	
	Happy Valley Road	4	178.8	6.0	202.8	62.0	$610,452	
Pinnacle Peak Road, East of Squaw Peak Parkway	Tatum Blvd	6	42.1	6.0	66.1	88.0	$325,116	$3,432,105
	40th Street	4	109.3	6.0	133.3	62.0	$416,547	
	Pinnacle Peak Road	4	219.3	6.0	243.3	62.0	$723,447	
	Deer Valley Road	6	245.0	6.0	269.0	88.0	$1,128,600	
	Beardsley Road	4	260.5	6.0	284.5	62.0	$838,395	
Pinnacle Peak Road, West of Squaw Peak Parkway	32nd Street	4	49.1	6.0	73.1	62.0	$248,589	$1,728,666
	Pinnacle Peak Road	4	251.9	6.0	275.9	62.0	$814,401	
	Deer Valley Road	6	128.1	6.0	152.1	88.0	$665,676	
Reata Pass, East of Thompson Peak Parkway	Pinnacle Peak Road	4		8 Point XSEC	358.0	62.0	$1,043,460	$5,720,220
	Deer Valley Road	6		8 Point XSEC	359.0	88.0	$1,485,000	
	Beardsley Road	4		8 Point XSEC	366.0	62.0	$1,065,780	
	Union Hills Drive	4		8 Point XSEC	364.0	62.0	$1,060,200	
	Bell Road	4		8 Point XSEC	366.0	62.0	$1,065,780	
File: BRDGCST3.WK1						TOTAL:	$21,142,486	$21,142,486

TABLE 5.1

SUMMARY OF CHANNEL/CULVERT INTERSECTIONS WITH THE OUTER LOOP HIGHWAY UPPER INDIAN BEND WASH REGIONAL DRAINAGE AND FLOOD CONTROL PLAN

CHANNEL ALIGNMENT	APPROXIMATE OUTER LOOP HIGHWAY STATION	Q_{100}[2] (cfs)	n	SLOPE (ft/ft)	CHANNEL BW (ft/ft)	CHANNEL[6] TW (ft)	CULVERT SIZE
Squaw Peak Parkway	1605 + 00	8831	.022[4]	.0089[4]	135.7[5]	189.7	N/A
Section 28, Desert Ridge	1758 + 75[1] 1773 + 25[1] 1787 + 50[1]	939[8] 799[8] 980[8]	- - -	- - -	- - -	- - -	3-10'x6' 3-8'x6' 3-10'x6'
52nd Street, Desert Ridge	1713 + 50[1]						
Rawhide Wash Channel	1812 + 00	13,384	.025	.0133[4]	131.2[5]	185.2	N/A
Local Drainage Along Scottsdale Road, South of Beardsley	1857 + 00	1,165	.025	.0154	12.4	66.4	N/A
Core North Detention Basin 38R-2	1890 + 50[9]	688	-	-	-	-	1-10'x4'
Core North Detention Basin 53R	1933 + 00[9] 1937 + 00[9]	1,796 722	-	-	-	-	2-10'x4' 1-10'x5'
Core South CP208 To CAP Dike 4 along NE curve of OLH	1963 + 50	381	-	-	-	-	2-6'x6'[7]

FOOTNOTES:

1. Data provided by BRW, Inc.
2. Discharges are based on Master Plan scenarios that ignore low-flow culvert water releases.
3. "n" values are those required to provide long-term, sediment transport equilibrium.
4. Slopes are set to provide long-term sediment transport equilibrium; does not apply to BRW data.
5. Bottomwidths are based on channel slopes that are at, or very near, the long-term equilibrium slope.
6. Includes two 15-foot wide maintenance roads and a 6-foot deep channel with 2:1 side slopes.
7. Assumes concrete box culvert operating under inlet control.
8. Based on HEC-1 model from April 1987 SLA report.
9. Data provided by Stanley Consultants, Inc.

NOTES: All data in this table is preliminary and subject to revision. This information should only be used for general planning purposes, not final design.

TABLE 5.2

PERFORMANCE SUMMARY
DETENTION BASIN 53R-CORE NORTH

Reata Pass Diversion	Maximum Reservoir W.S. Elevation (ft, MSL)	Storage at Maximum W.S. Elevation (Ac-Ft)	Maximum Outflow (cfs) (100-yr, 24-hr Event
To Core North	1622.43	1117	2277
To CAP Dike 4	1611.18	430	1478

NOTE: All low-flow culverts are inoperative.
Data taken from HEC-1 Files MP3.24I and MP9.24I.

TABLE 6.1

Jomax Road Channel

Equilibrium Slope Analysis, No Low-Flow Culverts

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: JOMAX2EQ.WK1

D50 Sediment Size (mm): 1.50

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	10-Year Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 407+90 to 386+90	2,100	0.0152	0.0150	163	0.0150	0.037	2.00	2.10	8.0	6.37	0.90	1.56	0.86	0.83	0.4	1	2100
STA 386+90 to 354+30	3,260	0.0153	0.0130	537	0.0130	0.045	2.00	2.37	32.6	6.07	0.74	2.10	2.79	2.71	7.5	3	1087
STA 354+30 to 319+40	3,490	0.0143	0.0140	1,073	0.0138	0.042	2.00	2.35	61.0	6.95	0.83	2.19	7.66	7.66	1.7	1	3490
STA 319+40 to 286+70	3,270	0.0153	0.0153	1,315	0.0155	0.040	2.00	2.43	63.7	7.90	0.92	2.27	12.66	12.69	-0.7	0	0
STA 286+70 to 230+60	5,610	0.0159	0.0140	2,599	0.0141	0.045	2.00	2.42	150.5	6.90	0.79	2.35	19.49	19.52	10.1	4	1403
Total:	17,730															9	

TABLE 6.2

Squaw Peak Parkway Channel

Equilibrium Slope Analysis, No Low-Flow Culverts

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: SQPK2EQ.WK1

D50 Sediment Size (mm): 1.50

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	10-Year Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 230+60 to 196+40	3,420	0.0091	0.0091	2,808	0.0087	0.027	2.00	2.47	120.1	9.09	1.04	2.38	20.81	20.88	1.4	1	3420
STA 196+40 to 163+30	3,310	0.0145	0.0145	2,629	0.0141	0.040	2.00	2.13	168.7	7.15	0.87	2.08	21.31	21.40	1.3	1	3310
STA 163+30 to 109+00	5,430	0.0114	0.0114	3,353	0.0112	0.028	2.00	2.39	138.3	9.79	1.13	2.32	35.20	35.07	1.1	1	5430
STA 109+00 to 75+30	3,370	0.0089	0.0089	3,666	0.0086	0.022	2.00	2.39	135.7	10.90	1.26	2.31	36.00	35.94	1.0	1	3370
STA 75+30 to 38+80	3,650	0.0082	0.0082	3,656	0.0081	0.020	2.00	2.38	128.2	11.57	1.35	2.29	37.30	36.94	0.4	1	3650
Total:	19,180															5	

TABLE 6.3

Pinnacle Peak Road Channel, East of Squaw Peak Parkway

Equilibrium Slope Analysis, No Low-Flow Culverts

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: PPE2EQ.WK1

D50 Sediment Size (mm): 1.50

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	10-Year Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 281+80 to 253+30	2,850	0.0095	0.0095	451	0.0093	0.030	2.00	2.63	17.1	7.67	0.93	2.13	2.23	2.22	0.6	1	2850
STA 253+30 to 220+50	3,280	0.0091	0.0091	790	0.0092	0.023	2.00	2.70	22.6	10.43	1.22	2.27	6.45	6.46	-0.3	0	0
STA 220+50 to 181+80	3,870	0.0078	0.0078	1,579	0.0076	0.025	2.00	2.69	57.3	9.37	1.05	2.48	10.25	10.15	0.8	1	3870
STA 181+80 to 163+30	1,850	0.0097	0.0097	1,737	0.0095	0.029	2.00	2.59	70.0	8.92	1.01	2.42	13.02	12.98	0.4	1	1850
Total:	11,850															3	

TABLE 6.4

Pinnacle Peak Road Channel, West of Squaw Peak Parkway

Equilibrium Slope Analysis, No Low-Flow Culverts

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: PPW2EQ.WK1

D50 Sediment Size (mm): 1.50

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	10-Year Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 172+20 to 144+10	2,810	0.0096	0.0055	468	0.0048	0.045	2.00	2.76	36.2	4.07	0.46	2.44	0.53	0.51	13.5	5	562
STA 144+10 to 124+30	1,980	0.0051	0.0051	668	0.0051	0.032	2.00	2.65	38.4	5.78	0.66	2.36	1.39	1.36	0.0	0	0
STA 124+30 to 105+60	1,870	0.0107	0.0086	713	0.0086	0.045	2.00	2.49	49.6	5.24	0.61	2.28	2.11	2.15	3.9	2	935
STA 105+60 to 32+50	7,310	0.0082	0.0082	1,367	0.0082	0.040	2.00	2.43	91.6	5.84	0.68	2.31	4.79	4.75	0.0	0	0
STA 32+50 to 10+00	2,250	0.0044	0.0044	1,299	0.0042	0.018	2.00	2.44	53.8	9.08	1.07	2.25	4.82	4.75	0.5	1	2250
Total:	16,220															8	

TABLE 6.5

Rawhide Wash Channel

Equilibrium Slope Analysis, No Low-Flow Culverts

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: RAW2EQ.WK1

D50 Sediment Size (mm): 1.50

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 335+60 to 293+00	4,260	0.0188	0.0188	4,431	0.0186	0.033	2.00	2.34	173.6	10.61	1.24	2.28	83.45	83.11	0.9	1	4260
STA 293+00 to 255+00	3,800	0.0237	0.0220	4,518	0.0219	0.040	2.00	2.34	199.0	9.50	1.11	2.28	83.22	83.11	6.8	3	1267
STA 255+00 to 212+70	4,230	0.0236	0.0220	4,580	0.0223	0.040	2.00	2.33	200.9	9.57	1.12	2.28	86.76	86.54	5.5	2	2115
STA 212+70 to 205+90	680	0.0295	0.0220	4,580	0.0223	0.040	2.00	2.33	200.9	9.57	1.12	2.28	86.76	86.54	4.9	2	340
STA 205+90 to 166+00	3,990	0.0201	0.0200	5,464	0.0202	0.035	2.00	2.13	255.5	9.86	1.20	2.10	101.17	101.23	-0.4	0	0
STA 166+00 to 58+10	10,790	0.0148	0.0133	5,341	0.0133	0.025	2.00	2.90	131.2	13.46	1.42	2.78	103.61	103.18	16.2	6	1798
STA 58+10 to 5+00	5,310	0.0113	0.0113	5,405	0.0136	0.025	2.00	2.73	144.9	13.16	1.43	2.64	104.61	104.95	-12.2	0	0
Total:	33,060															14	

Note: d=5' from STA 5+00 to 165+99

TABLE 6.6

Scottsdale Road Channel

Equilibrium Slope Analysis, No Low-Flow Culverts

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: SCT2EQ.WK1

D50 Sediment Size (mm): 1.50

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 335+30 to 309+10	2,620	0.0153	0.0153	266	0.0153	0.035	2.00	2.25	11.5	7.39	0.98	1.76	1.89	1.84	0.0	0	0
STA 309+10 to 297+20	1,190	0.0168	0.0168	378	0.0165	0.030	2.00	2.27	13.6	9.18	1.20	1.82	4.12	4.12	0.4	1	1190
STA 297+20 to 262+90	3,430	0.0175	0.0175	691	0.0175	0.040	2.00	2.28	34.4	7.78	0.96	2.04	6.95	7.01	0.0	0	0
STA 262+90 to 223+20	3,970	0.0151	0.0151	814	0.0151	0.026	2.00	2.26	28.5	10.90	1.36	1.99	11.90	11.84	0.0	0	0
STA 223+20 to 203+20	2,000	0.0228	0.0228	810	0.0223	0.040	2.00	2.26	36.2	8.78	1.08	2.04	12.26	12.14	1.0	1	2000
Total:	13,210															2	

TABLE 6.7

Pima Road Channel

Equilibrium Slope Analysis, No Low-Flow Culverts

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: PIMA2EQ.WK1

D50 Sediment Size (mm): 1.50

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	10-Year Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 297+20 to 275+70	2,150	0.0233	0.0233	277	0.0233	0.020	2.00	1.78	8.0	13.44	2.03	1.36	7.25	6.96	0.0	0	0
STA 275+70 to 250+40	2,530	0.0237	0.0190	839	0.0183	0.045	2.00	2.40	42.5	7.40	0.88	2.18	8.20	8.22	13.7	5	506
STA 250+40 to 237+50	1,290	0.0155	0.0150	979	0.0151	0.035	2.00	2.37	43.2	8.60	1.03	2.16	10.23	10.26	0.5	1	1290
STA 237+50 to 218+90	1,860	0.0161	0.0160	1,207	0.0165	0.035	2.00	2.19	58.7	8.72	1.07	2.05	14.50	14.55	-0.7	0	0
STA 218+90 to 179+60	3,930	0.0178	0.0170	1,665	0.0170	0.035	2.00	2.21	79.5	9.00	1.09	2.10	21.95	21.87	3.1	2	1965
STA 179+60 to 163+70	1,590	0.0189	0.0188	2,065	0.0191	0.045	2.00	2.20	120.5	7.51	0.91	2.13	23.25	23.34	-0.3	0	0
STA 163+70 to 126+90	3,680	0.0245	0.0203	2,048	0.0203	0.045	2.00	2.21	114.9	7.75	0.94	2.13	25.52	25.45	15.5	6	613
STA 126+90 to 78+00	4,890	0.0225	0.0191	2,404	0.0191	0.045	2.00	2.17	143.5	7.48	0.91	2.11	27.12	27.07	16.6	6	815
STA 78+00 to 17+00	6,100	0.0205	0.0200	2,361	0.0200	0.045	2.00	2.18	137.2	7.66	0.93	2.11	28.71	28.85	3.1	2	3050
Total:	28,020															22	

TABLE 6.8

Deer Valley Road Channel, Pima Rd to Hayden Rd

Equilibrium Slope Analysis, Pima Road Channel Is Not In Place

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: DVEQ.WK1

D50 Sediment Size (mm): 1.50

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	10-Year Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 56+50 to 49+20	730	0.0192	0.0192	51	0.0192	0.030	2.00	0.92	8.0	5.65	1.13	0.77	0.35	0.34	0.0	0	0
STA 49+20 to 34+70	1,450	0.0138	0.0138	331	0.0138	0.030	2.00	2.54	10.1	8.60	1.10	1.90	2.56	2.54	0.0	0	0
STA 34+70 to 16+40	1,830	0.0109	0.0109	696	0.0105	0.025	2.00	2.46	24.0	9.78	1.19	2.10	5.97	5.86	0.7	1	1830
STA 16+40 to 5+00	1,140	0.0132	0.0132	849	0.0117	0.030	2.00	2.50	33.1	8.92	1.06	2.21	7.21	7.24	1.7	1	1140
Total:	5,150															2	

TABLE 6.9

Deer Valley Road Channel, Hayden Rd to Sci Rd to Beardsley Rd

Equilibrium Slope Analysis, No Upstream Drainage Improvements Are In-Place

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: DVSEQ.WK1

D50 Sediment Size (mm): 1.50

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	10-Year Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 101+20 to 95+10	610	0.0066	0.0066	332	0.0066	0.018	2.00	2.62	7.8	9.73	1.26	1.87	1.53	3.08	0.0	0	0
STA 95+10 to 83+30	1,180	0.0085	0.0085	853	0.0080	0.018	2.00	2.52	23.2	11.98	1.44	2.14	7.79	7.64	0.6	1	1180
STA 83+30 to 63+30	2,000	0.0100	0.0100	1,579	0.0100	0.030	2.00	2.42	71.8	8.50	0.99	2.28	11.65	11.84	0.0	0	0
STA 63+30 to 53+70	960	0.0104	0.0104	2,261	0.0100	0.027	2.00	2.39	94.9	9.48	1.11	2.28	20.11	20.08	0.4	1	960
STA 53+70 to 22+80	3,090	0.0162	0.0162	2,279	0.0158	0.045	2.00	2.38	128.0	7.20	0.84	2.30	20.11	20.08	1.2	1	3090
STA 22+80 to 5+00	1,780	0.0169	0.0169	2,296	0.0156	0.045	2.00	2.43	126.0	7.23	0.83	2.34	20.10	20.08	2.3	1	1780
Total:	9,620															4	

TABLE 6.10

Reata Pass Channel

Equilibrium Slope Analysis, No Low-Flow Culverts

Upper Indian Bend Wash Regional Flood Control & Drainage Plan

10-Year Event

File: REATA2EQ.WK1

D50 Sediment Size (mm): 2.00

Sediment Gradation Coefficient: 4.00

Assumed Drop Structure Height: 3 ft

Channel Reach	Reach Length (ft)	Existing Ground Slope (ft/ft)	Channel Design Slope (ft/ft)	10-Year Discharge (cfs)	Equilibrium Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Hydraulic Depth (ft)	Sediment Transport Capacity (cfs)	Target Qs (cfs)	Total Vertical Drop Distance (ft)	Number of Drop Structures	Drop Structure Spacing (ft)
STA 291+80 to 276+10	1,570	0.0255	0.0255	7,560	0.0255	0.025	5.00	3.88	75.0	20.61	2.02	3.22	360.99	359.67	0.0	0	0
STA 276+10 to 257+10	1,900	0.0421	0.0255	7,654	0.0255	0.026	5.00	3.96	75.0	20.42	1.99	3.27	358.57	359.67	31.5	11	173
STA 257+10 to 233+80	2,330	0.0258	0.0258	7,640	0.0258	0.024	5.00	3.81	75.0	21.35	2.11	3.17	391.09	380.74	0.0	0	0
STA 233+80 to 189+80	4,400	0.0309	0.0270	7,433	0.0270	0.025	5.00	3.79	75.0	20.90	2.07	3.15	383.85	380.74	17.2	6	733
STA 189+80 to 179+30	1,050	0	0.0270	7,381	0.0270	0.025	5.00	3.77	75.0	20.85	2.07	3.14	380.18	380.74	-28.4	0	0
STA 179+30 to 153+50	2,580	0.0209	0.0209	9,182	0.0250	0.025	5.00	4.36	75.0	21.79	2.04	3.56	456.34	454.16	-10.6	0	0
STA 153+50 to 116+10	3,740	0.0347	0.0285	8,721	0.0285	0.025	5.00	4.08	75.0	22.41	2.15	3.36	516.85	516.05	23.2	8	468
STA 116+10 to 103+10	1,300	0	0.0280	8,877	0.0280	0.025	5.00	4.14	75.0	22.40	2.14	3.40	515.73	517.33	-36.4	0	0
STA 103+10 to 80+10	2,300	0.0087	0.0286	8,708	0.0286	0.025	5.00	4.07	75.0	22.43	2.16	3.36	518.48	518.11	-45.8	0	0
STA 80+10 to 68+60	1,150	0.0174	0.0305	9,360	0.0305	0.025	5.00	4.16	75.0	23.46	2.24	3.42	629.02	627.63	-15.1	0	0
STA 68+60 to 52+90	1570	0.0127	0.0304	9,425	0.0304	0.025	5.00	4.18	75.0	23.48	2.23	3.44	631.87	629.46	-27.8	0	0
STA 52+90 to 27+70	2520	0.0238	0.0306	9,338	0.0306	0.025	5.00	4.15	75.0	23.47	2.24	3.41	630.07	631.65	-17.1	0	0
STA 27+70 to 0+00	2770	0.0217	0.0318	8,963	0.0318	0.025	5.00	4.02	75.0	23.47	2.27	3.32	630.91	632.06	-28.0	0	0
Total:	29,180															25	

TABLE 7.1

SUMMARY OF CAP, DETENTION BASIN PERFORMANCE EXISTING DRAINAGE PATTERN VERSUS PROPOSED REGIONAL DRAINAGE PLAN
100 - YEAR, 24-HOUR EVENT

STRUCTURE	COMPUTER MODEL	EXISTING DRAINAGE PATTERN			WITH REGIONAL DRAINAGE PLAN		
		PEAK INFLOW	MAXIMUM RESERVOIR STAGE (MSL)	STORAGE VOLUME AT MAXIMUM RESERVOIR STAGE (A.F.)	PEAK INFLOW	MAXIMUM RESERVOIR STAGE (MSL)	STORAGE VOLUME AT MAXIMUM RESERVOIR STAGE (A.F)
CAP Dike 1	PSJ3 F3	10,388	1,535.77	3,377.0	14,148	1,535.75	3,365.0
CAP Dike 2	MP3 F3	13,250	1,539.58	4,576.0	18,689	1,538.81	4,195.0
CAP Dike 3 (Reata Pass Diverted to Dike 4)	MP9 F4	6,170	1,527.81	1,542.0	5,910	1,529.16	1,891.0
CAP Dike 3 (Reata Pass Diverted to Dike 3)	MP3 F3	12,429	1,531.89	2,663.0	5,910	1,533.13	3,053.0
CAP Dike 4 (Reata Pass Diverted to Dike 4)	CAP4 RP5	23,608	1,527.36	3,848.0	25,067	1,527.24	3,807.0
CAP Dike 4 (Reata Pass Diverted to Dike 3)	CP4R RP6	23,290	1,523.70	2,701.0	21,120	1,523.72	2,707.0

TABLE 9.1

WATERSHED CONDITIONS USED FOR DEVELOPING COST ESTIMATES

TABLE NUMBER	DRAINAGE AREA SIZE USED FOR RAINFALL REDUCTION	LOW-FLOW CULVERTS IN-PLACE	CHANNEL SLOPE[1]	REATA PASS DIVERSION	SPREADSHEET FILE
9.2	25	Yes	Existing Ground	Manmade Channel To CAP Dike 4	MPCOST2.WK1
9.3	25	No	Existing Ground	Manmade Channel To CAP Dike 4	MPCOST3.WK1
9.4	25	Yes	Existing Ground	Naural Flow To CAP Dike 3	MPCOST4.WK1
9.5	25	No	Existing Ground	Natural Flow To CAP Dike 3	MPCOST5.WK1

NOTES:

1) Reata Pass Channel uses a 2% slope

Table 9.2

Summary Of Project Installation Costs

Upper Indian Bend Wash Regional Drainage Plan

100-Year, 24-Hour Event

Scenario No. 1

Channel System	Channel Length (ft)	Total Excavation Cost	Total Lining Cost	Top-of-Bank Cutoff Wall (both sides)	Drop Structure Allowance	Low-Flow Culverts	Low-Flow Channel	Channel Access Ramps	Bridges	Construction Cost Subtotal	Right-Of-Way Requirements (acres)	Right-Of-Way Cost @ $50,000/ac.	Landscaping Cost @ $7,500/ac.	Engineering Services @ 7% of Construction Cost	Subtotal, With Landscaping, Right-Of-Way, & Engineering	Contingencies @ 20%	Total Estimated Installation Cost
Pima Road	28,020	$1,489,179	$4,782,080	$298,693	$319,348	$5,341,144	$149,347	$53,068	$1,471,483	$13,904,342	71.8	$3,588,721	$538,308	$973,304	$19,004,675	$3,800,935	$22,805,610
Rawhide Wash	33,060	4,778,161	5,642,240	352,420	401,695	0	176,210	62,614	6,095,178	17,508,517	240.7	12,036,709	1,805,506	1,225,596	32,576,329	6,515,266	39,091,594
Scottsdale Road	13,210	513,411	2,254,507	140,819	8,188	1,218,156	70,409	25,019	619,938	4,850,446	24.1	1,204,655	180,698	339,531	6,575,330	1,315,066	7,890,397
Deer Valley, Hayden to Pima	5,150	191,895	878,933	54,899	9,234	0	27,450	9,754	0	1,172,164	9.0	448,304	67,246	82,051	1,769,766	353,953	2,123,719
Deer Valley, Hayden to Scottsdale to Beardsley	9,620	476,045	1,641,813	102,549	53,490	0	51,275	18,220	342,936	2,686,328	22.8	1,141,118	171,168	188,043	4,186,657	837,331	5,023,989
Jomax Road, to Pinnacle Peak Rd	24,460	1,453,301	4,174,507	260,744	141,191	3,701,319	130,372	46,326	1,731,960	11,639,718	70.6	3,528,749	529,312	814,780	16,512,560	3,302,512	19,815,072
Pinnacle Peak Road, East & South of Intxn Squaw Peak Parkway	24,300	2,018,960	4,147,200	259,038	86,060	1,358,712	129,519	46,023	3,432,105	11,477,616	99.8	4,991,115	748,667	803,433	18,020,832	3,604,166	21,624,998
Pinnacle Peak Road, West of Squaw Peak Parkway	16,220	883,492	2,768,213	172,905	40,480	1,405,564	86,453	30,720	1,728,666	7,116,493	42.7	2,132,801	319,920	498,154	10,067,368	2,013,474	12,080,842
Reata Pass, Along Thompson Peak Parkway	29,180	5,929,862	5,707,478	311,059	1,620,375	1,452,416	155,529	55,265	5,720,220	20,952,205	240.7	12,036,826	1,805,524	1,466,654	36,261,209	7,252,242	43,513,451
TOTALS:	183,220	$17,734,305	$31,996,972	$1,953,125	$2,680,061	$14,477,310	$976,563	$347,008	$21,142,486	$91,307,829	822.2	$41,108,999	$6,166,350	$6,391,548	$144,974,727	$28,994,945	$173,969,672

Assumptions: *1. Low-flow culverts are installed.* *2. Areal reduction in rainfall for 25 sq mi* *3. Reata Pass routed to CAP Dike 4.* *4. Channels have soil-cement banks & earth bottoms.* *5. Drop structure costs based on preliminary equilibrium slope analysis, with no low-flow culverts* *6. Channel slopes match existing ground profile, except for Reata Pass, which is set at 0.0200 ft/ft.*

File: MPCOST2.WK1

Table 9.3

Summary Of Project Installation Costs

Upper Indian Bend Wash Regional Drainage Plan

100-Year, 24-Hour Event

Scenario No. 2

Channel System	Channel Length (ft)	Total Excavation Cost	Total Lining Cost	Top-of-Bank Cutoff Wall (both sides)	Drop Structure Allowance	Low-Flow Culverts	Low-Flow Channel	Channel Access Ramps	Bridges	Construction Cost Subtotal	Right-Of-Way Requirements (acres)	Right-Of-Way Cost @ $50,000/ac.	Landscaping Cost @ $7,500/ac.	Engineering Services @ 7% of Construction Cost	Subtotal, With Landscaping, Right-Of-Way, & Engineering	Contingencies @ 20%	Total Estimated Installation Cost
Pima Road	28,020	$1,905,019	$4,782,080	$298,693	$319,348	$0	$149,347	$53,068	$1,471,483	$8,979,038	93.3	$4,662,687	$699,403	$628,533	$14,969,660	$2,993,932	$17,963,592
Rawhide Wash	33,060	4,918,373	5,642,240	352,420	401,695	0	176,210	62,614	6,095,178	17,648,729	248.0	12,398,828	1,859,824	1,235,411	33,142,792	6,628,558	39,771,351
Scottsdale Road	13,210	565,201	2,254,507	140,819	8,188	0	70,409	25,019	619,938	3,684,080	26.8	1,338,409	200,761	257,886	5,481,136	1,096,227	6,577,363
Deer Valley, Hayden to Pima	5,150	185,626	878,933	54,899	9,234	0	27,450	9,754	0	1,165,895	8.6	432,114	64,817	81,613	1,744,439	348,888	2,093,327
Deer Valley, Hayden to Scottsdale to Beardsley	9,620	426,336	1,641,813	102,549	53,490	0	51,275	18,220	342,936	2,636,619	20.3	1,012,737	151,911	184,563	3,985,830	797,166	4,782,995
Jomax Road, to Pinnacle Peak Rd	24,460	1,743,591	4,174,507	260,744	141,191	0	130,372	46,326	1,731,960	8,228,690	85.6	4,278,466	641,770	576,008	13,724,935	2,744,987	16,469,921
Pinnacle Peak Road, East & South of Intcn Squaw Peak Parkway	24,300	2,357,091	4,147,200	259,038	86,060	0	129,519	46,023	3,432,105	10,457,036	117.3	5,864,388	879,658	731,992	17,933,074	3,586,615	21,519,689
Pinnacle Peak Road, West of Squaw Peak Parkway	16,220	1,036,650	2,768,213	172,905	40,480	0	86,453	30,720	1,728,666	5,864,086	50.6	2,528,354	379,253	410,486	9,182,179	1,836,436	11,018,615
Reata Pass, Along Thompson Peak Parkway	29,180	6,128,616	5,707,478	311,059	1,620,375	0	155,529	55,265	5,720,220	19,698,543	242.4	12,121,975	1,818,296	1,378,898	35,017,713	7,003,543	42,021,255
TOTALS:	183,220	$19,266,502	$31,996,972	$1,953,125	$2,680,061	$0	$976,563	$347,008	$21,142,486	$78,362,716	892.8	$44,637,958	$6,695,694	$5,485,390	$135,181,757	$27,036,351	$162,218,109

Assumptions: *1. No low-flow culverts are installed.* *2. Areal reduction in rainfall for 25 sq mi* *3. Reata Pass routed to CAP Dike 4.* *4. Channels have soil-cement banks & earth bottoms.* *5. Drop structure costs based on preliminary equilibrium slope analysis, with no low-flow culverts* *6. Channel slopes match exirting ground profile, except for Reata Pass, which is set at 0.0200 ft/ft.*

File: MPCOST3.WK1

Table 9.4

Summary Of Project Installation Costs

Upper Indian Bend Wash Regional Drainage Plan

100-Year, 24-Hour Event

Scenario No. 3

Channel System	Channel Length (ft)	Total Excavation Cost	Total Lining Cost	Top-of-Bank Cutoff Wall (both sides)	Drop Structure Allowance	Low-Flow Culverts	Low-Flow Channel	Channel Access Ramps	Bridges	Construction Cost Subtotal	Right-Of-Way Requirements (acres)	Right-Of-Way Cost @ $50,000/ac.	Landscaping Cost @ $7,500/ac.	Engineering Services @ 7% of Construction Cost	Subtotal, With Landscaping, Right-Of-Way, & Engineering	Contingencies @ 20%	Total Estimated Installation Cost
Pima Road	28,020	$2,967,030	$4,782,080	$298,693	$319,348	$5,341,144	$149,347	$53,068	$1,471,483	$15,382,192	148.1	$7,405,484	$1,110,823	$1,076,753	$24,975,252	$4,995,050	$29,970,303
Rawhide Wash	33,060	4,778,161	5,642,240	352,420	401,695	0	176,210	62,614	6,095,178	17,508,517	240.7	12,036,709	1,805,506	1,225,596	32,576,329	6,515,266	39,091,594
Scottsdale Road	13,210	513,411	2,254,507	140,819	8,188	1,218,156	70,409	25,019	619,938	4,850,446	24.1	1,204,655	180,698	339,531	6,575,330	1,315,066	7,890,397
Deer Valley, Hayden to Pima	5,150	191,895	878,933	54,899	9,234	0	27,450	9,754	0	1,172,164	9.0	448,304	67,246	82,051	1,769,766	353,953	2,123,719
Deer Valley, Hayden to Scottsdale to Beardsley	9,620	476,045	1,641,813	102,549	53,490	0	51,275	18,220	342,936	2,686,328	22.8	1,141,118	171,168	188,043	4,186,657	837,331	5,023,989
Jomax Road, to Pinnacle Peak Rd	24,460	1,453,301	4,174,507	260,744	141,191	3,701,319	130,372	46,326	1,731,960	11,639,718	70.6	3,528,749	529,312	814,780	16,512,560	3,302,512	19,815,072
Pinnacle Peak Road, East & South of Intxn Squaw Peak Parkway	24,300	2,018,960	4,147,200	259,038	86,060	1,358,712	129,519	46,023	3,432,105	11,477,616	99.8	4,991,115	748,667	803,433	18,020,832	3,604,166	21,624,998
Pinnacle Peak Road, West of Squaw Peak Parkway	16,220	883,492	2,768,213	172,905	40,480	1,405,564	86,453	30,720	1,728,666	7,116,493	42.7	2,132,801	319,920	498,154	10,067,368	2,013,474	12,080,842
Reata Pass, Along Thompson Peak Parkway	0	0	0	0	0	0	0	0	0	0	0.0	0	0	0	0	0	0
TOTALS:	154,040	$13,282,294	$26,289,493	$1,642,066	$1,059,686	$13,024,894	$821,033	$291,742	$15,422,266	$71,833,475	657.8	$32,888,936	$4,933,340	$5,028,343	$114,684,095	$22,936,819	$137,620,914

Assumptions: *1. Low-flow culverts are installed.* *2. Areal reduction in rainfall for 25 sq mi* *3. Reata Pass routed to CAP Dike 3.* *4. Channels have soil-cement banks & earth bottoms.* *5. Drop structure costs based on preliminary equilibrium slope analysis, with no low-flow culverts* *6. Channel slopes match existing ground profile, except for Reata Pass, which is set at 0.0200 ft/ft.*

File: MPCOST4.WK1

Table 9.5

Summary Of Project Installation Costs

Upper Indian Bend Wash Regional Drainage Plan

100-Year, 24-Hour Event

Scenario No. 4

Channel System	Channel Length (ft)	Total Excavation Cost	Total Lining Cost	Top-of-Bank Cutoff Wall (both sides)	Drop Structure Allowance	Low-Flow Culverts	Low-Flow Channel	Channel Access Ramps	Bridges	Construction Cost Subtotal	Right-Of-Way Requirements (acres)	Right-Of-Way Cost @ $50,000/ac.	Landscaping Cost @ $7,500/ac.	Engineering Services @ 7% of Construction Cost	Subtotal, With Landscaping, Right-Of-Way, & Engineering	Contingencies @ 20%	Total Estimated Installation Cost
Pima Road	28,020	$3,389,254	$4,782,080	$298,693	$319,348	$0	$149,347	$53,068	$1,471,483	$10,463,273	169.9	$8,495,939	$1,274,391	$732,429	$20,966,031	$4,193,206	$25,159,237
Rawhide Wash	33,060	4,918,373	5,642,240	352,420	401,695	0	176,210	62,614	6,095,178	17,648,729	248.0	12,398,828	1,859,824	1,235,411	33,142,792	6,628,558	39,771,351
Scottsdale Road	13,210	565,201	2,254,507	140,819	8,188	0	70,409	25,019	619,938	3,684,080	26.8	1,338,409	200,761	257,886	5,481,136	1,096,227	6,577,363
Deer Valley, Hayden to Pima	5,150	185,626	878,933	54,899	9,234	0	27,450	9,754	0	1,165,895	8.6	432,114	64,817	81,613	1,744,439	348,888	2,093,327
Deer Valley, Hayden to Scottsdale to Beardsley	9,620	426,336	1,641,813	102,549	53,490	0	51,275	18,220	342,936	2,636,619	20.3	1,012,737	151,911	184,563	3,985,830	797,166	4,782,995
Jomax Road, to Pinnacle Peak Rd	24,460	1,743,591	4,174,507	260,744	141,191	0	130,372	46,326	1,731,960	8,228,690	85.6	4,278,466	641,770	576,008	13,724,935	2,744,987	16,469,921
Pinnacle Peak Road, East & South of Intxn Squaw Peak Parkway	24,300	2,357,091	4,147,200	259,038	86,060	0	129,519	46,023	3,432,105	10,457,036	117.3	5,864,388	879,658	731,992	17,933,074	3,586,615	21,519,689
Pinnacle Peak Road, West of Squaw Peak Parkway	16,220	1,036,650	2,768,213	172,905	40,480	0	86,453	30,720	1,728,666	5,864,086	50.6	2,528,354	379,253	410,486	9,182,179	1,836,436	11,018,615
Reata Pass, Along Thompson Peak Parkway	0	0	0	0	0	0	0	0	0	0	0.0	0	0	0	0	0	0
TOTALS:	154,040	$14,622,121	$26,289,493	$1,642,066	$1,059,686	$0	$821,033	$291,742	$15,422,266	$60,148,408	727.0	$36,349,234	$5,452,385	$4,210,389	$106,160,415	$21,232,083	$127,392,498

Assumptions: *1. No low-flow culverts are installed.* *2. Areal reduction in rainfall for 25 sq mi* *3. Reata Pass routed to CAP Dike 3.* *4. Channels have soil-cement banks & earth bottoms.* *5. Drop structure costs based on preliminary equilibrium slope analysis, with no low-flow culverts* *6. Channel slopes match existing ground profile, except for Reata Pass, which is set at 0.0200 ft/ft.*

File: MPCOST5.WK1

APPENDICES

File: JOMAX4.WK1

TABLE A.1
Hydraulic Data & Cost Estimates for Jomax Road Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Channel Depth With Freeboard	Channel Topwidth With Freeboard	Approximate Toedown Depth (ft)	Unit Excavation Volume (cy/LF)
Start @ 163+30													
STA 163+30 to 168+30	500	5,680	0.0160	0.045	2.00	4.0	133.1	10.1	0.91	6.0	157.1	6.0	43.36
STA 168+30 to 175+60	730	5,680	0.0137	0.045	2.00	4.0	144.1	9.3	0.84	6.0	168.1	6.0	45.79
STA 175+60 to 183+60	800	5,680	0.0125	0.045	2.00	4.0	150.9	8.9	0.81	6.0	174.9	6.0	47.31
STA 183+60 to 191+50	790	5,680	0.0127	0.045	2.00	4.0	149.9	9.0	0.81	6.0	173.9	6.0	47.10
STA 191+50 to 196+40	490	5,680	0.0204	0.045	2.00	4.0	117.6	11.3	1.03	6.0	141.6	6.0	39.92
STA 196+40 to 208+70	1,230	6,008	0.0081	0.045	2.00	4.0	198.5	7.3	0.65	6.0	222.5	6.0	57.90
STA 208+70 to 218+70	1,000	6,008	0.0100	0.045	2.00	4.0	178.8	8.0	0.72	6.0	202.8	6.0	53.52
STA 218+70 to 228+50	980	6,008	0.0102	0.045	2.00	4.0	177.0	8.1	0.73	6.0	201.0	6.0	53.11
STA 228+50 to 230+60	210	6,008	0.0048	0.045	2.00	4.0	260.0	5.6	0.50	6.0	284.0	6.0	71.55
STA 230+60 to 238+00	740	5,596	0.0122	0.045	2.00	4.0	150.7	8.8	0.80	6.0	174.7	6.0	47.27
STA 238+00 to 245+50	750	5,596	0.0133	0.045	2.00	4.0	143.9	9.2	0.83	6.0	167.9	6.0	45.75
STA 245+50 to 251+50	600	5,596	0.0167	0.045	2.00	4.0	128.4	10.3	0.93	6.0	152.4	6.0	42.32
STA 251+50 to 258+50	700	5,596	0.0143	0.045	2.00	4.0	138.9	9.5	0.86	6.0	162.9	6.0	44.65
STA 258+50 to 263+30	480	5,596	0.0208	0.045	2.00	4.0	114.7	11.4	1.04	6.0	138.7	6.0	39.26
STA 263+30 to 269+70	640	5,596	0.0156	0.045	2.00	4.0	132.7	9.9	0.90	6.0	156.7	6.0	43.27
STA 269+70 to 277+50	780	5,596	0.0128	0.045	2.00	4.0	146.7	9.0	0.82	6.0	170.7	6.0	46.39
STA 277+50 to 283+50	600	5,596	0.0167	0.045	2.00	4.0	128.4	10.3	0.93	6.0	152.4	6.0	42.32
STA 283+50 to 286+70	320	5,596	0.0313	0.045	2.00	4.0	93.2	13.8	1.27	6.0	117.2	6.0	34.49
STA 286+70 to 293+20	650	2,837	0.0154	0.045	2.00	4.0	66.7	9.5	0.88	6.0	90.7	6.0	28.59
STA 293+20 to 298+60	540	2,837	0.0185	0.045	2.00	4.0	60.5	10.4	0.96	6.0	84.5	6.0	27.23
STA 298+60 to 305+20	660	2,837	0.0152	0.045	2.00	4.0	67.2	9.4	0.87	6.0	91.2	6.0	28.71
STA 305+20 to 311+50	630	2,837	0.0159	0.045	2.00	4.0	65.6	9.6	0.89	6.0	89.6	6.0	28.35
STA 311+50 to 319+40	790	2,837	0.0127	0.045	2.00	4.0	73.8	8.7	0.80	6.0	97.8	6.0	30.17
STA 319+40 to 327+20	780	2,468	0.0128	0.045	2.00	4.0	63.4	8.6	0.80	6.0	87.4	6.0	27.87
STA 327+20 to 334+60	740	2,468	0.0135	0.045	2.00	4.0	61.7	8.9	0.82	6.0	85.7	6.0	27.49
STA 334+60 to 341+70	710	2,468	0.0141	0.045	2.00	4.0	60.4	9.0	0.84	6.0	84.4	6.0	27.19
STA 341+70 to 348+30	660	2,468	0.0152	0.045	2.00	4.0	58.1	9.3	0.87	6.0	82.1	6.0	26.69
STA 348+30 to 354+30	600	2,468	0.0167	0.045	2.00	4.0	55.3	9.8	0.91	6.0	79.3	6.0	26.06
STA 354+30 to 359+50	520	1,234	0.0192	0.045	2.00	4.0	24.0	9.6	0.95	6.0	48.0	6.0	19.11
STA 359+50 to 366+30	680	1,234	0.0147	0.045	2.00	4.0	27.9	8.6	0.84	6.0	51.9	6.0	19.99
STA 366+30 to 373+40	710	1,234	0.0141	0.045	2.00	4.0	28.6	8.4	0.82	6.0	52.6	6.0	20.14
STA 373+40 to 379+80	640	1,234	0.0156	0.045	2.00	4.0	27.0	8.8	0.86	6.0	51.0	6.0	19.78
STA 379+80 to 386+90	710	1,234	0.0141	0.045	2.00	4.0	28.6	8.4	0.82	6.0	52.6	6.0	20.14
STA 386+90 to 394+40	750	408	0.0133	0.045	2.00	4.0	6.9	6.8	0.75	6.0	30.9	6.0	15.31
STA 394+40 to 401+10	670	408	0.0149	0.045	2.00	4.0	6.3	7.2	0.79	6.0	30.3	6.0	15.17
STA 401+10 to 406+90	580	408	0.0172	0.045	2.00	4.0	5.5	7.6	0.84	6.0	29.5	6.0	14.99
STA 406+90 to 407+90	100	408	0.0200	0.045	2.00	4.0	4.7	8.0	0.90	6.0	28.7	6.0	14.83
	24,460												

TABLE A.1
Hydraulic Data & Cost Estimates for Jonah Road Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF

Unit Excavation Cost ($/LF)	Total Channel Excavation Volume For This Reach (cy)	Unit Channel Lining Volume (cy/LF)	Unit Channel Lining Cost ($/LF)	Total Channel Lining Volume For This Reach (cy)	Total Excavation Cost For This Channel Reach	Total Lining Cost For This Channel Reach	Total Channel Construction Cost For This Reach	Channel Right-of-Way Requirements With Two 15 ft Maintenance Roads (acres)	Channel Right-of-Way Cost @ $50,000/Acre
$86.73	21,682	7.11	$170.67	3,556	$43,365	$85,333	$128,698	2.1	$107,404
$91.58	33,427	7.11	$170.67	5,191	$66,855	$124,587	$191,441	3.3	$165,958
$94.62	37,849	7.11	$170.67	5,689	$75,699	$136,533	$212,232	3.8	$188,157
$94.20	37,208	7.11	$170.67	5,618	$74,416	$134,827	$209,243	3.7	$184,936
$79.84	19,561	7.11	$170.67	3,484	$39,122	$83,627	$122,749	1.9	$96,540
$115.79	71,212	7.11	$170.67	8,747	$142,423	$209,920	$352,343	7.1	$356,534
$107.03	53,516	7.11	$170.67	7,111	$107,031	$170,667	$277,698	5.3	$267,241
$106.22	52,050	7.11	$170.67	6,969	$104,099	$167,253	$271,353	5.2	$259,852
$143.10	15,025	7.11	$170.67	1,493	$30,051	$35,840	$65,891	1.5	$75,682
$94.54	34,981	7.11	$170.67	5,262	$69,961	$126,293	$196,255	3.5	$173,890
$91.49	34,309	7.11	$170.67	5,333	$68,619	$128,000	$196,619	3.4	$170,331
$84.64	25,393	7.11	$170.67	4,267	$50,786	$102,400	$153,186	2.5	$125,654
$89.29	31,252	7.11	$170.67	4,978	$62,505	$119,467	$181,972	3.1	$155,000
$78.52	18,844	7.11	$170.67	3,413	$37,688	$81,920	$119,608	1.9	$92,926
$86.55	27,695	7.11	$170.67	4,551	$55,390	$109,227	$164,617	2.7	$137,176
$92.78	36,183	7.11	$170.67	5,547	$72,365	$133,120	$205,485	3.6	$179,731
$84.64	25,393	7.11	$170.67	4,267	$50,786	$102,400	$153,186	2.5	$125,654
$68.98	11,037	7.11	$170.67	2,276	$22,074	$54,613	$76,687	1.1	$54,070
$57.18	18,584	7.11	$170.67	4,622	$37,168	$110,933	$148,102	1.8	$90,024
$54.46	14,703	7.11	$170.67	3,840	$29,406	$92,160	$121,566	1.4	$70,986
$57.42	18,948	7.11	$170.67	4,693	$37,896	$112,640	$150,536	1.8	$91,811
$56.70	17,862	7.11	$170.67	4,480	$35,724	$107,520	$143,244	1.7	$86,478
$60.33	23,832	7.11	$170.67	5,618	$47,664	$134,827	$182,490	2.3	$115,844
$55.73	21,736	7.11	$170.67	5,547	$43,472	$133,120	$176,592	2.1	$105,111
$54.97	20,339	7.11	$170.67	5,262	$40,679	$126,293	$166,972	2.0	$98,263
$54.39	19,307	7.11	$170.67	5,049	$38,614	$121,173	$159,787	1.9	$93,205
$53.38	17,615	7.11	$170.67	4,693	$35,231	$112,640	$147,871	1.7	$84,929
$52.12	15,637	7.11	$170.67	4,267	$31,273	$102,400	$133,673	1.5	$75,257
$38.22	9,937	7.11	$170.67	3,698	$19,874	$88,747	$108,621	0.9	$46,552
$39.98	13,592	7.11	$170.67	4,836	$27,185	$116,053	$143,238	1.3	$63,964
$40.28	14,300	7.11	$170.67	5,049	$28,600	$121,173	$149,774	1.3	$67,344
$39.56	12,659	7.11	$170.67	4,551	$25,318	$109,227	$134,545	1.2	$59,511
$40.28	14,300	7.11	$170.67	5,049	$28,600	$121,173	$149,774	1.3	$67,344
$30.62	11,484	7.11	$170.67	5,333	$22,968	$128,000	$150,968	1.0	$52,431
$30.34	10,163	7.11	$170.67	4,764	$20,325	$114,347	$134,672	0.9	$46,340
$29.99	8,697	7.11	$170.67	4,124	$17,393	$98,987	$116,380	0.8	$39,594
$29.65	1,483	7.11	$170.67	711	$2,965	$17,067	$20,032	0.1	$6,740
	871,796			173,938	$1,743,591	$4,174,507	$5,918,098	85.6	$4,278,466

File: PPE4.WK1

TABLE A.2
Hydraulic Data & Cost Estimates for Pinnacle Peak Road Channel (East of Squaw Peak Parkway), No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Channel Depth With Freeboard	Channel Topwidth With Freeboard	Approximate Toedown Depth (ft)	Unit Excavation Volume (cy/LF)
Start @ 38+80													
STA 38+80 to 48+60	980	8,186	0.0102	0.045	2.00	4.0	241.9	8.2	0.73	6.0	265.9	6.0	67.52
STA 48+60 to 63+50	1,490	8,186	0.0067	0.045	2.00	4.0	298.6	6.7	0.60	6.0	322.6	6.0	80.14
STA 63+50 to 75+30	1,180	8,186	0.0085	0.045	2.00	4.0	265.6	7.5	0.67	6.0	289.6	6.0	72.79
STA 75+30 to 86+60	1,130	8,207	0.0088	0.045	2.00	4.0	260.5	7.6	0.68	6.0	284.5	6.0	71.67
STA 86+60 to 99+00	1,240	8,207	0.0081	0.045	2.00	4.0	273.0	7.3	0.65	6.0	297.0	6.0	74.44
STA 99+00 to 109+00	1,000	8,207	0.0100	0.045	2.00	4.0	245.0	8.1	0.73	6.0	269.0	6.0	68.21
STA 109+00 to 117+00	800	7,427	0.0125	0.045	2.00	4.0	197.9	9.0	0.81	6.0	221.9	6.0	57.76
STA 117+00 to 127+00	1,000	7,427	0.0100	0.045	2.00	4.0	221.5	8.1	0.73	6.0	245.5	6.0	63.00
STA 127+00 to 135+50	850	7,427	0.0118	0.045	2.00	4.0	204.1	8.8	0.79	6.0	228.1	6.0	59.13
STA 135+50 to 143+40	790	7,427	0.0127	0.045	2.00	4.0	196.7	9.1	0.81	6.0	220.7	6.0	57.48
STA 143+40 to 152+10	870	7,427	0.0115	0.045	2.00	4.0	206.5	8.7	0.78	6.0	230.5	6.0	59.66
STA 152+10 to 161+90	980	7,427	0.0102	0.045	2.00	4.0	219.3	8.2	0.73	6.0	243.3	6.0	62.50
STA 161+90 to 163+30	140	7,427	0.0143	0.045	2.00	4.0	185.0	9.6	0.87	6.0	209.0	6.0	54.89
STA 163+30 to 168+80	550	3,414	0.0145	0.045	2.00	4.0	83.1	9.4	0.86	6.0	107.1	6.0	32.24
STA 168+80 to 181+80	1,300	3,414	0.0077	0.045	2.00	4.0	115.1	6.9	0.63	6.0	139.1	6.0	39.36
STA 181+80 to 192+60	1,080	2,916	0.0093	0.045	2.00	4.0	89.1	7.5	0.69	6.0	113.1	6.0	33.58
STA 192+60 to 208+70	1,610	2,916	0.0062	0.045	2.00	4.0	109.3	6.2	0.57	6.0	133.3	6.0	38.07
STA 208+70 to 220+50	1,180	2,916	0.0085	0.045	2.00	4.0	93.3	7.2	0.66	6.0	117.3	6.0	34.50
STA 220+50 to 230+00	950	1,458	0.0105	0.045	2.00	4.0	40.3	7.5	0.72	6.0	64.3	6.0	22.74
STA 230+00 to 240+30	1,030	1,458	0.0097	0.045	2.00	4.0	42.1	7.3	0.69	6.0	66.1	6.0	23.14
STA 240+30 to 253+30	1,300	1,458	0.0077	0.045	2.00	4.0	47.7	6.5	0.62	6.0	71.7	6.0	24.38
STA 253+30 to 264+80	1,150	916	0.0087	0.045	2.00	4.0	26.9	6.6	0.64	6.0	50.9	6.0	19.75
STA 264+80 to 274+00	920	916	0.0109	0.045	2.00	4.0	23.6	7.2	0.71	6.0	47.6	6.0	19.03
STA 274+00 to 281+80	780	916	0.0090	0.045	2.00	4.0	26.4	6.7	0.65	6.0	50.4	6.0	19.64
	24300												

TABLE A.2
Hydraulic Data & Cost Estimates for Pinnacle Peak Road Channel (East of Squaw Peak Parkway), No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF

Unit Excavation Cost ($/LF)	Total Channel Excavation Volume For This Reach (cy)	Unit Channel Lining Volume (cy/LF)	Unit Channel Lining Cost ($/LF)	Total Channel Lining Volume For This Reach (cy)	Total Excavation Cost For This Channel Reach	Total Lining Cost For This Channel Reach	Total Channel Construction Cost For This Reach	Channel Right-of-Way Requirements With Two 15 ft Maintenance Roads (acres)	Channel Right-of-Way Cost @ $50,000/Acre
$135.05	66,172	7.11	$170.67	6,969	$132,344	$167,253	$299,598	6.7	$332,800
$160.28	119,409	7.11	$170.67	10,596	$238,817	$254,293	$493,111	12.1	$603,098
$145.58	85,894	7.11	$170.67	8,391	$171,788	$201,387	$373,174	8.7	$432,831
$143.34	80,985	7.11	$170.67	8,036	$161,970	$192,853	$354,823	8.2	$407,935
$148.88	92,305	7.11	$170.67	8,818	$184,609	$211,627	$396,236	9.3	$465,394
$136.43	68,213	7.11	$170.67	7,111	$136,426	$170,667	$307,092	6.9	$343,156
$115.52	46,208	7.11	$170.67	5,689	$92,416	$136,533	$228,949	4.6	$231,332
$126.00	63,001	7.11	$170.67	7,111	$126,001	$170,667	$296,668	6.3	$316,233
$118.25	50,258	7.11	$170.67	6,044	$100,515	$145,067	$245,582	5.0	$251,790
$114.96	45,410	7.11	$170.67	5,618	$90,821	$134,827	$225,648	4.5	$227,304
$119.32	51,906	7.11	$170.67	6,187	$103,812	$148,480	$252,292	5.2	$260,120
$125.00	61,252	7.11	$170.67	6,969	$122,503	$167,253	$289,757	6.1	$307,384
$109.78	7,685	7.11	$170.67	996	$15,370	$23,893	$39,263	0.8	$38,409
$64.49	17,734	7.11	$170.67	3,911	$35,468	$93,867	$129,334	1.7	$86,550
$78.72	51,171	7.11	$170.67	9,244	$102,341	$221,867	$324,208	5.0	$252,374
$67.17	36,270	7.11	$170.67	7,680	$72,540	$184,320	$256,860	3.5	$177,428
$76.14	61,296	7.11	$170.67	11,449	$122,592	$274,773	$397,366	6.0	$301,828
$69.01	40,714	7.11	$170.67	8,391	$81,429	$201,387	$282,815	4.0	$199,465
$45.48	21,604	7.11	$170.67	6,756	$43,208	$162,133	$205,342	2.1	$102,869
$46.28	23,833	7.11	$170.67	7,324	$47,667	$175,787	$223,453	2.3	$113,648
$48.75	31,689	7.11	$170.67	9,244	$63,379	$221,867	$285,245	3.0	$151,747
$39.49	22,708	7.11	$170.67	8,178	$45,417	$196,267	$241,683	2.1	$106,735
$38.06	17,509	7.11	$170.67	6,542	$35,019	$157,013	$192,032	1.6	$81,992
$39.28	15,320	7.11	$170.67	5,547	$30,639	$133,120	$163,759	1.4	$71,968
	1,178,545			172800	$2,357,091	$4,147,200	$6,504,291	117.3	$5,864,388

File: PPW4.WK1

TABLE A.3
Hydraulic Data & Cost Estimates for Pinnacle Peak Road Channel (West of Squaw Peak Parkway), No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Channel Depth With Freeboard	Channel Topwidth With Freeboard	Approximate Toedown Depth (ft)	Unit Excavation Volume (cy/LF)
Start @ 10+00													
STA 10+00 to 32+50	2,250	2,883	0.0044	0.045	2.00	4.0	128.1	5.3	0.48	6.0	152.1	6.0	42.25
STA 32+50 to 53+50	2,100	2,983	0.0095	0.045	2.00	4.0	89.9	7.6	0.70	6.0	113.9	6.0	33.76
STA 53+50 to 68+70	1,520	2,983	0.0099	0.045	2.00	4.0	88.3	7.7	0.71	6.0	112.3	6.0	33.40
STA 68+70 to 71+50	280	2,983	0.0071	0.045	2.00	4.0	104.2	6.6	0.61	6.0	128.2	6.0	36.93
STA 71+50 to 76+40	490	2,983	0.0061	0.045	2.00	4.0	112.7	6.2	0.56	6.0	136.7	6.0	38.82
STA 76+40 to 80+40	400	2,983	0.0013	0.045	2.00	4.0	251.9	2.9	0.26	6.0	275.9	6.0	69.75
STA 80+40 to 105+60	2,520	2,983	0.0077	0.045	2.00	4.0	100.0	6.9	0.63	6.0	124.0	6.0	36.00
STA 105+60 to 124+30	1,870	1,515	0.0107	0.045	2.00	4.0	41.7	7.6	0.72	6.0	65.7	6.0	23.04
STA 124+30 to 126+60	230	1,286	0.0004	0.045	2.00	4.0	183.6	1.7	0.15	6.0	207.6	6.0	54.58
STA 126+60 to 144+10	1,750	1,286	0.0057	0.045	2.00	4.0	49.1	5.6	0.53	6.0	73.1	6.0	24.70
STA 144+10 to 156+40	1,230	900	0.0081	0.045	2.00	4.0	27.3	6.4	0.62	6.0	51.3	6.0	19.86
STA 156+40 to 168+20	1,180	900	0.0085	0.045	2.00	4.0	26.7	6.5	0.63	6.0	50.7	6.0	19.71
STA 168+20 to 172+20	400	900	0.0175	0.045	2.00	4.0	17.4	8.8	0.89	6.0	41.4	6.0	17.65
	16,220												

TABLE A.3
Hydraulic Data & Cost Estimates for Pinnacle Peak Road Channel (West of Squaw Peak Parkway), No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF

Unit Excavation Cost ($/LF)	Total Channel Excavation Volume For This Reach (cy)	Unit Channel Lining Volume (cy/LF)	Unit Channel Lining Cost ($/LF)	Total Channel Lining Volume For This Reach (cy)	Total Excavation Cost For This Channel Reach	Total Lining Cost For This Channel Reach	Total Channel Construction Cost For This Reach	Channel Right-of-Way Requirements With Two 15 ft Maintenance Roads (acres)	Channel Right-of-Way Cost @ $50,000/Acre
$84.51	95,072	7.11	$170.67	16,000	$190,144	$384,000	$574,144	9.4	$470,412
$67.52	70,895	7.11	$170.67	14,933	$141,790	$358,400	$500,190	6.9	$346,910
$66.80	50,765	7.11	$170.67	10,809	$101,531	$259,413	$360,944	5.0	$248,261
$73.86	10,340	7.11	$170.67	1,991	$20,681	$47,787	$68,467	1.0	$50,840
$77.65	19,024	7.11	$170.67	3,484	$38,048	$83,627	$121,675	1.9	$93,765
$139.50	27,900	7.11	$170.67	2,844	$55,799	$68,267	$124,066	2.8	$140,437
$72.00	90,725	7.11	$170.67	17,920	$181,450	$430,080	$611,530	8.9	$445,480
$46.08	43,083	7.11	$170.67	13,298	$86,166	$319,147	$405,312	4.1	$205,364
$109.17	12,554	7.11	$170.67	1,636	$25,108	$39,253	$64,361	1.3	$62,733
$49.39	43,220	7.11	$170.67	12,444	$86,440	$298,667	$385,107	4.1	$207,174
$39.71	24,422	7.11	$170.67	8,747	$48,844	$209,920	$258,764	2.3	$114,851
$39.43	23,263	7.11	$170.67	8,391	$46,526	$201,387	$247,913	2.2	$109,324
$35.31	7,061	7.11	$170.67	2,844	$14,123	$68,267	$82,390	0.7	$32,802
	518,325			115,342	$1,036,650	$2,768,213	$3,804,863	50.6	$2,528,354

File: RAWHIDE4.WK1

TABLE A.4
Hydraulic Data & Cost Estimates for Rawhide Wash Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Channel Depth With Freeboard	Channel Topwidth With Freeboard	Approximate Toedown Depth (ft)	Unit Excavation Volume (cy/LF)
Start @ 5+00													
STA 5+00 to 15+70	1,070	12,585	0.0093	0.045	2.00	4.0	389.6	7.9	0.70	6.0	413.6	6.0	100.35
STA 15+70 to 26+60	1,090	12,585	0.0092	0.045	2.00	4.0	393.2	7.8	0.70	6.0	417.2	6.0	101.16
STA 26+60 to 34+30	770	12,585	0.0130	0.045	2.00	4.0	330.2	9.3	0.83	6.0	354.2	6.0	87.16
STA 34+30 to 43+30	900	12,585	0.0111	0.045	2.00	4.0	357.1	8.6	0.77	6.0	381.1	6.0	93.14
STA 43+30 to 52+70	940	12,585	0.0106	0.045	2.00	4.0	365.0	8.4	0.75	6.0	389.0	6.0	94.90
STA 52+70 to 58+10	540	12,585	0.0185	0.045	2.00	4.0	276.3	11.1	0.99	6.0	300.3	6.0	75.17
STA 58+10 to 67+90	980	12,402	0.0102	0.045	2.00	4.0	367.3	8.3	0.74	6.0	391.3	6.0	95.40
STA 67+90 to 75+00	710	12,402	0.0141	0.045	2.00	4.0	312.4	9.7	0.86	6.0	336.4	6.0	83.20
STA 75+00 to 83+40	840	12,402	0.0119	0.045	2.00	4.0	339.9	8.9	0.79	6.0	363.9	6.0	89.32
STA 83+40 to 88+80	540	12,402	0.0185	0.045	2.00	4.0	272.2	11.1	0.99	6.0	296.2	6.0	74.27
STA 88+80 to 94+80	600	12,402	0.0167	0.045	2.00	4.0	287.0	10.5	0.94	6.0	311.0	6.0	77.56
STA 94+80 to 101+80	700	12,402	0.0143	0.045	2.00	4.0	310.2	9.7	0.87	6.0	334.2	6.0	82.70
STA 101+80 to 107+70	590	12,402	0.0169	0.045	2.00	4.0	284.6	10.6	0.95	6.0	308.6	6.0	77.03
STA 107+70 to 114+60	690	12,402	0.0145	0.045	2.00	4.0	307.9	9.8	0.88	6.0	331.9	6.0	82.21
STA 114+60 to 120+80	620	12,402	0.0161	0.045	2.00	4.0	291.8	10.3	0.92	6.0	315.8	6.0	78.62
STA 120+80 to 127+30	650	12,402	0.0154	0.045	2.00	4.0	298.8	10.1	0.90	6.0	322.8	6.0	80.18
STA 127+30 to 135+20	790	12,402	0.0127	0.045	2.00	4.0	329.6	9.2	0.82	6.0	353.6	6.0	87.03
STA 135+20 to 141+90	670	12,402	0.0149	0.045	2.00	4.0	303.4	10.0	0.89	6.0	327.4	6.0	81.20
STA 141+90 to 148+50	660	12,402	0.0152	0.045	2.00	4.0	301.1	10.0	0.90	6.0	325.1	6.0	80.69
STA 148+50 to 154+90	640	12,402	0.0156	0.045	2.00	4.0	296.5	10.2	0.91	6.0	320.5	6.0	79.67
STA 154+90 to 160+50	560	12,402	0.0179	0.045	2.00	4.0	277.2	10.9	0.97	6.0	301.2	6.0	75.39
STA 160+50 to 166+00	550	12,402	0.0182	0.045	2.00	4.0	274.7	11.0	0.98	6.0	298.7	6.0	74.83
STA 166+00 to 171+50	550	12,661	0.0182	0.045	2.00	4.0	280.5	11.0	0.98	6.0	304.5	6.0	76.11
STA 171+50 to 175+50	400	12,661	0.0250	0.045	2.00	4.0	239.0	12.8	1.15	6.0	263.0	6.0	66.88
STA 175+50 to 181+70	620	12,661	0.0161	0.045	2.00	4.0	297.9	10.3	0.92	6.0	321.9	6.0	79.99
STA 181+70 to 186+80	510	12,661	0.0196	0.045	2.00	4.0	270.1	11.4	1.02	6.0	294.1	6.0	73.79
STA 186+80 to 190+90	410	12,661	0.0244	0.045	2.00	4.0	242.0	12.7	1.13	6.0	266.0	6.0	67.54
STA 190+90 to 195+10	420	12,661	0.0238	0.045	2.00	4.0	244.9	12.5	1.12	6.0	268.9	6.0	68.20
STA 195+10 to 201+00	590	12,661	0.0169	0.045	2.00	4.0	290.6	10.6	0.95	6.0	314.6	6.0	78.36
STA 201+00 to 205+90	490	12,661	0.0204	0.045	2.00	4.0	264.7	11.6	1.04	6.0	288.7	6.0	72.59
STA 205+90 to 209+50	360	10,481	0.0278	0.045	2.00	4.0	187.3	13.4	1.21	6.0	211.3	6.0	55.39
STA 209+50 to 212+70	320	10,481	0.0313	0.045	2.00	4.0	176.4	14.2	1.28	6.0	200.4	6.0	52.99
STA 212+70 to 216+20	350	10,481	0.0286	0.045	2.00	4.0	184.6	13.6	1.22	6.0	208.6	6.0	54.80
STA 216+20 to 220+70	450	10,481	0.0222	0.045	2.00	4.0	209.6	12.0	1.08	6.0	233.6	6.0	60.35
STA 220+70 to 225+70	500	10,481	0.0200	0.045	2.00	4.0	221.0	11.4	1.03	6.0	245.0	6.0	62.89
STA 225+70 to 229+60	390	10,481	0.0256	0.045	2.00	4.0	195.0	12.9	1.16	6.0	219.0	6.0	57.11
STA 229+60 to 233+60	400	10,481	0.0250	0.045	2.00	4.0	197.5	12.8	1.15	6.0	221.5	6.0	57.67
STA 233+60 to 237+80	420	10,481	0.0238	0.045	2.00	4.0	202.4	12.5	1.12	6.0	226.4	6.0	58.76
STA 237+80 to 241+90	410	10,481	0.0244	0.045	2.00	4.0	200.0	12.6	1.13	6.0	224.0	6.0	58.22
STA 241+90 to 246+90	500	10,481	0.0200	0.045	2.00	4.0	221.0	11.4	1.03	6.0	245.0	6.0	62.89
STA 246+90 to 250+90	400	10,481	0.0250	0.045	2.00	4.0	197.5	12.8	1.15	6.0	221.5	6.0	57.67
STA 250+90 to 255+00	410	10,481	0.0244	0.045	2.00	4.0	200.0	12.6	1.13	6.0	224.0	6.0	58.22
STA 255+00 to 258+90	390	10,339	0.0256	0.045	2.00	4.0	192.3	12.9	1.16	6.0	216.3	6.0	56.52
STA 258+90 to 261+70	280	10,339	0.0357	0.045	2.00	4.0	162.7	15.1	1.37	6.0	186.7	6.0	49.92
STA 261+70 to 266+50	480	10,339	0.0208	0.045	2.00	4.0	213.6	11.7	1.05	6.0	237.6	6.0	61.24
STA 266+50 to 271+30	480	10,339	0.0208	0.045	2.00	4.0	213.6	11.7	1.05	6.0	237.6	6.0	61.24
STA 271+30 to 276+40	510	10,339	0.0196	0.045	2.00	4.0	220.2	11.3	1.02	6.0	244.2	6.0	62.71
STA 276+40 to 281+40	500	10,339	0.0200	0.045	2.00	4.0	218.0	11.4	1.03	6.0	242.0	6.0	62.22
STA 281+40 to 284+90	350	10,339	0.0286	0.045	2.00	4.0	182.1	13.6	1.22	6.0	206.1	6.0	54.24
STA 284+90 to 290+10	520	10,339	0.0192	0.045	2.00	4.0	222.4	11.2	1.01	6.0	246.4	6.0	63.19
STA 290+10 to 293+00	290	10,339	0.0345	0.045	2.00	4.0	165.6	14.9	1.34	6.0	189.6	6.0	50.57
STA 293+00 to 298+30	530	10,182	0.0189	0.045	2.00	4.0	221.1	11.1	1.00	6.0	245.1	6.0	62.90
STA 298+30 to 304+30	600	10,182	0.0167	0.045	2.00	4.0	235.3	10.5	0.94	6.0	259.3	6.0	66.07
STA 304+30 to 310+80	650	10,182	0.0154	0.045	2.00	4.0	245.0	10.1	0.90	6.0	269.0	6.0	68.23
STA 310+80 to 315+60	480	10,182	0.0208	0.045	2.00	4.0	210.3	11.7	1.05	6.0	234.3	6.0	60.51
STA 315+60 to 321+20	560	10,182	0.0179	0.045	2.00	4.0	227.3	10.8	0.97	6.0	251.3	6.0	64.29
STA 321+20 to 325+90	470	10,182	0.0213	0.045	2.00	4.0	208.1	11.8	1.06	6.0	232.1	6.0	60.02
STA 325+90 to 330+90	500	10,182	0.0200	0.045	2.00	4.0	214.7	11.4	1.03	6.0	238.7	6.0	61.48
STA 330+90 to 335+60	470	10,182	0.0213	0.045	2.00	4.0	208.1	11.8	1.06	6.0	232.1	6.0	60.02
	33,060												

TABLE A.4
Hydraulic Data & Cost Estimates for Rawhide Wash Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF

Unit Excavation Cost ($/LF)	Total Channel Excavation Volume For This Reach (cy)	Unit Channel Lining Volume (cy/LF)	Unit Channel Lining Cost ($/LF)	Total Channel Lining Volume For This Reach (cy)	Total Excavation Cost For This Channel Reach	Total Lining Cost For This Channel Reach	Total Channel Construction Cost For This Reach	Channel Right-of-Way Requirements With Two 15 ft Maintenance Roads (acres)	Channel Right-of-Way Cost @ $50,000/Acre
$200.70	107,374	7.11	$170.67	7,609	$214,748	$182,613	$397,361	10.9	$544,792
$202.32	110,263	7.11	$170.67	7,751	$220,525	$186,027	$406,552	11.2	$559,529
$174.32	67,113	7.11	$170.67	5,476	$134,226	$131,413	$265,640	6.8	$339,589
$186.29	83,830	7.11	$170.67	6,400	$167,659	$153,600	$321,259	8.5	$424,739
$189.79	89,203	7.11	$170.67	6,684	$178,406	$160,427	$338,833	9.0	$452,127
$150.34	40,591	7.11	$170.67	3,840	$81,182	$92,160	$173,342	4.1	$204,705
$190.81	93,495	7.11	$170.67	6,969	$186,989	$167,253	$354,243	9.5	$473,928
$166.40	59,070	7.11	$170.67	5,049	$118,141	$121,173	$239,314	6.0	$298,596
$178.64	75,029	7.11	$170.67	5,973	$150,057	$143,360	$293,417	7.6	$379,831
$148.54	40,106	7.11	$170.67	3,840	$80,211	$92,160	$172,371	4.0	$202,199
$155.13	46,538	7.11	$170.67	4,267	$93,076	$102,400	$195,476	4.7	$234,872
$165.41	57,893	7.11	$170.67	4,978	$115,786	$119,467	$235,253	5.9	$292,607
$154.05	45,445	7.11	$170.67	4,196	$90,891	$100,693	$191,584	4.6	$229,321
$164.42	56,723	7.11	$170.67	4,907	$113,446	$117,760	$231,206	5.7	$286,656
$157.25	48,747	7.11	$170.67	4,409	$97,494	$105,813	$203,307	4.9	$246,099
$160.37	52,119	7.11	$170.67	4,622	$104,239	$110,933	$215,172	5.3	$263,243
$174.05	68,750	7.11	$170.67	5,618	$137,500	$134,827	$272,327	7.0	$347,860
$162.41	54,406	7.11	$170.67	4,764	$108,812	$114,347	$223,159	5.5	$274,870
$161.39	53,259	7.11	$170.67	4,693	$106,517	$112,640	$219,157	5.4	$269,036
$159.34	50,987	7.11	$170.67	4,551	$101,975	$109,227	$211,202	5.1	$257,488
$150.77	42,217	7.11	$170.67	3,982	$84,434	$95,573	$180,007	4.3	$212,919
$149.66	41,157	7.11	$170.67	3,911	$82,314	$93,867	$176,181	4.2	$207,538
$152.23	41,863	7.11	$170.67	3,911	$83,726	$93,867	$177,592	4.2	$211,183
$133.76	26,752	7.11	$170.67	2,844	$53,504	$68,267	$121,771	2.7	$134,508
$159.97	49,592	7.11	$170.67	4,409	$99,183	$105,813	$204,996	5.0	$250,461
$147.58	37,633	7.11	$170.67	3,627	$75,266	$87,040	$162,306	3.8	$189,702
$135.09	27,693	7.11	$170.67	2,916	$55,387	$69,973	$125,360	2.8	$139,279
$136.40	28,644	7.11	$170.67	2,987	$57,289	$71,680	$128,969	2.9	$144,100
$156.71	46,230	7.11	$170.67	4,196	$92,459	$100,693	$193,152	4.7	$233,371
$145.19	35,571	7.11	$170.67	3,484	$71,142	$83,627	$154,769	3.6	$179,235
$110.78	19,941	7.11	$170.67	2,560	$39,882	$61,440	$101,322	2.0	$99,695
$105.97	16,956	7.11	$170.67	2,276	$33,912	$54,613	$88,525	1.7	$84,643
$109.61	19,181	7.11	$170.67	2,489	$38,362	$59,733	$98,096	1.9	$95,863
$120.71	27,159	7.11	$170.67	3,200	$54,318	$76,800	$131,118	2.7	$136,151
$125.79	31,447	7.11	$170.67	3,556	$62,895	$85,333	$148,228	3.2	$157,844
$114.22	22,272	7.11	$170.67	2,773	$44,544	$66,560	$111,104	2.2	$111,461
$115.33	23,066	7.11	$170.67	2,844	$46,132	$68,267	$114,399	2.3	$115,470
$117.52	24,679	7.11	$170.67	2,987	$49,358	$71,680	$121,038	2.5	$123,618
$116.43	23,869	7.11	$170.67	2,916	$47,737	$69,973	$117,710	2.4	$119,523
$125.79	31,447	7.11	$170.67	3,556	$62,895	$85,333	$148,228	3.2	$157,844
$115.33	23,066	7.11	$170.67	2,844	$46,132	$68,267	$114,399	2.3	$115,470
$116.43	23,869	7.11	$170.67	2,916	$47,737	$69,973	$117,710	2.4	$119,523
$113.03	22,041	7.11	$170.67	2,773	$44,082	$66,560	$110,642	2.2	$110,267
$99.85	13,979	7.11	$170.67	1,991	$27,957	$47,787	$75,744	1.4	$69,633
$122.47	29,394	7.11	$170.67	3,413	$58,787	$81,920	$140,707	2.9	$147,419
$122.47	29,394	7.11	$170.67	3,413	$58,787	$81,920	$140,707	2.9	$147,419
$125.42	31,982	7.11	$170.67	3,627	$63,964	$87,040	$151,004	3.2	$160,514
$124.45	31,112	7.11	$170.67	3,556	$62,224	$85,333	$147,557	3.1	$156,111
$108.48	18,985	7.11	$170.67	2,489	$37,969	$59,733	$97,703	1.9	$94,847
$126.38	32,860	7.11	$170.67	3,698	$65,719	$88,747	$154,466	3.3	$164,954
$101.14	14,666	7.11	$170.67	2,062	$29,332	$49,493	$78,825	1.5	$73,090
$125.81	33,339	7.11	$170.67	3,769	$66,678	$90,453	$157,132	3.3	$167,340
$132.15	39,644	7.11	$170.67	4,267	$79,289	$102,400	$181,689	4.0	$199,265
$136.45	44,347	7.11	$170.67	4,622	$88,694	$110,933	$199,627	4.5	$223,096
$121.02	29,045	7.11	$170.67	3,413	$58,089	$81,920	$140,009	2.9	$145,616
$128.57	36,001	7.11	$170.67	3,982	$72,001	$95,573	$167,575	3.6	$180,811
$120.03	28,207	7.11	$170.67	3,342	$56,415	$80,213	$136,628	2.8	$141,384
$122.96	30,741	7.11	$170.67	3,556	$61,482	$85,333	$146,815	3.1	$154,194
$120.03	28,207	7.11	$170.67	3,342	$56,415	$80,213	$136,628	2.8	$141,384
	2,459,186			235,093	$4,918,373	$5,642,240	$10,560,613	248.0	$12,398,828

File: SCT4.WK1

TABLE A.5
Hydraulic Data & Cost Estimates for Scottsdale Road Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Channel Depth With Freeboard	Channel Topwidth With Freeboard	Approximate Toedown Depth (ft)	Unit Excavation Volume (cy/LF)
Start @ 203+20													
STA 203+20 to 204+60	140	2,002	0.0393	0.045	2.00	4.0	27.7	14.0	1.37	6.0	51.7	6.0	19.94
STA 204+60 to 209+80	520	2,002	0.0192	0.045	2.00	4.0	41.0	10.2	0.97	6.0	65.0	6.0	22.90
STA 209+80 to 215+60	580	2,002	0.0172	0.045	2.00	4.0	43.5	9.7	0.92	6.0	67.5	6.0	23.44
STA 215+60 to 219+80	420	2,002	0.0238	0.045	2.00	4.0	36.6	11.2	1.08	6.0	60.6	6.0	21.90
STA 219+80 to 223+20	340	2,002	0.0294	0.045	2.00	4.0	32.6	12.3	1.19	6.0	56.6	6.0	21.01
STA 223+20 to 232+70	950	2,024	0.0105	0.045	2.00	4.0	57.1	7.8	0.73	6.0	81.1	6.0	26.47
STA 232+70 to 238+80	610	2,024	0.0164	0.045	2.00	4.0	45.2	9.5	0.90	6.0	69.2	6.0	23.82
STA 238+80 to 245+60	680	2,024	0.0147	0.045	2.00	4.0	47.9	9.1	0.85	6.0	71.9	6.0	24.42
STA 245+60 to 251+90	630	2,024	0.0159	0.045	2.00	4.0	46.0	9.4	0.88	6.0	70.0	6.0	24.00
STA 251+90 to 257+40	550	2,024	0.0182	0.045	2.00	4.0	42.8	10.0	0.94	6.0	66.8	6.0	23.28
STA 257+40 to 262+90	550	2,024	0.0182	0.045	2.00	4.0	42.8	10.0	0.94	6.0	66.8	6.0	23.28
STA 262+90 to 268+50	560	1,681	0.0179	0.045	2.00	4.0	35.3	9.7	0.93	6.0	59.3	6.0	21.63
STA 268+50 to 274+40	590	1,681	0.0169	0.045	2.00	4.0	36.4	9.5	0.91	6.0	60.4	6.0	21.86
STA 274+40 to 279+40	500	1,681	0.0200	0.045	2.00	4.0	33.2	10.2	0.98	6.0	57.2	6.0	21.16
STA 279+40 to 284+20	480	1,681	0.0208	0.045	2.00	4.0	32.5	10.4	1.00	6.0	56.5	6.0	21.00
STA 284+20 to 291+00	680	1,681	0.0147	0.045	2.00	4.0	39.3	8.9	0.85	6.0	63.3	6.0	22.50
STA 291+00 to 297+20	620	1,681	0.0161	0.045	2.00	4.0	37.4	9.3	0.89	6.0	61.4	6.0	22.08
STA 297+20 to 303+30	610	991	0.0164	0.045	2.00	4.0	20.4	8.7	0.87	6.0	44.4	6.0	18.31
STA 303+30 to 309+10	580	991	0.0172	0.045	2.00	4.0	19.8	8.9	0.89	6.0	43.8	6.0	18.17
STA 309+10 to 316+80	770	718	0.0130	0.045	2.00	4.0	15.9	7.5	0.77	6.0	39.9	6.0	17.30
STA 316+80 to 322+50	570	718	0.0175	0.045	2.00	4.0	13.1	8.5	0.88	6.0	37.1	6.0	16.68
STA 322+50 to 327+40	490	718	0.0204	0.045	2.00	4.0	11.8	9.1	0.95	6.0	35.8	6.0	16.40
STA 327+40 to 335+30	790	718	0.0127	0.045	2.00	4.0	16.1	7.4	0.76	6.0	40.1	6.0	17.36
	13,210												

TABLE A.5
Hydraulic Data & Cost Estimates for Scottsdale Road Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF

Unit Excavation Cost ($/LF)	Total Channel Excavation Volume For This Reach (cy)	Unit Channel Lining Volume (cy/LF)	Unit Channel Lining Cost ($/LF)	Total Channel Lining Volume For This Reach (cy)	Total Excavation Cost For This Channel Reach	Total Lining Cost For This Channel Reach	Total Channel Construction Cost For This Reach	Channel Right-of-Way Requirements With Two 15 ft Maintenance Roads (acres)	Channel Right-of-Way Cost @ $50,000/Acre
$39.87	2,791	7.11	$170.67	996	$5,582	$23,893	$29,476	0.3	$13,132
$45.79	11,905	7.11	$170.67	3,698	$23,811	$88,747	$112,558	1.1	$56,720
$46.89	13,598	7.11	$170.67	4,124	$27,195	$98,987	$126,182	1.3	$64,910
$43.80	9,198	7.11	$170.67	2,987	$18,397	$71,680	$90,077	0.9	$43,655
$42.03	7,145	7.11	$170.67	2,418	$14,289	$58,027	$72,316	0.7	$33,783
$52.94	25,149	7.11	$170.67	6,756	$50,298	$162,133	$212,431	2.4	$121,178
$47.65	14,533	7.11	$170.67	4,338	$29,066	$104,107	$133,173	1.4	$69,466
$48.84	16,607	7.11	$170.67	4,836	$33,213	$116,053	$149,267	1.6	$79,534
$48.00	15,119	7.11	$170.67	4,480	$30,238	$107,520	$137,758	1.4	$72,309
$46.57	12,806	7.11	$170.67	3,911	$25,613	$93,867	$119,479	1.2	$61,098
$46.57	12,806	7.11	$170.67	3,911	$25,613	$93,867	$119,479	1.2	$61,098
$43.26	12,114	7.11	$170.67	3,982	$24,228	$95,573	$119,801	1.1	$57,429
$43.72	12,896	7.11	$170.67	4,196	$25,793	$100,693	$126,486	1.2	$61,196
$42.32	10,580	7.11	$170.67	3,556	$21,159	$85,333	$106,493	1.0	$50,056
$41.99	10,078	7.11	$170.67	3,413	$20,155	$81,920	$102,075	1.0	$47,647
$45.01	15,303	7.11	$170.67	4,836	$30,606	$116,053	$146,659	1.5	$72,800
$44.16	13,689	7.11	$170.67	4,409	$27,378	$105,813	$133,191	1.3	$65,014
$36.61	11,167	7.11	$170.67	4,338	$22,335	$104,107	$126,441	1.0	$52,081
$36.35	10,540	7.11	$170.67	4,124	$21,081	$98,987	$120,067	1.0	$49,118
$34.60	13,322	7.11	$170.67	5,476	$26,643	$131,413	$158,056	1.2	$61,739
$33.36	9,507	7.11	$170.67	4,053	$19,013	$97,280	$116,293	0.9	$43,871
$32.79	8,034	7.11	$170.67	3,484	$16,069	$83,627	$99,695	0.7	$37,000
$34.72	13,713	7.11	$170.67	5,618	$27,425	$134,827	$162,252	1.3	$63,576
	282,600			93,938	$565,201	$2,254,507	$2,819,707	26.8	$1,338,409

File: PIMA4.WK1

TABLE A.6
Hydraulic Data & Cost Estimates for Pima Road Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Channel Depth With Freeboard	Channel Topwidth With Freeboard	Approximate Toedown Depth (ft)	Unit Excavation Volume (cy/LF)
Start @ 17+00													
STA 17+00 to 20+00	300	5,972	0.0167	0.045	2.00	4.0	137.2	10.3	0.93	6.0	161.2	6.0	44.27
STA 20+00 to 25+90	590	5,972	0.0169	0.045	2.00	4.0	136.1	10.4	0.94	6.0	160.1	6.0	44.01
STA 25+90 to 30+00	410	5,972	0.0244	0.045	2.00	4.0	113.1	12.3	1.12	6.0	137.1	6.0	38.90
STA 30+00 to 35+30	530	5,972	0.0189	0.045	2.00	4.0	128.8	10.9	0.99	6.0	152.8	6.0	42.41
STA 35+30 to 39+90	460	5,972	0.0217	0.045	2.00	4.0	119.9	11.7	1.06	6.0	143.9	6.0	40.42
STA 39+90 to 44+70	480	5,972	0.0208	0.045	2.00	4.0	122.5	11.4	1.04	6.0	146.5	6.0	41.00
STA 44+70 to 50+20	550	5,972	0.0182	0.045	2.00	4.0	131.3	10.7	0.97	6.0	155.3	6.0	42.95
STA 50+20 to 54+10	390	5,972	0.0256	0.045	2.00	4.0	110.2	12.6	1.15	6.0	134.2	6.0	38.27
STA 54+10 to 59+70	560	5,972	0.0179	0.045	2.00	4.0	132.5	10.6	0.96	6.0	156.5	6.0	43.22
STA 59+70 to 64+30	460	5,972	0.0217	0.045	2.00	4.0	119.9	11.7	1.06	6.0	143.9	6.0	40.42
STA 64+30 to 68+50	420	5,972	0.0238	0.045	2.00	4.0	114.5	12.2	1.11	6.0	138.5	6.0	39.21
STA 68+50 to 73+20	470	5,972	0.0213	0.045	2.00	4.0	121.2	11.6	1.05	6.0	145.2	6.0	40.71
STA 73+20 to 78+00	480	5,972	0.0208	0.045	2.00	4.0	122.5	11.4	1.04	6.0	146.5	6.0	41.00
STA 78+00 to 81+90	390	6,104	0.0256	0.045	2.00	4.0	112.7	12.6	1.15	6.0	136.7	6.0	38.82
STA 81+90 to 86+70	480	6,104	0.0208	0.045	2.00	4.0	125.3	11.5	1.04	6.0	149.3	6.0	41.61
STA 86+70 to 91+60	490	6,104	0.0204	0.045	2.00	4.0	126.6	11.3	1.03	6.0	150.6	6.0	41.91
STA 91+60 to 96+30	470	6,104	0.0213	0.045	2.00	4.0	123.9	11.6	1.05	6.0	147.9	6.0	41.32
STA 96+30 to 100+10	380	6,104	0.0263	0.045	2.00	4.0	111.2	12.8	1.17	6.0	135.2	6.0	38.49
STA 100+10 to 105+10	500	6,104	0.0200	0.045	2.00	4.0	127.9	11.2	1.02	6.0	151.9	6.0	42.20
STA 105+10 to 109+10	400	6,104	0.0250	0.045	2.00	4.0	114.2	12.5	1.14	6.0	138.2	6.0	39.15
STA 109+10 to 113+90	480	6,104	0.0208	0.045	2.00	4.0	125.3	11.5	1.04	6.0	149.3	6.0	41.61
STA 113+90 to 118+00	410	6,104	0.0244	0.045	2.00	4.0	115.6	12.3	1.12	6.0	139.6	6.0	39.47
STA 118+00 to 122+20	420	6,104	0.0238	0.045	2.00	4.0	117.0	12.2	1.11	6.0	141.0	6.0	39.79
STA 122+20 to 126+90	470	6,104	0.0213	0.045	2.00	4.0	123.9	11.6	1.05	6.0	147.9	6.0	41.32
STA 126+90 to 130+30	340	5,050	0.0294	0.045	2.00	4.0	86.5	13.4	1.23	6.0	110.5	6.0	33.01
STA 130+30 to 133+20	290	5,050	0.0345	0.045	2.00	4.0	79.7	14.4	1.32	6.0	103.7	6.0	31.50
STA 133+20 to 137+40	420	5,050	0.0238	0.045	2.00	4.0	96.4	12.1	1.11	6.0	120.4	6.0	35.21
STA 137+40 to 141+40	400	5,050	0.0250	0.045	2.00	4.0	94.1	12.4	1.13	6.0	118.1	6.0	34.68
STA 141+40 to 146+30	490	5,050	0.0204	0.045	2.00	4.0	104.4	11.2	1.02	6.0	128.4	6.0	36.97
STA 146+30 to 150+60	430	5,050	0.0233	0.045	2.00	4.0	97.6	12.0	1.09	6.0	121.6	6.0	35.47
STA 150+60 to 154+60	400	5,050	0.0250	0.045	2.00	4.0	94.1	12.4	1.13	6.0	118.1	6.0	34.68
STA 154+60 to 158+60	400	5,050	0.0250	0.045	2.00	4.0	94.1	12.4	1.13	6.0	118.1	6.0	34.68
STA 158+60 to 163+70	510	5,050	0.0196	0.045	2.00	4.0	106.5	11.0	1.00	6.0	130.5	6.0	37.45
STA 163+70 to 168+00	430	5,156	0.0233	0.045	2.00	4.0	99.7	12.0	1.09	6.0	123.7	6.0	35.93
STA 168+00 to 173+90	590	5,156	0.0169	0.045	2.00	4.0	117.2	10.3	0.94	6.0	141.2	6.0	39.82
STA 173+90 to 179+60	570	5,156	0.0175	0.045	2.00	4.0	115.1	10.5	0.95	6.0	139.1	6.0	39.36
STA 179+60 to 184+50	490	4,165	0.0204	0.045	2.00	4.0	85.7	11.1	1.02	6.0	109.7	6.0	32.81
STA 184+50 to 190+40	590	4,165	0.0169	0.045	2.00	4.0	94.2	10.2	0.93	6.0	118.2	6.0	34.72
STA 190+40 to 197+30	690	4,165	0.0145	0.045	2.00	4.0	102.1	9.5	0.86	6.0	126.1	6.0	36.46
STA 197+30 to 202+00	470	4,165	0.0213	0.045	2.00	4.0	83.8	11.3	1.04	6.0	107.8	6.0	32.41
STA 202+00 to 207+60	560	4,165	0.0179	0.045	2.00	4.0	91.7	10.4	0.96	6.0	115.7	6.0	34.16
STA 207+60 to 212+80	520	4,165	0.0192	0.045	2.00	4.0	88.3	10.8	0.99	6.0	112.3	6.0	33.40
STA 212+80 to 218+90	610	4,165	0.0164	0.045	2.00	4.0	95.8	10.0	0.92	6.0	119.8	6.0	35.08
STA 218+90 to 224+70	580	3,028	0.0172	0.045	2.00	4.0	67.2	10.1	0.93	6.0	91.2	6.0	28.72
STA 224+70 to 231+90	720	3,028	0.0139	0.045	2.00	4.0	75.2	9.1	0.84	6.0	99.2	6.0	30.49
STA 231+90 to 237+50	560	3,028	0.0179	0.045	2.00	4.0	66.0	10.2	0.95	6.0	90.0	6.0	28.45
STA 237+50 to 243+70	620	2,222	0.0161	0.045	2.00	4.0	50.3	9.5	0.89	6.0	74.3	6.0	24.97
STA 243+70 to 250+40	670	2,222	0.0149	0.045	2.00	4.0	52.5	9.2	0.86	6.0	76.5	6.0	25.43
STA 250+40 to 254+40	400	1,914	0.0250	0.045	2.00	4.0	33.9	11.4	1.10	6.0	57.9	6.0	21.31
STA 254+40 to 259+40	500	1,914	0.0200	0.045	2.00	4.0	38.3	10.3	0.99	6.0	62.3	6.0	22.28
STA 259+40 to 263+30	390	1,914	0.0256	0.045	2.00	4.0	33.4	11.6	1.11	6.0	57.4	6.0	21.20
STA 263+30 to 268+30	500	1,914	0.0200	0.045	2.00	4.0	38.3	10.3	0.99	6.0	62.3	6.0	22.28
STA 268+30 to 271+40	310	1,914	0.0323	0.045	2.00	4.0	29.4	12.8	1.24	6.0	53.4	6.0	20.32
STA 271+40 to 275+70	430	1,914	0.0233	0.045	2.00	4.0	35.3	11.1	1.06	6.0	59.3	6.0	21.61
STA 275+70 to 279+40	370	632	0.0270	0.045	2.00	4.0	7.9	9.9	1.07	6.0	31.9	6.0	15.54
STA 279+40 to 282+60	320	632	0.0313	0.045	2.00	4.0	7.0	10.5	1.15	6.0	31.0	6.0	15.34
STA 282+60 to 286+30	370	632	0.0270	0.045	2.00	4.0	7.9	9.9	1.07	6.0	31.9	6.0	15.54
STA 286+30 to 292+20	590	632	0.0169	0.045	2.00	4.0	11.2	8.2	0.86	6.0	35.2	6.0	16.27
STA 292+20 to 297+20	500	632	0.0200	0.045	2.00	4.0	10.0	8.8	0.93	6.0	34.0	6.0	16.00
	28,020												

TABLE A.6
Hydraulic Data & Cost Estimates for Pima Road Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Unit Excavation Cost ($/LF)	Total Channel Excavation Volume For This Reach (cy)	Unit Channel Lining Volume (cy/LF)	Unit Channel Lining Cost ($/LF)	Total Channel Lining Volume For This Reach (cy)	Total Excavation Cost For This Channel Reach	Total Lining Cost For This Channel Reach	Total Channel Construction Cost For This Reach	Channel Right-of-Way Requirements With Two 15 ft Maintenance Roads (acres)	Channel Right-of-Way Cost @ $50,000/Acre
$88.54	13,281	7.11	$170.67	2,133	$26,563	$51,200	$77,763	1.3	$65,847
$88.02	25,967	7.11	$170.67	4,196	$51,934	$100,693	$152,628	2.6	$128,710
$77.81	15,950	7.11	$170.67	2,916	$31,900	$69,973	$101,873	1.6	$78,622
$84.82	22,477	7.11	$170.67	3,769	$44,954	$90,453	$135,407	2.2	$111,233
$80.84	18,593	7.11	$170.67	3,271	$37,186	$78,507	$115,692	1.8	$91,813
$82.00	19,681	7.11	$170.67	3,413	$39,362	$81,920	$121,282	1.9	$97,251
$85.91	23,624	7.11	$170.67	3,911	$47,249	$93,867	$141,115	2.3	$116,976
$76.54	14,925	7.11	$170.67	2,773	$29,851	$66,560	$96,411	1.5	$73,512
$86.44	24,204	7.11	$170.67	3,982	$48,408	$95,573	$143,981	2.4	$119,879
$80.84	18,593	7.11	$170.67	3,271	$37,186	$78,507	$115,692	1.8	$91,813
$78.43	16,469	7.11	$170.67	2,987	$32,939	$71,680	$104,619	1.6	$81,213
$81.42	19,135	7.11	$170.67	3,342	$38,270	$80,213	$118,483	1.9	$94,521
$82.00	19,681	7.11	$170.67	3,413	$39,362	$81,920	$121,282	1.9	$97,251
$77.64	15,141	7.11	$170.67	2,773	$30,281	$66,560	$96,841	1.5	$74,624
$83.23	19,975	7.11	$170.67	3,413	$39,950	$81,920	$121,870	2.0	$98,769
$83.82	20,535	7.11	$170.67	3,484	$41,070	$83,627	$124,697	2.0	$101,569
$82.64	19,420	7.11	$170.67	3,342	$38,839	$80,213	$119,052	1.9	$95,992
$76.99	14,627	7.11	$170.67	2,702	$29,254	$64,853	$94,108	1.4	$72,064
$84.40	21,099	7.11	$170.67	3,556	$42,198	$85,333	$127,532	2.1	$104,392
$78.29	15,659	7.11	$170.67	2,844	$31,318	$68,267	$99,585	1.5	$77,210
$83.23	19,975	7.11	$170.67	3,413	$39,950	$81,920	$121,870	2.0	$98,769
$78.94	16,182	7.11	$170.67	2,916	$32,364	$69,973	$102,338	1.6	$79,820
$79.57	16,710	7.11	$170.67	2,987	$33,420	$71,680	$105,100	1.6	$82,456
$82.64	19,420	7.11	$170.67	3,342	$38,839	$80,213	$119,052	1.9	$95,992
$66.02	11,223	7.11	$170.67	2,418	$22,445	$58,027	$80,472	1.1	$54,846
$62.99	9,134	7.11	$170.67	2,062	$18,268	$49,493	$67,761	0.9	$44,517
$70.42	14,788	7.11	$170.67	2,987	$29,576	$71,680	$101,256	1.5	$72,526
$69.36	13,872	7.11	$170.67	2,844	$27,744	$68,267	$96,011	1.4	$67,980
$73.93	18,114	7.11	$170.67	3,484	$36,227	$83,627	$119,854	1.8	$89,063
$70.94	15,252	7.11	$170.67	3,058	$30,503	$73,387	$103,890	1.5	$74,830
$69.36	13,872	7.11	$170.67	2,844	$27,744	$68,267	$96,011	1.4	$67,980
$69.36	13,872	7.11	$170.67	2,844	$27,744	$68,267	$96,011	1.4	$67,980
$74.89	19,097	7.11	$170.67	3,627	$38,194	$87,040	$125,234	1.9	$93,959
$71.87	15,452	7.11	$170.67	3,058	$30,904	$73,387	$104,291	1.5	$75,866
$79.63	23,492	7.11	$170.67	4,196	$46,984	$100,693	$147,677	2.3	$115,924
$78.73	22,437	7.11	$170.67	4,053	$44,874	$97,280	$142,154	2.2	$110,659
$65.62	16,078	7.11	$170.67	3,484	$32,156	$83,627	$115,783	1.6	$78,548
$69.43	20,482	7.11	$170.67	4,196	$40,964	$100,693	$141,658	2.0	$100,379
$72.92	25,159	7.11	$170.67	4,907	$50,318	$117,760	$168,078	2.5	$123,617
$64.82	15,232	7.11	$170.67	3,342	$30,464	$80,213	$110,678	1.5	$74,363
$68.33	19,131	7.11	$170.67	3,982	$38,263	$95,573	$133,836	1.9	$93,676
$66.80	17,369	7.11	$170.67	3,698	$34,738	$88,747	$123,485	1.7	$84,942
$70.15	21,396	7.11	$170.67	4,338	$42,793	$104,107	$146,900	2.1	$104,917
$57.44	16,656	7.11	$170.67	4,124	$33,312	$98,987	$132,299	1.6	$80,708
$60.98	21,951	7.11	$170.67	5,120	$43,903	$122,880	$166,783	2.1	$106,773
$56.90	15,931	7.11	$170.67	3,982	$31,861	$95,573	$127,435	1.5	$77,144
$49.93	15,479	7.11	$170.67	4,409	$30,958	$105,813	$136,771	1.5	$74,261
$50.87	17,041	7.11	$170.67	4,764	$34,081	$114,347	$148,428	1.6	$81,867
$42.62	8,523	7.11	$170.67	2,844	$17,047	$68,267	$85,313	0.8	$40,353
$44.56	11,141	7.11	$170.67	3,556	$22,281	$85,333	$107,615	1.1	$52,954
$42.41	8,270	7.11	$170.67	2,773	$16,540	$66,560	$83,100	0.8	$39,135
$44.56	11,141	7.11	$170.67	3,556	$22,281	$85,333	$107,615	1.1	$52,954
$40.64	6,299	7.11	$170.67	2,204	$12,597	$52,907	$65,504	0.6	$29,688
$43.22	9,293	7.11	$170.67	3,058	$18,587	$73,387	$91,973	0.9	$44,054
$31.08	5,750	7.11	$170.67	2,631	$11,500	$63,147	$74,647	0.5	$26,304
$30.69	4,910	7.11	$170.67	2,276	$9,819	$54,613	$64,433	0.4	$22,422
$31.08	5,750	7.11	$170.67	2,631	$11,500	$63,147	$74,647	0.5	$26,304
$32.55	9,602	7.11	$170.67	4,196	$19,203	$100,693	$119,896	0.9	$44,177
$31.99	7,998	7.11	$170.67	3,556	$15,996	$85,333	$101,329	0.7	$36,720
	952,510			199,253	$1,905,019	$4,782,080	$6,687,099	93.3	$4,662,687

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TABLE A.7
Hydraulic Data & Cost Estimates for Pima Road Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 3
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Channel Depth With Freeboard	Channel Topwidth With Freeboard	Approximate Toedown Depth (ft)	Unit Excavation Volume (cy/LF)
Start @ 17+00													
STA 17+00 to 20+00	300	20,717	0.0167	0.045	2.00	4.0	480.6	10.6	0.94	6.0	504.6	6.0	120.58
STA 20+00 to 25+90	590	20,717	0.0169	0.045	2.00	4.0	476.6	10.7	0.95	6.0	500.6	6.0	119.68
STA 25+90 to 30+00	410	20,717	0.0244	0.045	2.00	4.0	397.0	12.8	1.14	6.0	421.0	6.0	102.00
STA 30+00 to 35+30	530	20,717	0.0189	0.045	2.00	4.0	451.6	11.3	1.00	6.0	475.6	6.0	114.14
STA 35+30 to 39+90	460	20,717	0.0217	0.045	2.00	4.0	420.6	12.1	1.07	6.0	444.6	6.0	107.25
STA 39+90 to 44+70	480	20,717	0.0208	0.045	2.00	4.0	429.7	11.8	1.05	6.0	453.7	6.0	109.27
STA 44+70 to 50+20	550	20,717	0.0182	0.045	2.00	4.0	460.1	11.1	0.98	6.0	484.1	6.0	116.02
STA 50+20 to 54+10	390	20,717	0.0256	0.045	2.00	4.0	387.2	13.1	1.17	6.0	411.2	6.0	99.81
STA 54+10 to 59+70	560	20,717	0.0179	0.045	2.00	4.0	464.3	11.0	0.97	6.0	488.3	6.0	116.95
STA 59+70 to 64+30	460	20,717	0.0217	0.045	2.00	4.0	420.6	12.1	1.07	6.0	444.6	6.0	107.25
STA 64+30 to 68+50	420	20,717	0.0238	0.045	2.00	4.0	401.8	12.6	1.12	6.0	425.8	6.0	103.08
STA 68+50 to 73+20	470	20,717	0.0213	0.045	2.00	4.0	425.2	12.0	1.06	6.0	449.2	6.0	108.26
STA 73+20 to 78+00	480	20,717	0.0208	0.045	2.00	4.0	429.7	11.8	1.05	6.0	453.7	6.0	109.27
STA 78+00 to 81+90	390	20,724	0.0256	0.045	2.00	4.0	387.3	13.1	1.17	6.0	411.3	6.0	99.84
STA 81+90 to 86+70	480	20,724	0.0208	0.045	2.00	4.0	429.8	11.8	1.05	6.0	453.8	6.0	109.30
STA 86+70 to 91+60	490	20,724	0.0204	0.045	2.00	4.0	434.3	11.7	1.04	6.0	458.3	6.0	110.29
STA 91+60 to 96+30	470	20,724	0.0213	0.045	2.00	4.0	425.3	12.0	1.06	6.0	449.3	6.0	108.29
STA 96+30 to 100+10	380	20,724	0.0263	0.045	2.00	4.0	382.3	13.3	1.18	6.0	406.3	6.0	98.73
STA 100+10 to 105+10	500	20,724	0.0200	0.045	2.00	4.0	438.7	11.6	1.03	6.0	462.7	6.0	111.28
STA 105+10 to 109+10	400	20,724	0.0250	0.045	2.00	4.0	392.2	12.9	1.15	6.0	416.2	6.0	100.94
STA 109+10 to 113+90	480	20,724	0.0208	0.045	2.00	4.0	429.8	11.8	1.05	6.0	453.8	6.0	109.30
STA 113+90 to 118+00	410	20,724	0.0244	0.045	2.00	4.0	397.1	12.8	1.14	6.0	421.1	6.0	102.03
STA 118+00 to 122+20	420	20,724	0.0238	0.045	2.00	4.0	402.0	12.6	1.12	6.0	426.0	6.0	103.11
STA 122+20 to 126+90	470	20,724	0.0213	0.045	2.00	4.0	425.3	12.0	1.06	6.0	449.3	6.0	108.29
STA 126+90 to 130+30	340	5,050	0.0294	0.045	2.00	4.0	86.5	13.4	1.23	6.0	110.5	6.0	33.01
STA 130+30 to 133+20	290	5,050	0.0345	0.045	2.00	4.0	79.7	14.4	1.32	6.0	103.7	6.0	31.50
STA 133+20 to 137+40	420	5,050	0.0238	0.045	2.00	4.0	96.4	12.1	1.11	6.0	120.4	6.0	35.21
STA 137+40 to 141+40	400	5,050	0.0250	0.045	2.00	4.0	94.1	12.4	1.13	6.0	118.1	6.0	34.68
STA 141+40 to 146+30	490	5,050	0.0204	0.045	2.00	4.0	104.4	11.2	1.02	6.0	128.4	6.0	36.97
STA 146+30 to 150+60	430	5,050	0.0233	0.045	2.00	4.0	97.6	12.0	1.09	6.0	121.6	6.0	35.47
STA 150+60 to 154+60	400	5,050	0.0250	0.045	2.00	4.0	94.1	12.4	1.13	6.0	118.1	6.0	34.68
STA 154+60 to 158+60	400	5,050	0.0250	0.045	2.00	4.0	94.1	12.4	1.13	6.0	118.1	6.0	34.68
STA 158+60 to 163+70	510	5,050	0.0196	0.045	2.00	4.0	106.5	11.0	1.00	6.0	130.5	6.0	37.45
STA 163+70 to 168+00	430	5,156	0.0233	0.045	2.00	4.0	99.7	12.0	1.09	6.0	123.7	6.0	35.93
STA 168+00 to 173+90	590	5,156	0.0169	0.045	2.00	4.0	117.2	10.3	0.94	6.0	141.2	6.0	39.82
STA 173+90 to 179+60	570	5,156	0.0175	0.045	2.00	4.0	115.1	10.5	0.95	6.0	139.1	6.0	39.36
STA 179+60 to 184+50	490	4,165	0.0204	0.045	2.00	4.0	85.7	11.1	1.02	6.0	109.7	6.0	32.81
STA 184+50 to 190+40	590	4,165	0.0169	0.045	2.00	4.0	94.2	10.2	0.93	6.0	118.2	6.0	34.72
STA 190+40 to 197+30	690	4,165	0.0145	0.045	2.00	4.0	102.1	9.5	0.86	6.0	126.1	6.0	36.46
STA 197+30 to 202+00	470	4,165	0.0213	0.045	2.00	4.0	83.8	11.3	1.04	6.0	107.8	6.0	32.41
STA 202+00 to 207+60	560	4,165	0.0179	0.045	2.00	4.0	91.7	10.4	0.96	6.0	115.7	6.0	34.16
STA 207+60 to 212+80	520	4,165	0.0192	0.045	2.00	4.0	88.3	10.8	0.99	6.0	112.3	6.0	33.40
STA 212+80 to 218+90	610	4,165	0.0164	0.045	2.00	4.0	95.8	10.0	0.92	6.0	119.8	6.0	35.08
STA 218+90 to 224+70	580	3,028	0.0172	0.045	2.00	4.0	67.2	10.1	0.93	6.0	91.2	6.0	28.72
STA 224+70 to 231+90	720	3,028	0.0139	0.045	2.00	4.0	75.2	9.1	0.84	6.0	99.2	6.0	30.49
STA 231+90 to 237+50	560	3,028	0.0179	0.045	2.00	4.0	66.0	10.2	0.95	6.0	90.0	6.0	28.45
STA 237+50 to 243+70	620	2,222	0.0161	0.045	2.00	4.0	50.3	9.5	0.89	6.0	74.3	6.0	24.97
STA 243+70 to 250+40	670	2,222	0.0149	0.045	2.00	4.0	52.5	9.2	0.86	6.0	76.5	6.0	25.43
STA 250+40 to 254+40	400	1,914	0.0250	0.045	2.00	4.0	33.9	11.4	1.10	6.0	57.9	6.0	21.31
STA 254+40 to 259+40	500	1,914	0.0200	0.045	2.00	4.0	38.3	10.3	0.99	6.0	62.3	6.0	22.28
STA 259+40 to 263+30	390	1,914	0.0256	0.045	2.00	4.0	33.4	11.6	1.11	6.0	57.4	6.0	21.20
STA 263+30 to 268+30	500	1,914	0.0200	0.045	2.00	4.0	38.3	10.3	0.99	6.0	62.3	6.0	22.28
STA 268+30 to 271+40	310	1,914	0.0323	0.045	2.00	4.0	29.4	12.8	1.24	6.0	53.4	6.0	20.32
STA 271+40 to 275+70	430	1,914	0.0233	0.045	2.00	4.0	35.3	11.1	1.06	6.0	59.3	6.0	21.61
STA 275+70 to 279+40	370	632	0.0270	0.045	2.00	4.0	7.9	9.9	1.07	6.0	31.9	6.0	15.54
STA 279+40 to 282+60	320	632	0.0313	0.045	2.00	4.0	7.0	10.5	1.15	6.0	31.0	6.0	15.34
STA 282+60 to 286+30	370	632	0.0270	0.045	2.00	4.0	7.9	9.9	1.07	6.0	31.9	6.0	15.54
STA 286+30 to 292+20	590	632	0.0169	0.045	2.00	4.0	11.2	8.2	0.86	6.0	35.2	6.0	16.27
STA 292+20 to 297+20	500	632	0.0200	0.045	2.00	4.0	10.0	8.8	0.93	6.0	34.0	6.0	16.00
	28,020												

TABLE A.7
Hydraulic Data & Cost Estimates for Pima Road Channel, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 3
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Unit Excavation Cost ($/LF)	Total Channel Excavation Volume For This Reach (cy)	Unit Channel Lining Volume (cy/LF)	Unit Channel Lining Cost ($/LF)	Total Channel Lining Volume For This Reach (cy)	Total Excavation Cost For This Channel Reach	Total Lining Cost For This Channel Reach	Total Channel Construction Cost For This Reach	Channel Right-of-Way Requirements With Two 15 ft Maintenance Roads (acres)	Channel Right-of-Way Cost @ $50,000/Acre
$241.16	36,174	7.11	$170.67	2,133	$72,348	$51,200	$123,548	3.7	$184,095
$239.37	70,613	7.11	$170.67	4,196	$141,226	$100,693	$241,920	7.2	$359,320
$204.00	41,821	7.11	$170.67	2,916	$83,641	$69,973	$153,615	4.2	$212,251
$228.27	60,492	7.11	$170.67	3,769	$120,983	$90,453	$211,437	6.2	$307,590
$214.50	49,334	7.11	$170.67	3,271	$98,668	$78,507	$177,175	5.0	$250,601
$218.53	52,448	7.11	$170.67	3,413	$104,896	$81,920	$186,816	5.3	$266,501
$232.04	63,810	7.11	$170.67	3,911	$127,620	$93,867	$221,486	6.5	$324,546
$199.63	38,927	7.11	$170.67	2,773	$77,855	$66,560	$144,415	3.9	$197,490
$233.89	65,490	7.11	$170.67	3,982	$130,980	$95,573	$226,553	6.7	$333,133
$214.50	49,334	7.11	$170.67	3,271	$98,668	$78,507	$177,175	5.0	$250,601
$206.15	43,292	7.11	$170.67	2,987	$86,583	$71,680	$158,263	4.4	$219,757
$216.53	50,883	7.11	$170.67	3,342	$101,767	$80,213	$181,980	5.2	$258,512
$218.53	52,448	7.11	$170.67	3,413	$104,896	$81,920	$186,816	5.3	$266,501
$199.69	38,939	7.11	$170.67	2,773	$77,877	$66,560	$144,437	4.0	$197,548
$218.60	52,463	7.11	$170.67	3,413	$104,927	$81,920	$186,847	5.3	$266,581
$220.58	54,043	7.11	$170.67	3,484	$108,087	$83,627	$191,713	5.5	$274,650
$216.59	50,899	7.11	$170.67	3,342	$101,797	$80,213	$182,010	5.2	$258,590
$197.45	37,516	7.11	$170.67	2,702	$75,033	$64,853	$139,886	3.8	$190,294
$222.55	55,638	7.11	$170.67	3,556	$111,276	$85,333	$196,609	5.7	$282,795
$201.89	40,378	7.11	$170.67	2,844	$80,755	$68,267	$149,022	4.1	$204,889
$218.60	52,463	7.11	$170.67	3,413	$104,927	$81,920	$186,847	5.3	$266,581
$204.06	41,833	7.11	$170.67	2,916	$83,666	$69,973	$153,639	4.2	$212,314
$206.21	43,304	7.11	$170.67	2,987	$86,609	$71,680	$158,289	4.4	$219,823
$216.59	50,899	7.11	$170.67	3,342	$101,797	$80,213	$182,010	5.2	$258,590
$66.02	11,223	7.11	$170.67	2,418	$22,445	$58,027	$80,472	1.1	$54,846
$62.99	9,134	7.11	$170.67	2,062	$18,268	$49,493	$67,761	0.9	$44,517
$70.42	14,788	7.11	$170.67	2,987	$29,576	$71,680	$101,256	1.5	$72,526
$69.36	13,872	7.11	$170.67	2,844	$27,744	$68,267	$96,011	1.4	$67,980
$73.93	18,114	7.11	$170.67	3,484	$36,227	$83,627	$119,854	1.8	$89,063
$70.94	15,252	7.11	$170.67	3,058	$30,503	$73,387	$103,890	1.5	$74,830
$69.36	13,872	7.11	$170.67	2,844	$27,744	$68,267	$96,011	1.4	$67,980
$69.36	13,872	7.11	$170.67	2,844	$27,744	$68,267	$96,011	1.4	$67,980
$74.89	19,097	7.11	$170.67	3,627	$38,194	$87,040	$125,234	1.9	$93,959
$71.87	15,452	7.11	$170.67	3,058	$30,904	$73,387	$104,291	1.5	$75,866
$79.63	23,492	7.11	$170.67	4,196	$46,984	$100,693	$147,677	2.3	$115,924
$78.73	22,437	7.11	$170.67	4,053	$44,874	$97,280	$142,154	2.2	$110,659
$65.62	16,078	7.11	$170.67	3,484	$32,156	$83,627	$115,783	1.6	$78,548
$69.43	20,482	7.11	$170.67	4,196	$40,964	$100,693	$141,658	2.0	$100,379
$72.92	25,159	7.11	$170.67	4,907	$50,318	$117,760	$168,078	2.5	$123,617
$64.82	15,232	7.11	$170.67	3,342	$30,464	$80,213	$110,678	1.5	$74,363
$68.33	19,131	7.11	$170.67	3,982	$38,263	$95,573	$133,836	1.9	$93,676
$66.80	17,369	7.11	$170.67	3,698	$34,738	$88,747	$123,485	1.7	$84,942
$70.15	21,396	7.11	$170.67	4,338	$42,793	$104,107	$146,900	2.1	$104,917
$57.44	16,656	7.11	$170.67	4,124	$33,312	$98,987	$132,299	1.6	$80,708
$60.98	21,951	7.11	$170.67	5,120	$43,903	$122,880	$166,783	2.1	$106,773
$56.90	15,931	7.11	$170.67	3,982	$31,861	$95,573	$127,435	1.5	$77,144
$49.93	15,479	7.11	$170.67	4,409	$30,958	$105,813	$136,771	1.5	$74,261
$50.87	17,041	7.11	$170.67	4,764	$34,081	$114,347	$148,428	1.6	$81,867
$42.62	8,523	7.11	$170.67	2,844	$17,047	$68,267	$85,313	0.8	$40,353
$44.56	11,141	7.11	$170.67	3,556	$22,281	$85,333	$107,615	1.1	$52,954
$42.41	8,270	7.11	$170.67	2,773	$16,540	$66,560	$83,100	0.8	$39,135
$44.56	11,141	7.11	$170.67	3,556	$22,281	$85,333	$107,615	1.1	$52,954
$40.64	6,299	7.11	$170.67	2,204	$12,597	$52,907	$65,504	0.6	$29,688
$43.22	9,293	7.11	$170.67	3,058	$18,587	$73,387	$91,973	0.9	$44,054
$31.08	5,750	7.11	$170.67	2,631	$11,500	$63,147	$74,647	0.5	$26,304
$30.69	4,910	7.11	$170.67	2,276	$9,819	$54,613	$64,433	0.4	$22,422
$31.08	5,750	7.11	$170.67	2,631	$11,500	$63,147	$74,647	0.5	$26,304
$32.55	9,602	7.11	$170.67	4,196	$19,203	$100,693	$119,896	0.9	$44,177
$31.99	7,998	7.11	$170.67	3,556	$15,996	$85,333	$101,329	0.7	$36,720
	1,694,627			199,253	$3,389,254	$4,782,080	$8,171,334	169.9	$8,495,939

File: DV4.WK1

TABLE A.8
Hydraulic Data & Cost Estimates for Deer Valley Road Channel (Hayden to Pima Roads), No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Channel Depth With Freeboard	Channel Topwidth With Freeboard	Approximate Toedown Depth (ft)	Unit Excavation Volume (cy/LF)
Start @ 5+00													
STA 5+00 to 8+40	340	1,009	0.0147	0.045	2.00	4.0	22.2	8.4	0.83	6.0	46.2	6.0	18.71
STA 8+40 to 16+40	800	1,009	0.0125	0.045	2.00	4.0	24.4	7.8	0.77	6.0	48.4	6.0	19.20
STA 16+40 to 26+80	1,040	1,009	0.0096	0.045	2.00	4.0	28.3	6.9	0.68	6.0	52.3	6.0	20.07
STA 26+80 to 34+70	790	1,009	0.0127	0.045	2.00	4.0	24.2	7.8	0.77	6.0	48.2	6.0	19.16
STA 34+70 to 43+80	910	559	0.0110	0.045	2.00	4.0	12.8	6.7	0.70	6.0	36.8	6.0	16.62
STA 43+80 to 49+20	540	559	0.0185	0.045	2.00	4.0	8.8	8.3	0.89	6.0	32.8	6.0	15.73
STA 49+20 to 55+00	580	559	0.0172	0.045	2.00	4.0	9.3	8.1	0.86	6.0	33.3	6.0	15.84
STA 55+00 to 56+50	150	559	0.0267	0.045	2.00	4.0	6.5	9.6	1.05	6.0	30.5	6.0	15.23
	5,150												

TABLE A.8
Hydraulic Data & Cost Estimates for Deer Valley Road Channel (Hayden to Pima Roads), No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF

Unit Excavation Cost ($/LF)	Total Channel Excavation Volume For This Reach (cy)	Unit Channel Lining Volume (cy/LF)	Unit Channel Lining Cost ($/LF)	Total Channel Lining Volume For This Reach (cy)	Total Excavation Cost For This Channel Reach	Total Lining Cost For This Channel Reach	Total Channel Construction Cost For This Reach	Channel Right-of-Way Requirements With Two 15 ft Maintenance Roads (acres)	Channel Right-of-Way Cost @ $50,000/Acre
$37.42	6,361	7.11	$170.67	2,418	$12,723	$58,027	$70,749	0.6	$29,736
$38.39	15,357	7.11	$170.67	5,689	$30,715	$136,533	$167,248	1.4	$71,979
$40.13	20,870	7.11	$170.67	7,396	$41,739	$177,493	$219,233	2.0	$98,248
$38.32	15,135	7.11	$170.67	5,618	$30,269	$134,827	$165,096	1.4	$70,920
$33.23	15,120	7.11	$170.67	6,471	$30,240	$155,307	$185,547	1.4	$69,744
$31.47	8,496	7.11	$170.67	3,840	$16,993	$92,160	$109,153	0.8	$38,927
$31.69	9,189	7.11	$170.67	4,124	$18,377	$98,987	$117,364	0.8	$42,136
$30.46	2,285	7.11	$170.67	1,067	$4,569	$25,600	$30,169	0.2	$10,423
	92,813			36,622	$185,626	$878,933	$1,064,559	8.6	$432,114

File: DVS4.WK1

TABLE A.9
Hydraulic Data & Cost Estimates for Deer Valley Road Channel (West of Hayden & South on Scottsdale Road), No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF
Channel Freeboard (ft): 2

Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Manning's Roughness	Horizontal Component of Side-slope	Flow Depth (ft)	Bottom Width (ft)	Velocity (fps)	Froude Number	Channel Depth With Freeboard	Channel Topwidth With Freeboard	Approximate Toedown Depth (ft)	Unit Excavation Volume (cy/LF)
Start @ 5+00													
STA 5+00 to 10+90	590	2,118	0.0169	0.045	2.00	4.0	46.6	9.7	0.91	6.0	70.6	6.0	24.14
STA 10+90 to 15+90	500	2,118	0.0200	0.045	2.00	4.0	42.7	10.4	0.99	6.0	66.7	6.0	23.26
STA 15+90 to 22+80	690	2,118	0.0145	0.045	2.00	4.0	50.6	9.0	0.85	6.0	74.6	6.0	25.03
STA 22+80 to 27+80	500	1,564	0.0200	0.045	2.00	4.0	30.7	10.1	0.98	6.0	54.7	6.0	20.59
STA 27+80 to 35+50	770	1,564	0.0130	0.045	2.00	4.0	38.8	8.3	0.80	6.0	62.8	6.0	22.41
STA 35+50 to 40+50	500	1,564	0.0200	0.045	2.00	4.0	30.7	10.1	0.98	6.0	54.7	6.0	20.59
STA 40+50 to 48+30	780	1,564	0.0128	0.045	2.00	4.0	39.1	8.3	0.79	6.0	63.1	6.0	22.47
STA 48+30 to 53+70	540	1,564	0.0185	0.045	2.00	4.0	32.0	9.8	0.94	6.0	56.0	6.0	20.89
STA 53+70 to 63+30	960	1,564	0.0104	0.045	2.00	4.0	43.7	7.6	0.72	6.0	67.7	6.0	23.50
STA 63+30 to 71+40	810	1,511	0.0123	0.045	2.00	4.0	38.5	8.1	0.78	6.0	62.5	6.0	22.33
STA 71+40 to 83+30	1,190	1,511	0.0084	0.045	2.00	4.0	47.3	6.8	0.64	6.0	71.3	6.0	24.28
STA 83+30 to 95+10	1,180	756	0.0085	0.045	2.00	4.0	21.9	6.3	0.63	6.0	45.9	6.0	18.64
STA 95+10 to 101+20	610	756	0.0066	0.045	2.00	4.0	25.4	5.7	0.56	6.0	49.4	6.0	19.41
	9,620												

TABLE A.9
Hydraulic Data & Cost Estimates for Deer Valley Road Channel (West of Hayden & South on Scottsdale Road), No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
All Calculations Are Based On Existing Ground Slopes, 25 Sq Mi ARF

Unit Excavation Cost ($/LF)	Total Channel Excavation Volume For This Reach (cy)	Unit Channel Lining Volume (cy/LF)	Unit Channel Lining Cost ($/LF)	Total Channel Lining Volume For This Reach (cy)	Total Excavation Cost For This Channel Reach	Total Lining Cost For This Channel Reach	Total Channel Construction Cost For This Reach	Channel Right-of-Way Requirements With Two 15 ft Maintenance Roads (acres)	Channel Right-of-Way Cost @ $50,000/Acre
$48.27	14,240	7.11	$170.67	4,196	$28,481	$100,693	$129,174	1.4	$68,138
$46.52	11,630	7.11	$170.67	3,556	$23,261	$85,333	$108,594	1.1	$55,483
$50.06	17,272	7.11	$170.67	4,907	$34,544	$117,760	$152,304	1.7	$82,880
$41.19	10,297	7.11	$170.67	3,556	$20,594	$85,333	$105,927	1.0	$48,596
$44.82	17,256	7.11	$170.67	5,476	$34,513	$131,413	$165,926	1.6	$82,063
$41.19	10,297	7.11	$170.67	3,556	$20,594	$85,333	$105,927	1.0	$48,596
$44.94	17,528	7.11	$170.67	5,547	$35,055	$133,120	$168,175	1.7	$83,372
$41.78	11,281	7.11	$170.67	3,840	$22,562	$92,160	$114,722	1.1	$53,311
$46.99	22,557	7.11	$170.67	6,827	$45,113	$163,840	$208,953	2.2	$107,696
$44.65	18,084	7.11	$170.67	5,760	$36,168	$138,240	$174,408	1.7	$85,971
$48.56	28,895	7.11	$170.67	8,462	$57,790	$203,093	$260,883	2.8	$138,323
$37.27	21,990	7.11	$170.67	8,391	$43,979	$201,387	$245,366	2.1	$102,747
$38.82	11,841	7.11	$170.67	4,338	$23,682	$104,107	$127,789	1.1	$55,561
	213,168			68,409	$426,336	$1,641,813	$2,068,149	20.3	$1,012,737

File: RPS.WK1

TABLE A.10
Hydraulic Data & Cost Estimates for Reata Pass Channel, 8-Point Section, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Low-Flow Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
Channel Freeboard (ft): 2.0
Supercritical Slope, 25 Sq Mi ARF, Thompson Peak Parkway Alignment
Channel n=.045, Overbank n=.055

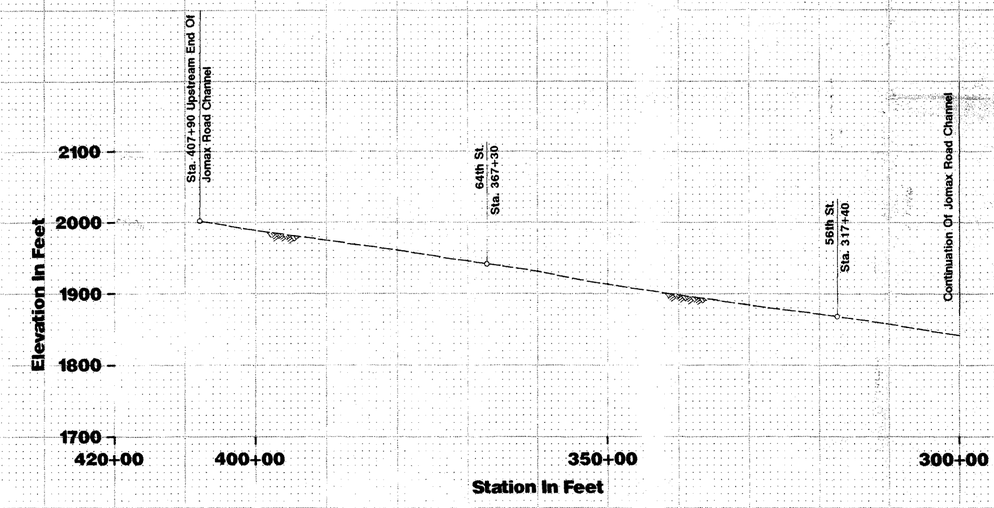
Channel Reach	Reach Length (ft)	Discharge (cfs)	Bed Slope (ft/ft)	Low-Flow Bottomwidth (ft)	Low-Flow Depth (ft)	Low-Flow Side-Slope	Bench Width, Each Side (ft)	Bench Side-Slope	Flow Depth (ft)	Approximate Toedown Depth (ft)
Start @ 0+00										
STA 0+00 to 9+00	900	19,449	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 9+00 to 19+90	1090	19,449	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 19+90 to 27+70	780	19,449	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 27+70 to 34+20	650	16,623	0.0200	75	3.0	5.0	100	5.0	6.59	8.5
STA 34+20 to 39+20	500	16,623	0.0200	75	3.0	5.0	100	5.0	6.59	8.5
STA 39+20 to 42+40	320	16,623	0.0200	75	3.0	5.0	100	5.0	6.59	8.5
STA 42+40 to 52+90	1050	16,623	0.0200	75	3.0	5.0	100	5.0	6.59	8.5
STA 52+90 to 55+40	250	20,251	0.0200	75	3.0	5.0	100	5.0	7.06	8.5
STA 55+40 to 68+60	1320	20,251	0.0200	75	3.0	5.0	100	5.0	7.06	8.5
STA 68+60 to 75+10	650	20,279	0.0200	75	3.0	5.0	100	5.0	7.06	8.5
STA 75+10 to 80+10	500	20,279	0.0200	75	3.0	5.0	100	5.0	7.06	8.5
STA 80+10 to 88+60	850	18,601	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 88+60 to 103+10	1450	18,601	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 103+10 to 107+60	450	19,172	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 107+60 to 116+10	850	19,172	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 116+10 to 122+10	600	18,719	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 122+10 to 127+60	550	18,719	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 127+60 to 134+60	700	18,719	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 134+60 to 140+60	600	18,719	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 140+60 to 146+10	550	18,719	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 146+10 to 151+60	550	18,719	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 151+60 to 153+50	190	18,719	0.0200	75	3.0	5.0	100	5.0	6.91	8.5
STA 153+50 to 156+40	290	19,524	0.0200	75	3.0	5.0	100	5.0	7.06	8.5
STA 156+40 to 160+00	360	19,524	0.0200	75	3.0	5.0	100	5.0	7.06	8.5
STA 160+00 to 167+30	730	19,524	0.0200	75	3.0	5.0	100	5.0	7.06	8.5
STA 167+30 to 179+30	1200	19,524	0.0200	75	3.0	5.0	100	5.0	7.06	8.5
STA 179+30 to 181+30	200	15,025	0.0200	75	3.0	5.0	100	5.0	6.26	8.5
STA 181+30 to 189+80	850	15,025	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 189+80 to 198+30	850	14,986	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 198+30 to 205+30	700	14,986	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 205+30 to 209+80	450	14,986	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 209+80 to 214+10	430	14,986	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 214+10 to 219+10	500	14,986	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 219+10 to 224+30	520	14,986	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 224+30 to 233+80	950	14,986	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 233+80 to 241+80	800	15,403	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 241+80 to 250+50	870	15,403	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 250+50 to 255+10	460	15,403	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 255+10 to 257+10	200	15,403	0.0200	75	3.0	5.0	100	5.0	6.43	8.5
STA 257+10 to 262+90	580	14,330	0.0200	75	3.0	5.0	100	5.0	6.08	8.5
STA 262+90 to 269+40	650	14,330	0.0200	75	3.0	5.0	100	5.0	6.08	8.5
STA 269+40 to 273+00	360	14,330	0.0200	75	3.0	5.0	100	5.0	6.08	8.5
STA 273+00 to 276+10	310	14,330	0.0200	75	3.0	5.0	100	5.0	6.08	8.5
STA 276+10 to 285+10	900	14,935	0.0200	75	3.0	5.0	100	5.0	6.26	8.5
STA 285+10 to 291+80	670	14,935	0.0200	75	3.0	5.0	100	5.0	6.26	8.5
	29,180									
Additional Quantities for Protecting Upper Bench from Erosion Due to Side Inflows	14,590									
Total System Cost With Upper Bench Protection										

File: RP5.WK1

TABLE A.10
Hydraulic Data & Cost Estimates for Reata Pass Channel, 8-Point Section, No Low-Flow Culverts
Upper Indian Bend Wash Regional Flood Control & Drainage Plan, Reata Pass to CAP Dike 4
Excavation Cost/c.y.: $2.00
Low-Flow Channel Lining: Soil Cement Banks, Earth Bottom
Horizontal Lining Width (ft): 8.00
Lining Cost/c.y.: $24.00
Channel Freeboard (ft): 2.0
Supercritical Slope, 25 Sq Mi ARF, Thompson Peak Parkway Alignment
Channel n=.045, Overbank n=.055

Excavation Volume For Channel Prism (cy)	Excavation Volume For Bank-Lining & Toe-Down (cy)	Total Excavation Volume (cy)	Total Excavation Cost	Low-Flow Bank-Lining Volume (cy)	Low-Flow Bank-Lining Cost	Total Channel Construction Cost For This Reach	Channel Topwidth With Freeboard (ft)	Right-of-Way, Based On Channel Topwidth With Freeboard (acres)
74,906	20,583	95,490	$190,979	6,133	$147,200	$338,179	364	7.5
90,720	24,929	115,649	$231,297	7,428	$178,276	$409,573	364	9.1
64,919	17,839	82,758	$165,515	5,316	$127,573	$293,089	364	6.5
51,306	14,866	66,172	$132,344	4,430	$106,311	$238,655	361	5.4
39,466	11,435	50,902	$101,803	3,407	$81,778	$183,581	361	4.1
25,259	7,319	32,577	$65,154	2,181	$52,338	$117,492	361	2.7
82,880	24,014	106,894	$213,787	7,156	$171,733	$385,520	361	8.7
21,314	5,718	27,032	$54,063	1,704	$40,889	$94,952	366	2.1
112,538	30,189	142,727	$285,454	8,996	$215,893	$501,348	366	11.1
55,417	14,866	70,282	$140,565	4,430	$106,311	$246,876	366	5.5
42,628	11,435	54,063	$108,127	3,407	$81,778	$189,904	366	4.2
70,745	19,440	90,185	$180,369	5,793	$139,022	$319,392	364	7.1
120,682	33,162	153,844	$307,689	9,881	$237,156	$544,845	364	12.1
37,453	10,292	47,745	$95,490	3,067	$73,600	$169,090	364	3.8
70,745	19,440	90,185	$180,369	5,793	$139,022	$319,392	364	7.1
49,938	13,722	63,660	$127,320	4,089	$98,133	$225,453	364	5.0
45,776	12,579	58,355	$116,710	3,748	$89,956	$206,665	364	4.6
58,260	16,009	74,270	$148,540	4,770	$114,489	$263,028	364	5.9
49,938	13,722	63,660	$127,320	4,089	$98,133	$225,453	364	5.0
45,776	12,579	58,355	$116,710	3,748	$89,956	$206,665	364	4.6
45,776	12,579	58,355	$116,710	3,748	$89,956	$206,665	364	4.6
15,814	4,345	20,159	$40,318	1,295	$31,076	$71,393	364	1.6
24,724	6,632	31,357	$62,713	1,976	$47,431	$110,145	366	2.4
30,692	8,233	38,926	$77,851	2,453	$58,880	$136,731	366	3.0
62,237	16,695	78,932	$157,865	4,975	$119,396	$277,260	366	6.1
102,307	27,444	129,752	$259,504	8,178	$196,267	$455,770	366	10.1
14,908	4,574	19,483	$38,965	1,363	$32,711	$71,676	358	1.6
65,279	19,440	84,719	$169,438	5,793	$139,022	$308,460	359	7.0
65,279	19,440	84,719	$169,438	5,793	$139,022	$308,460	359	7.0
53,759	16,009	69,769	$139,537	4,770	$114,489	$254,026	359	5.8
34,560	10,292	44,851	$89,702	3,067	$73,600	$163,302	359	3.7
33,024	9,834	42,858	$85,716	2,930	$70,329	$156,045	359	3.5
38,400	11,435	49,835	$99,669	3,407	$81,778	$181,447	359	4.1
39,936	11,893	51,828	$103,656	3,544	$85,049	$188,705	359	4.3
72,959	21,727	94,686	$189,372	6,474	$155,378	$344,750	359	7.8
61,439	18,296	79,736	$159,471	5,452	$130,844	$290,316	359	6.6
66,815	19,897	86,712	$173,425	5,929	$142,293	$315,718	359	7.2
35,328	10,520	45,848	$91,696	3,135	$75,236	$166,931	359	3.8
15,360	4,574	19,934	$39,868	1,363	$32,711	$72,579	359	1.6
41,855	13,265	55,120	$110,240	3,953	$94,862	$205,102	356	4.7
46,907	14,866	61,772	$123,545	4,430	$106,311	$229,856	356	5.3
25,979	8,233	34,212	$68,425	2,453	$58,880	$127,305	356	2.9
22,371	7,090	29,461	$58,921	2,113	$50,702	$109,624	356	2.5
67,088	20,583	87,671	$175,343	6,133	$147,200	$322,543	358	7.4
49,943	15,323	65,266	$130,533	4,566	$109,582	$240,115	358	5.5
2,343,405	667,357	3,010,763	$6,021,526	198,856.3	$4,772,551	$10,794,077		242.4
	53,545		$107,091	38,955.3	$934,927	$1,042,018		
			$6,128,616		$5,707,478	$11,836,095		

PLATES

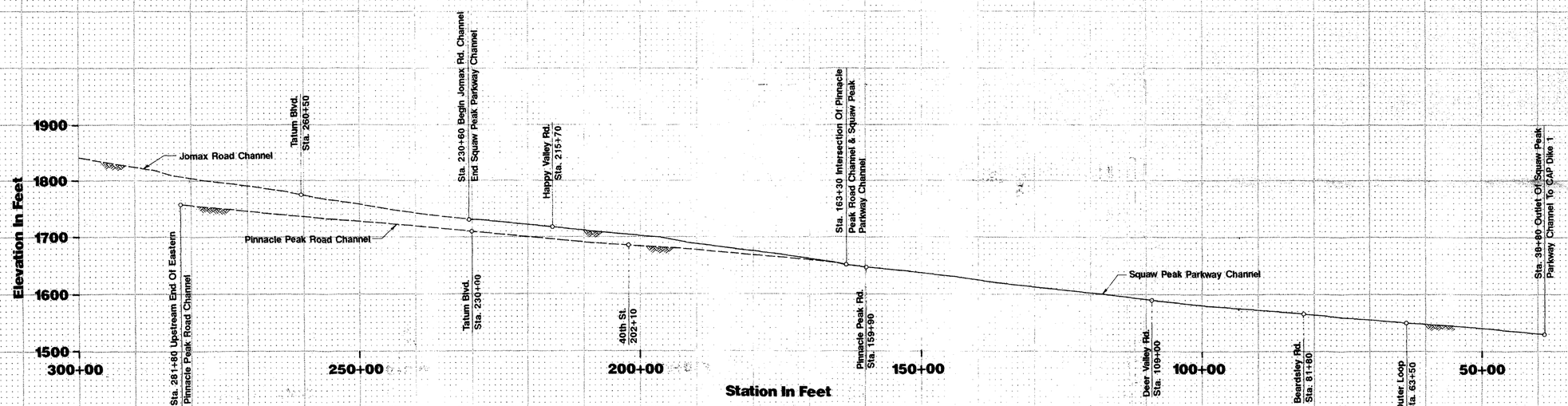


Summary Of Channel Stationing

Squaw Peak Parkway Channel - Sta. 38+80 to 230+60

Eastern Pinnacle Peak Road Channel - Sta. 163+30 to 281+80

Jomax Road Channel - Sta. 230+60 to 407+90



SCALE

Vertical : 1 Inch = 100 Feet

Horizontal : 1 Inch = 1000 Feet

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	PLOTTED		
NOTE BOOK	TEMPLATE		
NO.	AREAS		
	AREAS CHECKED		

ORIGINAL SURVEY	SURVEYED	BY	DATE
	PLOTTED		
NOTE BOOK	TEMPLATE		
NO.	AREAS		
	AREAS CHECKED		

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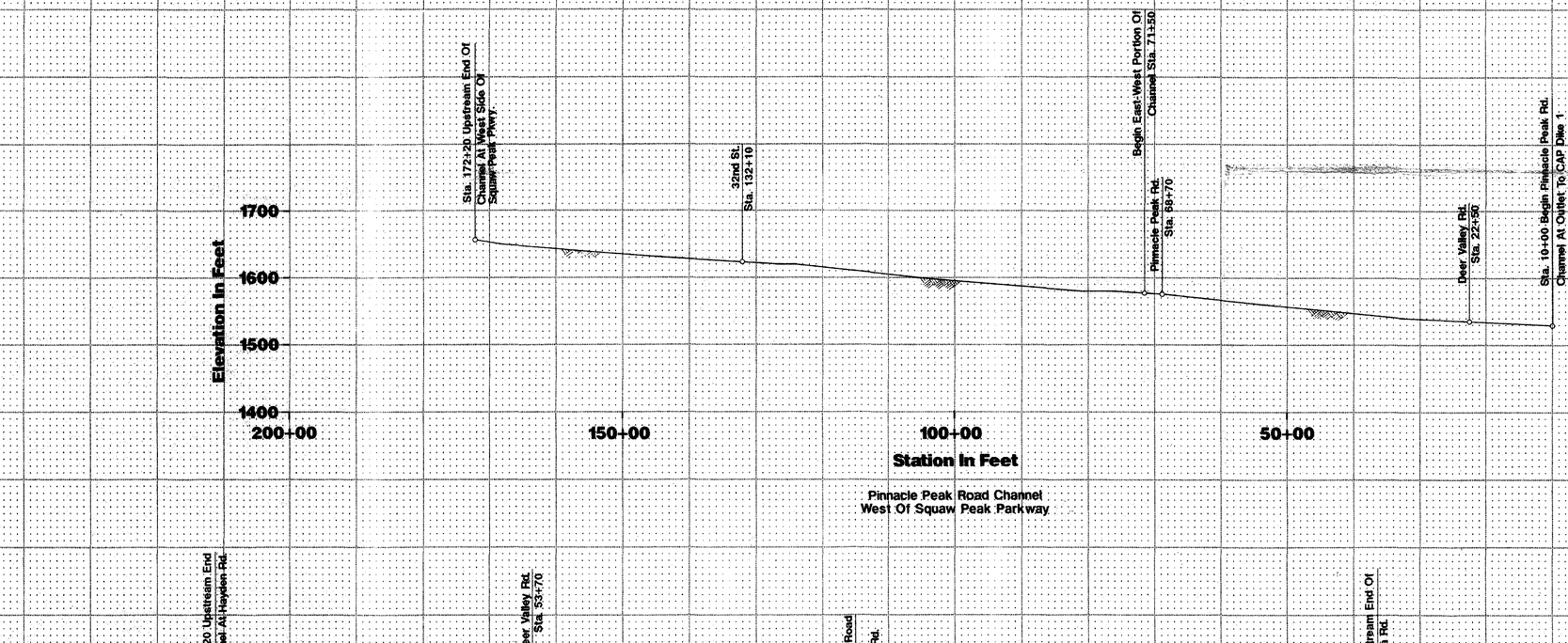
PROJECT NO. AC02670 | DRAWN BY KMA

DATE 6/15/92 | CHECKED BY RLW

EXISTING GROUND PROFILE

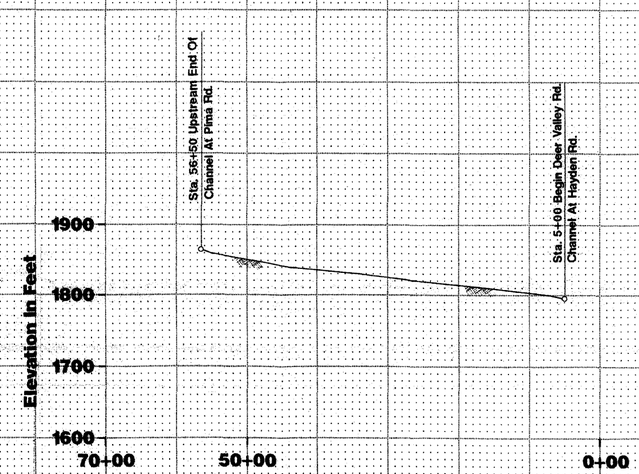
SQUAW PEAK PARKWAY CHANNEL
JOMAX ROAD CHANNEL
EAST PINNACLE PEAK ROAD CHANNEL

PLATE 5



Pinnacle Peak Road Channel
West Of Squaw Peak Parkway

Deer Valley Road Channel Around
Northwest Corner Of Core North



Deer Valley Road Channel
From Hayden Road To Pima Road

SCALE

Vertical : 1 Inch = 100 Feet

Horizontal : 1 Inch = 1000 Feet

FINAL SURVEY	BY	DATE
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NOTE BOOK		
AREAS		
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ORIGINAL SURVEY	BY	DATE
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NOTE BOOK		
AREAS		
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AREAS CHECKED		

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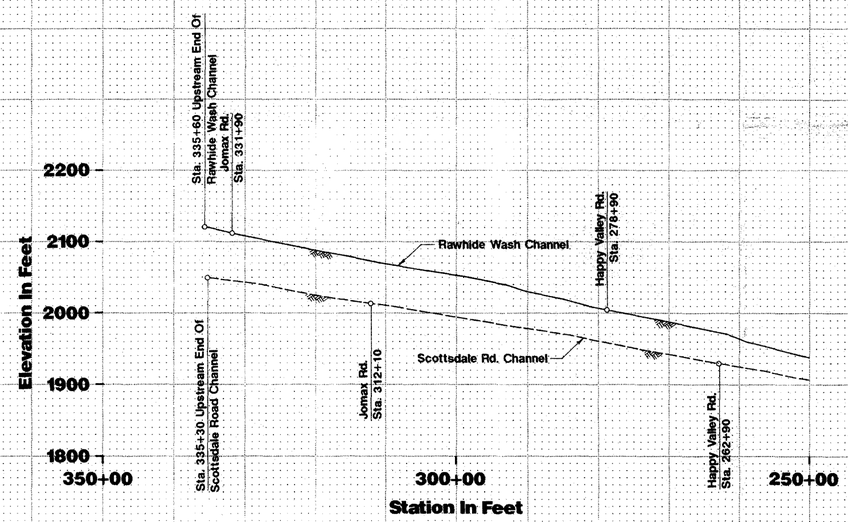
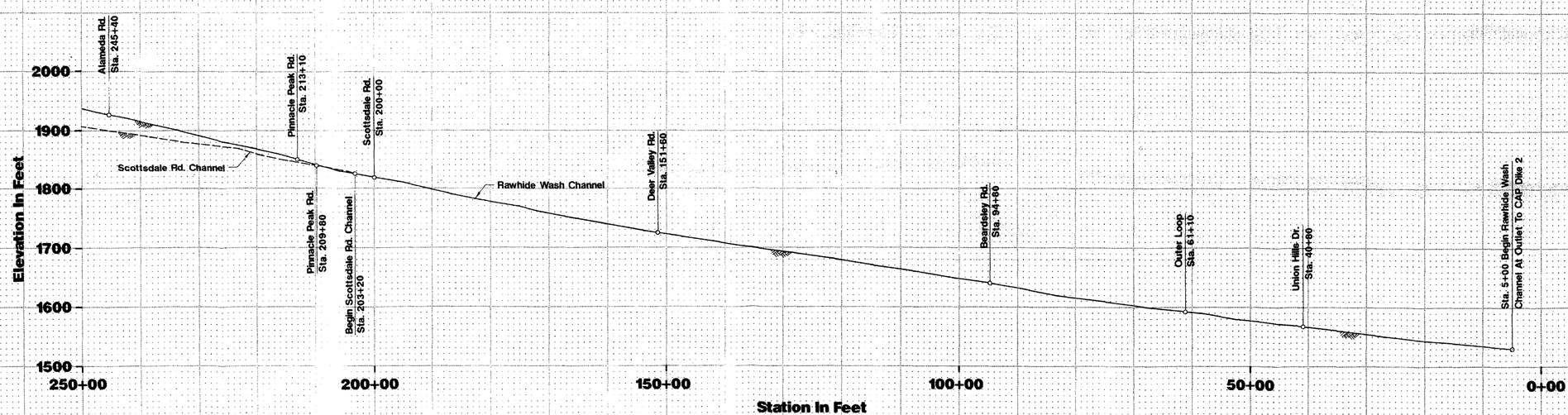
EXISTING GROUND PROFILE

WEST PINNACLE PEAK ROAD CHANNEL
DEER VALLEY ROAD CHANNEL - WEST
DEER VALLEY ROAD CHANNEL - EAST

PLATE 6

HIGHWAY FEDERAL AID SHEET
PLATE 3-FULL CROSS SECTION-FULL DOT
3M TRUEDYNE
PRINTED IN U.S.A.





SCALE

Vertical : 1 Inch = 100 Feet

Horizontal : 1 Inch = 1000 Feet

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TEMPLATE		
AREAS		
AREAS CHECKED		
NOTE BOOK NO.		

ORIGINAL SURVEY	BY	DATE
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TEMPLATE		
AREAS		
AREAS CHECKED		
NOTE BOOK NO.		

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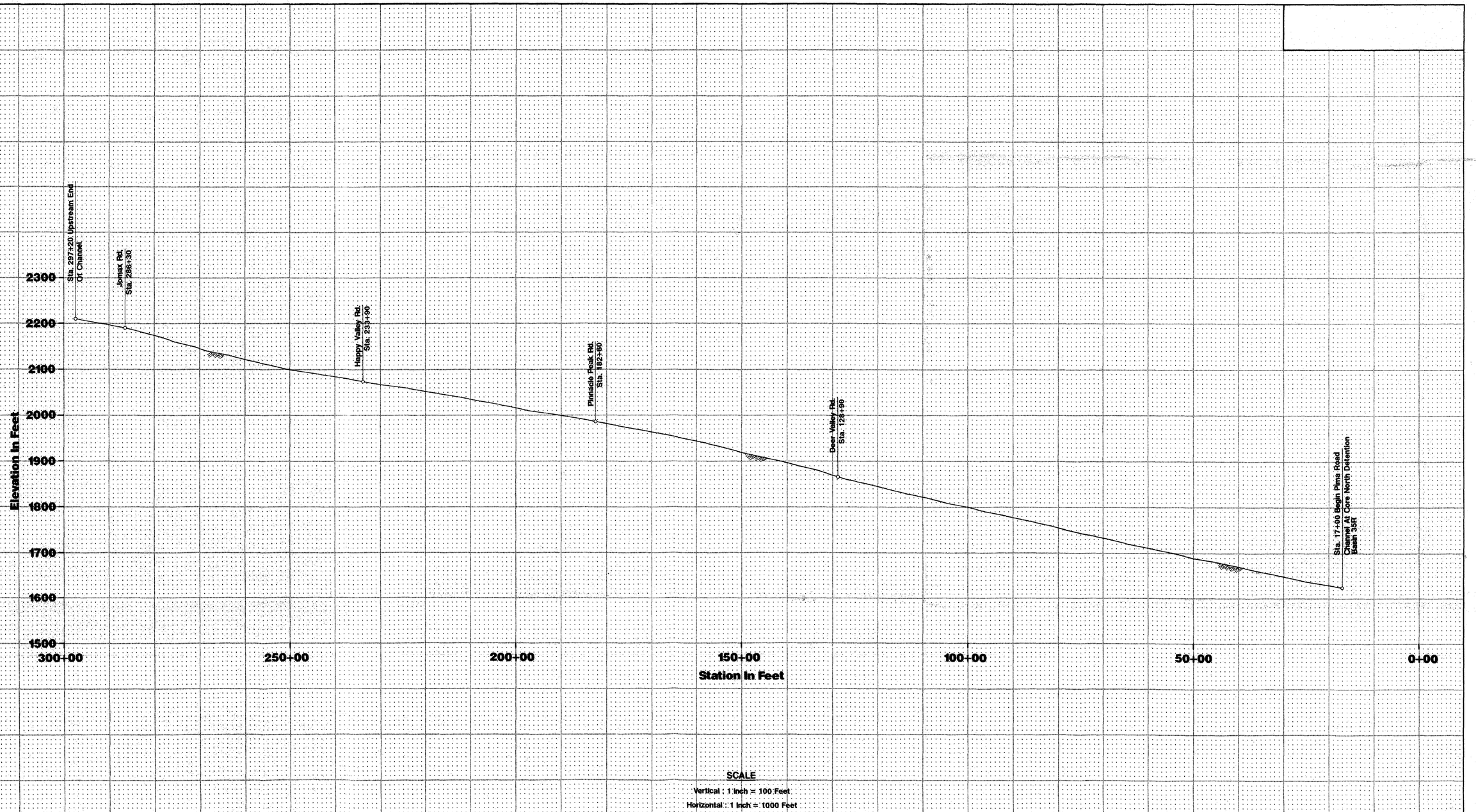
EXISTING GROUND PROFILE

RAWHIDE WASH CHANNEL
SCOTTSDALE ROAD CHANNEL

PLATE 7

HIGHWAY FEDERAL AID SHEET
PLATE 3-FULL CROSS SECTION-FULL DOT
TELEDYNE
PRINTED IN U.S.A.



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ORIGINAL SURVEY		BY	DATE
NOTE BOOK NO.	SURVEYED		
	PLOTTED		
	TEMPLATE		
	AREAS		
	AREAS CHECKED		

SCALE

Vertical : 1 Inch = 100 Feet

Horizontal : 1 Inch = 1000 Feet

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DATE 6/15/92 CHECKED BY RLW

EXISTING GROUND PROFILE

PIMA ROAD CHANNEL

PLATE 8

HIGHWAY FEDERAL AID SHEET
PLATE 3-FULL CROSS SECTION-FULL DOT
WHITELEYNE
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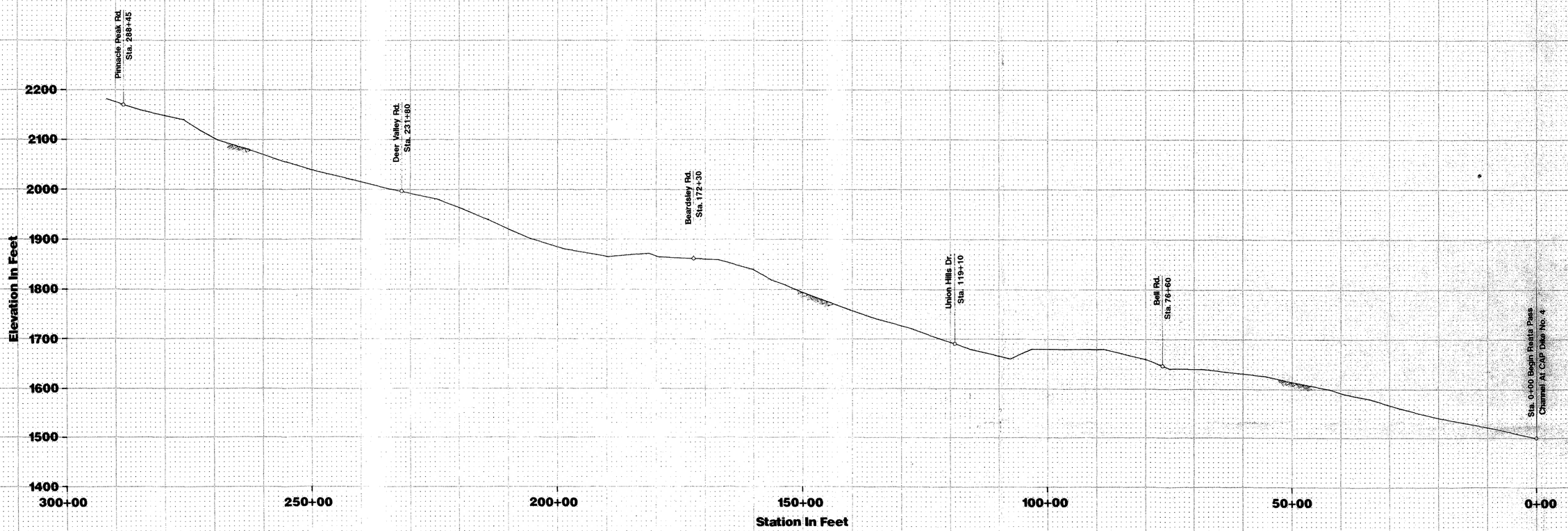

FINAL SURVEY		BY	DATE
NOTE BOOK NO.	SURVEYED		
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	TEMPLATE		
	AREAS		
	AREAS CHECKED		

ORIGINAL SURVEY		BY	DATE
NOTE BOOK NO.	SURVEYED		
	PLOTTED		
	TEMPLATE		
	AREAS		
	AREAS CHECKED		

SCALE

Vertical : 1 Inch = 100 Feet

Horizontal : 1 Inch = 1000 Feet

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DATE 6/15/92 | CHECKED BY RLW

EXISTING GROUND PROFILE

REATA PASS CHANNEL

EAST SIDE OF THOMPSON PEAK PARKWAY

PLATE 9

HIGHWAY FEDERAL AID SHEET
PLATE 3-FULL CROSS SECTION-FULL DOT
TELEDYNE
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