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CONCEPT DRAINAGE STUDY
PARADISE VALLEY FAN TERRACE
PART OF PERIPHERAL AREAS C AND D

ST-886374

Water Resources Associates, Inc.
Hydrologists • Hydrogeologists • Engineers • Forensic Hydrologists

CONCEPT DRAINAGE STUDY
PARADISE VALLEY FAN TERRACE
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ST-886374

C3302

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TABLE OF CONTENTS

	PAGE NUMBER
TABLE OF CONTENTS	i
LIST OF FIGURES	iii
LIST OF TABLES	v
LIST OF PLATES	viii
LIST OF APPENDICES	ix
EXECUTIVE SUMMARY	1
1.0 INTRODUCTION	9
2.0 DRAINAGE BASIN CHARACTERISTICS	11
2.1 Drainage Area	11
2.2 Land Use	12
2.3 Soil Data and Vegetation	13
2.4 Existing Drainage Improvements	15
3.0 HYDROLOGIC MODEL (HEC-1)	17
3.1 Delineation of Drainage Sub-Basins	18
3.2 Interception/Infiltration	19
3.3 Overland Flow Parameters	22
3.4 Channel Routing Parameters	25
3.5 Rainfall Parameters	28
3.6 Sensitive Analysis and Model Verification	29
3.6.1 Rainfall Sensitivity Analysis	29
3.6.2 Verification of HEC-1 Model	36
4.0 DESIGN GUIDELINES FOR RETENTION BASIN ANALYSIS	44
5.0 DESIGN GUIDELINES FOR CHANNELIZATION ANALYSIS	51
5.1 Lined and Unlined Channels	51
5.2 Channel Cut-off Walls	53
5.3 Drop Structures	54
5.4 Bridges	56
5.5 Low-flow Culverts	57

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TABLE OF CONTENTS
(Continued)

	PAGE NUMBER
6.0 DISCUSSION OF FLOODPLAIN MANAGEMENT OPTIONS	59
6.1 Channelization	59
6.1.1 Cave Creek Road Channel	61
6.1.2 Lone Mountain, Dixileta and Dynamite Road Channels	62
6.1.3 Channelization Summary	63
6.2 Detention/Retention Basins	65
6.2.1 Retention Basin Analysis	68
6.2.2 Detention Basin Analysis	69
6.2.3 Detention/Retention Basin Summary	71
6.3 Tatum Ranch	72
6.4 Desert Ridge	79
6.4.1 Preliminary BRW Concept	80
6.4.2 Major Interceptor Channel Concept	81
6.4.2.1 Channel Hydrology	82
6.4.2.2 Channel Hydraulics	86
6.4.2.3 Cost Estimates	91
6.5 FEMA Flood Insurance Study	93
6.6 "No Action" Alternative	96
7.0 STORM DRAIN SYSTEM	100
7.1 Hydraulic Assumptions	100
7.2 Concept Design Dimensions	102
7.3 Cost Estimates	103
8.0 CONCLUSIONS AND RECOMMENDATIONS	104
9.0 PUBLIC MEETING	109
FIGURES	111
TABLES	133
PLATES	167
APPENDICES	178

LIST OF FIGURES

FIGURE NUMBER	DESCRIPTION	PAGE NUMBER
1.1	Study Boundaries	112
3.1	Percent of Impervious Cover vs. Dwelling Units Per Acre	113
3.2	Comparison of Rainfall Distributions	114
3.3	Comparison of Peak Discharge Using NOAA vs Osborn/Renard Rainfall	115
3.4	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3020	116
3.5	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3135	117
3.6	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3150	118
3.7	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3160	119
3.8	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3400	120
3.9	Graphical Plot of Independent Peak Discharge Calculations for Concentration Point 3490	121
3.10	Unit Discharge vs. Drainage Area (100-Year Event, Non-sheetflow Areas)	122

LIST OF FIGURES
(Continued)

FIGURE NUMBER	DESCRIPTION	PAGE NUMBER
3.11	Unit Discharge vs. Drainage Area (100-Year Event, Sheetflow Areas)	123
4.1	Cross-Section of Typical Retention Basin Outlet System	124
5.1	Concrete-Lined Channel Section	125
5.2	Soil-Cement Bank, Earth Bottom Channel Section	126
5.3	Earth Channel Section	127
5.4	Channel Lining Cut-off Wall	128
5.5	Typical Soil-Cement Drop Structure Geometry	129
5.6	Soil-Cement Drop Structure Costs	130
5.7	Bank-Lining Costs Required for Drop Structures	131
5.8	Cross-Section of Low-flow Culvert Outlets From Man Made Channels	132

LIST OF TABLES

TABLE NUMBER	DESCRIPTION	PAGE NUMBER
2.1	Summary of Hydrologic Soil Group Assigned to Soil Complex/Associations	134
3.1	Summary of SCS Curve Numbers as a Function of Storm Duration and Hydrologic Soil Group	136
3.2	Summary of Overland Flow Roughness Values	137
3.3	Summary of Areal Rainfall Values	138
3.4	Summary of Sensitivity Analyses for Rainfall Parameters	139
3.5	Summary of Independent Peak Discharge Calculations for Verification of HEC-1 Model Results	140
4.1	Summary of Regression Equations for Estimating Retention Basin Size and Cost	141
4.2	Summary of Extreme Values in the Data Sets Used for Retention Basin Regression Equations	143
5.1	Summary of Construction Costs for Cutoff Wall Along Top of Channel Bank	144
6.1	Peak Discharge Summary for Cave Creek Road Channel, 100-Year, 24 Hour Event	145
6.2	Reduction in Peak Discharge for Areas Downstream of Cave Creek Road Channel 100-Year, 24 Hour Event	146
6.3	Summary of Peak Discharge Values and Channel Hydraulics for Lone Mountain, Dixileta, and Dynamite Roads, 100-Year, 24 Hour Event	147

LIST OF TABLES
(Continued)

TABLE NUMBER	DESCRIPTION	PAGE NUMBER
6.4	Reduction in Peak Discharge for Areas Downstream of Lone Mountain, Dixileta, and Dynamite Road Channels, 100-Year, 24 Hour Event	148
6.5	Summary of Retention Basin Analysis	149
6.6	Summary of Detention Basin Analysis	150
6.7	Peak Discharge Summary for Original East-West Desert Ridge Channel Beginning at Jomax Road	151
6.8	Peak Discharge Summary for East-West Desert Ridge Channel Along Pinnacle Peak Road	152
6.9	Peak Discharge Summary for North-South Desert Ridge Channel	153
6.10	Summary of Hydraulic Parameters for Desert Ridge Channel Analysis	154
6.11	Summary of Channel Costs, No Low-flow Culverts, Desert Ridge Original East-West Alignment Beginning at Jomax Road	155
6.12	Summary of Channel Costs, with Original East-West Alignment Beginning at Jomax Road	156
6.13	Summary of Channel Costs Desert Ridge East-West Pinnacle Peak Road Alignment	157
6.14	Summary of Channel Costs, No Low-flow Culverts Desert Ridge North-South Alignment	158
6.15	Summary of Channel Costs, with Low-flow Culverts Desert Ridge North-South Alignment	159

LIST OF TABLES
(Continued)

TABLE NUMBER	DESCRIPTION	PAGE NUMBER
6.16	Summary of Total Channel Construction Costs Desert Ridge Channel Alignments	160
6.17	Channel Right-of-Way requirements, Concrete Lined Section (n= 0.020), 100-Year, 24 Hour Event	161
6.18	Right-of-Way Summary for Desert Ridge Channels, Soil Cement Bank Lining	162
6.19	Right-of-Way Summary for Desert Ridge Channels, Earth Lining	163
7.1	Cost of Mainline Pipe Storm Drains (cost per lin. ft.)	164
7.2	Summary of Construction Cost Estimates for Storm Drains	166

LIST OF PLATES

PLATE NUMBER	DESCRIPTION	PAGE NUMBER
1	Hydraulic Soil Group Map and Rainfall Zones	168
2	Watershed Sub-Basin Delineations and HEC-1 Routing Schematic for Existing Drainage Pattern	169
3	Schematic Diagram for Area-Wide Channel System	170
4	HEC-1 Routing Schematic for Cave Creek Road Channel	171
5	HEC-1 Routing Schematic for Lone Mountain, Dixileta, and Dynamite Road Channels	172
6A	HEC-1 Routing Schematic for Desert Ridge Interceptor Channels, Jomax and Scottsdale Road Alignments	173
6B	HEC-1 Routing Schematic for Desert Ridge Interceptor Channel Along Pinnacle Peak Road	174
7	Centerline Channel Profile for Original East-West Desert Ridge Channel	175
8	Centerline Channel Profile for North-South Desert Ridge Channel	176
9	Schematic Diagram for Storm Drain System	177

LIST OF APPENDICES

APPENDIX NUMBER	DESCRIPTION	PAGE NUMBER
A	Regression Data for Retention Basin Design Equations (123 Spreadsheets)	A-1
B	Channel Design Curves	B-1
C	Channel Design Spreadsheets for Desert Ridge	C-1
D	Mailing List for Public Meeting Attendees and Peripheral Area C & D Planning Advisory Committee Members	D-1

EXECUTIVE SUMMARY

In recent years, the metropolitan Phoenix area has demonstrated an affinity for urban expansion into previously undeveloped portions of the Sonoran Desert. The natural beauty of these desert areas has created an urgency for municipalities to formulate planning guidelines that will ensure that the impending urbanization of these areas will be done in a controlled manner that will preserve and enhance the aesthetics of the desert.

In support of this goal, the City of Phoenix has recently published a General Plan for Peripheral Areas C and D (City of Phoenix Planning Department, October, 1987). This plan provides a very comprehensive discussion of policies and recommendations related to all aspects of urbanizing the desert environment. An integral part of the General Plan identifies the need for a drainage system that would support a major objective of preservation of natural desert washes and native Sonoran vegetation and wildlife.

In recognition of these issues, the City of Phoenix retained Water Resources Associates, Inc. (WRA) to undertake a drainage study for approximately 56 square miles of Peripheral Areas C and D. This study area can generally be described as that area located east of Cave Creek, west of Scottsdale Road, and bounded on the north and south by the Carefree Highway and the Central Arizona Project Aquaduct, respectively. Figure 1.1 illustrates the study area boundaries.

The primary objectives of this study are to quantify the rainfall/runoff response of the watershed and to evaluate the effectiveness and cost of different drainage solutions that might be appropriate for this area.

A substantial amount of technical data has been generated in pursuit of these objectives. Section 2.0 and 3.0 of this report provide details of the hydrology analysis that culminated in the development of HEC-1 models that were used to quantify the rainfall/runoff response of the watershed. Sections 4.0 and 5.0 provide regression equations and design curves that can be used to determine the size and cost of retention basins and flood control channels. Section 6.0 focuses on detailed discussions of specific floodplain management options that were considered for possible use within the study area, while Section 7.0 presents the results of an analysis of an underground storm drain system. Section 8.0 is devoted to a summary of conclusions and recommendations. The complete report is accompanied by numerous illustrations and tables of hydrologic, hydraulic, and cost estimating data. Complete computer printouts of all HEC-1 input and output data are also provided as a separate volume.

The following floodplain management options were investigated as part of this study:

1. Channelization

2. Detention/Retention Basins

3. Tatum Ranch Drainage Plan

4. Desert Ridge Drainage Plan

5. FEMA Floodplain Management Policies

6. "No Action" Alternative.

A brief summary of each option is provided in the following paragraphs.

Channelization

Channelization is an extremely effective solution for the interception and conveyance of floodwaters through an urban area. However, the efficiency of this solution must be weighed against the sometimes high installation cost and aesthetic impact that such a solution would have on the natural beauty of the desert environment.

The channelization schemes that were analyzed in this report were all of the "interceptor" variety, i.e. none of the channels were aligned along an existing wash. With two exceptions, all of the evaluated channel alignments were located adjacent to existing or proposed roadways. The natural desert washes would remain undisturbed, except at their intersection with the roadways.

The interceptor channel concept includes low-flow culverts that would continue to release water (at a controlled, non-flooding rate) to those downstream segments of natural washes that are intersected by the interceptor channel. These low-flow culverts are included to maintain the natural vegetation community along the desert washes.

Interceptor channels were evaluated along Cave Creek Road, Lone Mountain Road, Dynamite Road, Dixileta Road, south of Jomax Road, Pinnacle Peak Road, and Scottsdale Road. Cost estimates were only developed for the channel alignments south of Jomax Road, along Pinnacle Peak Road, and along Scottsdale Road. Costs were developed as a function of three different channel lining options:

1. Concrete banks and bed

2. Soil cement banks, earth bed

3. Earth banks and bed

The results of the analysis indicated that the soil cement option is a preferred approach for a major interceptor channel. Tables 6.10 through 6.19 present tabulated design parameters and cost summaries for the three referenced channel alignments. In summary, fully lined concrete sections are more expensive than the soil cement option, while the earth lined section is considered unsuitable for use in a steep-sloped, sheetflow environment. Earth channels are exceptionally prone to erosion damage.

Although not subjected to a hydrologic and hydraulic analysis, a complete channelization system was developed for the entire study area. This system, which is illustrated on Plate 3, shows specific channel alignments and outfall locations. The proposed channel alignments follow the transportation grid presented in the 1987 General Plan for Peripheral Areas C and D.

Within the Paradise Valley Fan Terrace study area, channelization might find more economic justification in the southern parts of the watershed that are exposed to a higher sheetflow potential. Channel systems could eliminate, or substantially reduce, flooding over large areas inundated by sheetflow. Since the northern areas of the watershed are less prone to sheetflow, (i.e., more incised topography) they would appear to benefit less from channelization. However, if development in these northern areas is not confined to "high ground" that exists between washes, this assessment could change.

Detention/Retention Basins

Detention/retention basin concepts were evaluated to determine their effectiveness in reducing downstream peak discharge and the associated flooding potential. Due to the long, narrow shape of the majority of the watershed sub-basins, the downstream benefits from retention/detention basins

usually begin to diminish within 1.5 to 2.0 miles below the basin location. This shape of watershed tends to generate peak flow rates from runoff relivatively close to a measurement point, i.e., due to the long travel times, the downstream peaks are gone by the time the flood wave from the upper watershed arrives.

Achieving retention/detention goals through small on-site storage basins (scattered throughout the watershed) would probably be more cost effective and beneficial than attempting to provide the required storage capacity at large regional basins. The regional basin concept also tends to be somewhat impractical for the sheetflow environment that is characteristic of large portions of this study area, i.e., long diversion dikes or interceptor channels are required.

The lack of natural, deep valleys through the study area will also require that retention/detention storage capacity be achieved through "below ground" excavation. This type of storage can be expensive because of the 1 to 2 percent ground slopes in the study area that require substantial overburden excavation to reach level ground (see discussion Section 4.0 of this report).

Cost estimates were not developed for specific detention or retention sites. Instead, regression equations were developed to provide estimates of cost, right-of-way requirements, storage volumes, and excavation volumes. These equations provide a versatile tool for site-specific basin analyses anywhere within the study area.

In summary, large regional detention/retention basins are not considered practical for the study area. However, the concept of on-site retention is a feasible alternative for migrating the increase in runoff that will result from continued development of the watershed.

Tatum Ranch Drainage Plan

The Tatum Ranch "wash improvement and stabilization plan" was reviewed for possible application to other portions of the study area. This plan consists of artificial berming along natural washes and the excavation of interceptor channels to capture sheetflow and divert such flow into the bermed washes.

Such a plan has the potential to disrupt the natural equilibrium of the existing desert washes. This disruption will occur in response to the unnatural concentration of flows into the existing washes. Such concentration of flow also creates problems in disposing of the water at the downstream end of the bermed washes, i.e., directing concentrated flows onto adjacent properties. The visual impact of the bermed washes would also seem to project an unnatural "look" to the surrounding areas.

In summary, if the Tatum Ranch concept is to be considered for application to other regions of the study area, it should only be done after careful study has resolved any adverse impacts associated with the concentration of runoff into natural desert washes.

Desert Ridge Drainage Plan

The drainage system for Desert Ridge will require the construction of wide, shallow, man-made channels that will parallel the natural drainage pattern through the area. These designated drainage corridors will utilize landscaped wing dikes to intercept off-site sheetflow and divert such flows into the drainage corridors.

Unlike the Tatum Ranch concept, the Desert Ridge drainage corridors do not include raised berms along the edge of the channels. Accordingly, the interior sheetflow will have free access to the channels.

The drainage corridors will be excavated to convey all design flows in a completely depressed channel section. The channels are to be re-vegetated to a natural desert setting and will include provisions for a bike path. Channel outflows will be carried to the CAP detention dikes.

In summary, the Desert Ridge concept was found to be very appealing and would be recommended throughout the flat, southerly regions of the study area.

FEMA Floodplain Management Policies

Preliminary FEMA floodplain delineations have designated nearly all of the study area located south of Dixileta Road to be in an Alluvial Fan, Special Flood Hazard Area. Only major structural flood control measures will be considered by FEMA for removing a designated alluvial fan from a Special Flood Hazard Area. Accordingly, unless such structural measures are installed, all new development must not only protect against flood damage, but must also purchase flood insurance.

Compliance with FEMA floodplain management policies for this alluvial fan designated area will effectively mitigate the flood damage potential for future devleopment, but will do nothing to remove the flood damage potential to existing development.

"No Action" Alternative

The City of Phoenix has already adopted drainage design criteria that requires new development to retain all runoff from the 100-year, 2-hour storm. This criteria also requires that the peak discharge after development not exceed the peak discharge prior to development.

If these present drainage criteria are coupled with the FEMA policies for floodplain management on alluvial fans, the "No Action" alternative will be an effective option towards mitigating flood damage to future development. However, as with the FEMA Floodplain Management Option, this alternative will do nothing to eliminate potential flood damage to existing development.

Summary

The information presented in this study does not recommend a specific drainage plan that could immediately be sent before area residents or the City Council for adoption. The watershed drainage and development issues are far too complex to lend themselves to a simple, single solution within the manhour limitations alloted to this study. Instead, this study should be regarded as the first step towards the identification of drainage problems and the advantages and disadvantages of several drainage solutions that might have application to different regions of the watershed.

A substantial database has evolved from the technical analyses conducted as part of this study. Portions of this database have been used to develop engineering design aids for the preliminary analysis of retention basins and channels. A very comprehensive set of HEC-1 models have been created for predicting the runoff response at virtually any point within the study area. Numerous channel alignments and storm drain systems have been evaluated at varying levels of detail to provide estimates of effectiveness, size, and cost. This database can easily be accessed to provide guidance on the development of site-specific plans that may be proposed as part of future watershed urbanization.

This flexible approach is in concert with the 1987 General Plan for this area, which states: ".... more detailed planning for Areas C and D will be an ongoing process. The Plan must be monitored and refined regularly to ensure that it is kept current and responsive to changing conditions and community needs".

1.0 INTRODUCTION

This report documents the engineering analysis that was performed to develop drainage and flood control concepts for an approximate 56 square mile area identified as the Paradise Valley Fan Terrace. The study area, which includes portions of the City of Phoenix Peripheral Areas C and D, is part of a 107 square mile watershed that was modeled for this project.

The study area is located north of the Central Arizona Project (CAP) Aqueduct and includes that portion of the City of Phoenix located west of Scottsdale Road, south of the Carefree Highway, and east of Cave Creek. The project boundaries are illustrated in Figure 1.1.

A major product of this study was the development of a detailed hydrologic model (HEC-1) to determine the watershed rainfall/runoff response for both existing and future land-use conditions. This hydrologic data is used as the basis for evaluating several drainage and flood control concepts for the study area. The data is also used to develop graphical and mathematical relationships that can be used for quick analyses of both retention basin and channel size and costs.

Due to the large size of the study area, and the uncertainty as to how future site-specific development may occur, it was not considered practical to recommend a preferred flood control plan for the area. Such an approach would provide no flexibility for possible changes in future land-use projections, nor would it afford area land owners an opportunity for input on the type of drainage and flood control measures to be installed in the watershed. Instead, a study approach was pursued that provides regionalized technical design aides (graphs and equations) for retention basins and open channels, as well as a technical discussion of other floodplain management options that might be considered for use in the study area. This alternate study approach provides the City of Phoenix with a menu of options that can

used to customize a site-specific drainage plan for different regions of the study area. This versatility will provide City planners with the flexibility to easily respond to changing land-use projections and to offer area residents numerous options for floodplain management.

2.0 DRAINAGE BASIN CHARACTERISTICS

2.1 DRAINAGE AREA

The project study limits were previously illustrated in Figure 1.1. Both Plates 1 and 2 show the watershed boundaries associated with the study limits. As stated previously, the contributing watershed encompasses approximately 107 square miles. A substantial portion of the watershed lies within the City of Scottsdale.

The watershed, which drains from the northeast to the southwest, is characterized by literally hundreds of parallel and intermittently braided, ephemeral washes. The northeastern portion of the watershed (generally east of Scottsdale Road and north of Jomax Road) exhibits a somewhat rolling topography, which tends to minimize the potential for widespread sheet flow. However, the southwestern region of the watershed has much less topographic undulation and is prone to sheet flow during large flood events.

The majority of the watershed, located west to north of the McDowell Mountains, might best be defined as a piedmont plain. This region contains both a pediment and alluvial plain. Portions of this region have also been referred to as an alluvial fan terrace in a recent Soil Survey Report (Soil Conservation Service, 1986). 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 deposition.

In the author's opinion, there are no true alluvial fans in the project watershed. There are no mountain fronts in the study area from which an alluvial fan could emanate. The morphology of the region could best be described in terms of a land surface that exhibits a gradual increase in ground slope as one moves through the watershed from the southwest to the northeast. This gradual increase in slope is accomplished by more undulating

topography and a stream pattern that becomes less prone to sheetflow with increasing ground slope (i.e., higher elevations).

Pediments and alluvial plains are difficult to analyze from both a hydrologic and hydraulic perspective because of the absence of large capacity channels and the potential for sheetflow. Although the majority of the pediment surface has fairly well defined swales and washes, the alluvial plain (generally south of Dynamite Road) 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 would probably exhibit a wide, shallow sheetflow pattern.

North from Jomax Road to about Dixileta Road, the small washes on the alluvial plain begin to exhibit more definition and corresponding channel capacity. This region represents a transition area between the alluvial plain (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, low density, residential development, most notably in the Carefree area and south of Carefree along Cave Creek Road. New development will soon be underway at Tatum Ranch, which straddles Cave Creek Road between Lone Mountain Road and Dynamite Road.

This existing development was simulated in the HEC-1 model (for existing floodplain conditions) by inputting a percent of impervious cover that was based on an actual count of existing structures observed on a 1989 circa aerial photograph of the watershed. Future land-use conditions were also modeled by using an increase in the percent of impervious cover. This increase was correlated with both City of Phoenix and City of Scottsdale planning projections (dwelling units per acre) for the watershed. These land-use projections include residential (with densities ranging from 1/5 to 22 dwelling units per acre), tourist accommodations, low intensity resort, commercial, cultural/institutional, office, light industrial, and open space.

It is important to consider both existing and future land-use plans in the hydrologic modeling process because of the impact that such uses have on the runoff response of the watershed. An increase in the amount of impervious land cover will cause an increase in peak discharge rates and flooding potential.

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, April, 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 (i.e., 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 (i.e. very slow infiltration rate).

The composition of the project watershed, in terms of hydrologic soil groups, is presented on Plate 1. The information in this figure 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 3.2 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, etc. 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 desires of local residents and the City planning staffs of both Phoenix and Scottsdale.

2.4 EXISTING DRAINAGE IMPROVEMENTS

With the exception of Tatum Ranch, the low density of development that presently exists in the watershed has not been accompanied by any major flood control or drainage improvements. 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 may create some minor diversion of runoff during the more frequent floods, such as a 2-year event. However, 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 their natural drainage path. During field inspections of the watershed, it was also observed that 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 dikes are at the downstream limits of the study area and offer no protection to upstream areas. Cave Buttes Dam is also on the perimeter of the study area and offers no protection within the study area. However, the Cave Buttes auxiliary dike that crosses Cave Creek Road will intercept some flows that would have historically flowed southwest along the road.

The pending development of Tatum Ranch has produced several interceptor channels, bermed washes, and one large retention basin. These drainage improvements are discussed in detail in Section 6.3 of this report.

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 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, i.e., 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 to simulate the natural drainage pattern in the basin. Plate 2 presents an illustration of the drainage patterns, sub-basin boundaries, and concentration points used to model both existing and future runoff conditions. 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 publication entitled "HEC-1 Models for the Paradise Valley Fan Terrace, Concept Drainage Study."

The February 1, 1985 version of HEC-1 was used for all models presented in this study. Use of the June 1, 1988 version will yield different peak discharge results.

3.1 DELINEATION OF DRAINAGE SUB-BASINS

As stated previously, the project watershed consists of approximately 107 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 much smaller sub-basins of relatively homogeneous hydrologic characteristics. The number and size of sub-basins is also dictated by the number of locations at which hydrologic output data is desired, i.e., detention basin outlets, channel locations, etc.

In order to meet this criteria, the overall watershed was divided into 139 sub-basins (this number varied slightly to reflect differences in concentration points that were required when evaluating some of the concept drainage alternatives). Plate 2 illustrates the sub-basin delineations that were used to model both existing and future flooding conditions for the watershed.

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 lower half of the watershed. Northeast of this area, the desert washes are more 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 distributed uniformly 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.

3.2 INTERCEPTION/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 five days preceding the design storm.

Even though the project area is located in a desert environment that would more typically be represented by a drier antecedent moisture condition (AMC I), the use of AMC II is a prudent assumption to use for the hydrologic analysis and design of any drainage system plan or floodplain delineation.

A base curve number was developed for each of the four hydrologic soil groups (A,B,C, and D) under the assumption of 15% cover density and a "desert brush" vegetation community. Since this study utilizes large portions of the HEC-1 model previously developed for the General Drainage Plan For North Scottsdale, Arizona, WRA 1989, the same baseline curve numbers from the Scottsdale study were used for this study. As described in the Scottsdale study, curve number development was based on information contained in a City of Scottsdale drainage design manual, SCS Technical Release 55(June, 1986), and Runoff Curve Numbers for Semiarid Range and Forest Conditions, ASAE, Woodward, 1973. A summary of baseline curve numbers 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% HSG A (CN = 60)
2. 50% HSG A + 50% HSG B (CN = 67)
3. 100% HSG B (CN = 74)
4. 50% HSG B + 50% HSG C (CN = 78)
5. 100% HSG C (CN = 82)
6. 100% HSG D (CN = 86)

These six categories reflect adjustments made for the multiple HSG soil complex/associations discussed 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 (i.e., no development). These final baseline curve numbers were used in all the HEC-1 models created for this study. Any attempts to revise these models to investigate storm durations less than 24-hours, should employ the appropriate curve number in accordance with Table 3.1.

Modeling adjustments to reflect increased runoff due to urbanization (both existing and future) 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% and 72% impervious area values for commercial/business and industrial districts, respectively, were used without any adjustments within the Scottsdale portion of the watershed. Other land-use classifications within the Phoenix portion of the watershed include "Mixed Use" (85% impervious), "Commercial" (86% impervious), "Hillside" (5.4% impervious), and "Floodplain" and "Park" (1% impervious).

The percents of impervious area in TR-55 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% 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. These values are listed as part of the HEC-1 input data published in the separate document of project computer data.

Percent of impervious cover for existing land-use conditions was based on a visual count of every structure that was visible on a circa 1989 aerial photograph of the watershed. A transparency of the watershed sub-basin delineations (Plate 2) was superimposed over the photographs in order that the number of existing structures could be identified for each sub-basin. The total number of structures in each sub-basin was then divided by the total sub-basin area in order to arrive at an average number of dwelling units per acre. This average value was then used in conjunction with Figure 3.1 to determine the percent of impervious cover for each sub-basin.

3.3 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. Overland flow slopes were computed from the length and elevation measurements taken from the 1"=2000' USGS quadrangle maps. In a few instances two overland flow planes were input to the HEC-1 model to describe a sub-basin.

Overland flow data for areas located within the City of Scottsdale were not changed from those used in the 1989 North Scottsdale study (sub-basin series 1000). However, for the new Phoenix watershed area (sub-basin series 3000) added to the Scottsdale HEC-1 model, a slightly different approach was used. The Phoenix area was divided into three large sub-areas that exhibited relatively homogeneous overland flow characteristics. Overland flow lengths and slopes were measured for two of these three sub-areas in the same manner described above. An average length and slope was then computed for each sub-area and used for all sub-basins within that sub-area.

The third large sub-area encompasses the alluvial plain (or fan terrace) located in the southern half of the watershed. Overland flow lengths for this region were based on a 1"=1000', 1984 aerial photograph of the watershed. Sub-basins were 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 counted 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 (for the fan terrace) has been adopted from the SLA report, with no changes, for use in this study.

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' to 400') 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 1/4-inch 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 3.2.

3.4 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"=2000' USGS quadrangle maps. 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).

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. As part of the 1987 SLA study, Mr. Ward field-surveyed six typical washes on the fan terrace. Using Manning's Equation, the average bankfull capacity of a typical wash was found to be approximately

80 cfs. This information was combined with the peak discharge data from HEC-1 to develop the following procedure for the selection of channel geometry across the lower fan terrace.

1. Using the number of washes/rills counted for each fan terrace sub-basin, and the HEC-1 peak discharge through each sub-basin, determine the average discharge per wash and the depth and velocity associated with the discharge.

2. Compare the results from Step 1 to the previously computed average, bankfull capacity of 80 cfs per wash. If this 80 cfs capacity is exceeded, sheetflow can be expected.

3. Concurrently with Step 2, compute the velocity, Froude Number, and depth of flow resulting from the HEC-1 channel geometry carrying the total HEC-1 peak discharge through each sub-basin. If the computed wash capacity from Step 1 is approximately equal to, or less than 80 cfs (Step 2), flow can be expected to be contained within the small washes and the HEC-1 channel velocity should be approximately the same as the wash velocity computed from Step 1. If these two velocities are not approximately the same, the HEC-1 channel geometry is revised, the model re-run, and this procedure is repeated. The Froude Number for the HEC-1 channel is also examined as part of this step. These Froude Numbers should be close to critical flow since that is the predominant flow regime across alluvial fans.

4. If the wash-capacity comparison made in Step 2 indicates substantial overflow would occur, then the computed wash-velocity will be erroneously high, since the wash would not actually carry all the excess water, i.e., it would spread across the desert as sheetflow. Under this condition, the computed HEC-1 channel velocities should be lower than those accompanying the excessive concentration of flow in a small wash.

The acceptance of a HEC-1 channel velocity for this case is a matter of engineering judgement. Such judgement should be based on a review of the flow depth, velocity, and Froude Number accompanying the HEC-1 channel geometry. This data should be compared to what would be considered as realistic flow conditions across a wide swath of desert. In order to maintain proper timing of the flood-wave movement across the terrace, velocity might be given more weight than the other two parameters. Sheetflow velocities in the 3-7 fps range were considered realistic for this study, with the higher velocities being used where the sheetflow unit discharge was higher (i.e., on the steeper portions of the fan terrace).

Where necessary, the HEC-1 channel geometry was adjusted to provide a realistic range of flow velocities.

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 conditions) at Sub-Basin 27 is 9831 cfs, while approximately 4.5 miles downstream at SUB 29, it is only 6577 cfs. This attenuation was created by increasing the channel bottom width from 1500 feet in Sub-Basin 27 to 2500 feet in Sub-Basin 28, and to 3500 feet in Sub-Basin 29. This sensitivity justifies a careful examination of the channel hydraulics across the fan terrace. Substantial hydrograph attenuation should be anticipated to reflect transmission losses and overbank storage.

Nearly all the 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.

3.5 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, etc.), 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 3.3 summarizes the areally reduced rainfall depths that are representative of different portions of the watershed. The rainfall depths in Table 3.3 are based on a 24-hour storm duration and a 10-square mile areal reduction factor.

The rainfall values in Table 3.3 were distributed over a 24-hour duration using the SCS Type IIA rainfall distribution. This distribution is graphically depicted in Figure 3.2, along with three other distributions that were considered for possible use in this project.

The selection of the rainfall parameters used in this study was based on an extensive sensitivity analysis conducted for the previously referenced 1989 North Scottsdale drainage study. Details of this analysis are discussed in Section 3.6 of this report.

3.6 SENSITIVITY ANALYSIS AND MODEL VERIFICATION

In order to generate a level of confidence in the HEC-1 modeling results, and to examine the sensitivity of certain rainfall parameters on the runoff response of the watershed, additional HEC-1 modeling and independent hydrologic calculations were performed.

The rainfall sensitivity analysis was conducted as part of the North Scottsdale study and was confined to an approximate 7 square mile block of drainage area containing 13 sub-basins (SUB 2240 through SUB 2340). An expansion of the original HEC-1 model verification analysis also utilized several sub-basins within the City of Phoenix boundaries. The results of these analyses are discussed in the following two subsections of this report.

3.6.1 Rainfall Sensitivity Analysis

Using the previously described HEC-1 model, and the kinematic wave option for both overland flow and channel routing, the following rainfall parameters were investigated:

1. Rainfall Amount - Two sources of rainfall data were used: 1) NOAA Atlas 2, Volume VIII, Arizona; and 2) a combination of NOAA Atlas data and City of Phoenix rainfall data (Storm Drain Design Manual, September, 1985), whichever gave the higher value for a given frequency-duration.

2. Spatial Distribution - The rainfall was assumed to be evenly distributed over the entire contributing watershed. However, the sensitivity of peak discharge was examined as a function of changing the "areal reduction factor" from 10 square miles to 50 square miles.

3. Temporal Distribution - The following rainfall distributions were evaluated: 1) HEC-1 hypothetical, 24-hour; 2) HEC-1 hypothetical, 1-hour; 3) SCS Type II, 24-hour; 4) SCS Type IIA, 24-hour; 5) City of Phoenix, 24-hour.

Where possible, several combinations of these parameters were examined in order to establish the maximum and minimum values that might occur in peak discharge. However, some combinations were not possible. For example, the SCS Type II, IIA, and City of Phoenix distributions only utilize the 24-hour rainfall value. Since the NOAA values were higher than the Phoenix rainfall values for this duration, only the NOAA data was used for these storms.

It should be noted that there is also some incompatibility in comparing run-off response due to changes in areal reduction factors. The HEC-1 program incorporates an internal subroutine that automatically adjusts all rainfall values (on the PH card) for a given storm duration when using the hypothetical rainfall distribution. This adjustment is initiated by the user inputting a desired drainage area size to be used by the program in computing the areal reduction factor - (ARF). In contrast, the two SCS distributions and the City of Phoenix distribution are adjusted as a result of the user modifying only the 24-hour rainfall for the desired areal reduction.

Another possible incompatibility in computing the ARF arises from the fact that the HEC-1 algorithm (see Equation 3.12 in the HEC-1 User's Manual, September, 1981) produces different results from that obtained from Figure 14 in the NOAA Atlas. Both methods produce nearly identical results for the 24-hour duration, but slightly different values for the shorter durations. For example, HEC-1 will generate an ARF of 0.815 for a 1-hour duration at 50 square miles, while NOAA produces an ARF of 0.800 for the same conditions. It should be noted, however, that these small discrepancies may be due to errors in visually reading such values from graphs.

The results of the rainfall sensitivity analysis are presented in tabular form in Table 3.4. It should be noted that the sub-basins listed in Table 3.4 are part of the 1989 North Scottsdale drainage study.

During a review of the sensitivity analysis results, some additional investigation was performed relative to recent research on areal reduction factors and accuracy of the rainfall data published in the NOAA Atlas. Using gage data from Walnut Gulch (AZ) and other portions of Arizona and New Mexico, Zehr and Myers (1984) concluded that "reductions of point rainfalls for area-size in the semi-arid Southwest are greater than previously published nationwide average depth-area curves". This research indicates that ARFs from NOAA do <u>not</u> reduce point rainfall values as much as new data indicate they could be reduced. In other words, ARFs from NOAA will tend to produce <u>higher</u> areal rainfall (and corresponding peak discharge) than the new data in NWS HYDRO-40 suggests is realistic for areas of the southwest.

To test the sensitivity of these factors, the Type IIA, 24-hour storm was run with ARFs for 10 square miles and 50 square miles, using the data from both NOAA and NWS HYDRO-40. For each of the two drainage area sizes, the average Q_{100} ratios (using ARFs from the two referenced sources) for 15 concentration points in the test basins were compared in order to quantify the change in runoff response. The resulting ratios were found to be as follows:

$$\frac{Q_{100},\ 10\ \text{sq. mi. ARF, NOAA}}{Q_{100},\ 10\ \text{sq. mi. ARF, HYDRO-40}} = 1.10$$

$$\frac{Q_{100},\ 50\ \text{sq. mi. ARF, NOAA}}{Q_{100},\ 50\ \text{sq. mi. ARF, HYDRO-40}} = 1.17$$

As expected, the NOAA Atlas values produce 10% to 17% higher runoff values than the new HYDRO-40 data.

A second research article deals with the accuracy of short-duration rainfall in the NOAA Atlas. Again, using gage data from Walnut Gulch, Arizona, Osborn and Renard (1988) demonstrate that the Walnut Gulch data produces substantially higher rainfall values from short-duration rainfall (1-hour and less) for infrequent events such as the 50- and 100-year storms. For example, the rainfall depths for the 100-year event were approximately 27% greater than the data provided by the NOAA Atlas for the same watershed. Using a kinematic cascade rainfall-runoff model (KINEROS), Osborn and Renard demonstrate that such changes in rainfall create changes in peak discharge and runoff volume on the order of 31% to 200% and 37% to 200%, respectively.

Although the Osborn/Renard report only addresses the rainfall data for Walnut Gulch, one might assume (for the purpose of a sensitivity analysis) that the same relative increases in short-duration rainfall might apply to other regions in Arizona that are exposed to short-duration, convective thunderstorms. Accordingly, the 1-hour, 15-minute, and 5-minute rainfall values for the HEC-1 hypothetical 100-year , 1-hour storm were increased by factors of 1.29, 1.25, and 1.20, respectively, to match the increases calculated by Osborn and Renard for these duration-frequency combinations.

When input to the HEC-1 model, these new rainfall values caused an average increase of 59% in Q_{100} at the 15 concentration points used in the sensitivity analysis. This data was based on a 100-year, 1-hour storm. The analysis was not extended to longer duration events because the Osborn/ Renard data did not go beyond a 1-hour storm. Figure 3.3 graphically depicts the variation in peak discharge for a 1-hour, HEC-1 hypothetical storm when using NOAA rainfall versus the Osborn/Renard data.

Based on the results of this analysis, the following recommendations were made relative to rainfall parameters for use in the General Drainage Plan for the North Scottsdale Area and for this study of the Paradise Valley Fan Terrace:

1. Rainfall Amount - Use NOAA Atlas 2, Volume VIII, Arizona, with separate rainfall calculations for the northern, central, and southern portions of the watershed.

The higher City of Phoenix rainfall data is not recommended for combination with the NOAA data for the following reasons:

a) The City's Storm Drain Design Manual indicates the rainfall is based on WTBM-44, which is believed to be the same data as the NOAA Atlas. Most of the differences with the published Phoenix rainfall data and the NOAA data generated for this study area, occur for durations of 1-hour or less. All rainfall values for less than a 1-hour duration are based on a percentage of the 1-hour value. Accordingly, if the 1-hour value is in error, all lesser duration values will be in error. An analysis of the published Phoenix 1-hour rainfall values indicate that they are slightly in error when compared to the 1-hour rainfall values that result from using the statistical equations presented on page 15 of the NOAA Atlas. This, of course, causes all lesser duration values to also be in error.

b) If the Phoenix numbers are not based on the NOAA Atlas, there would arise a problem of technically justifying a valid basis for combining data from two different sources. If a combination of data were to be made that would cause an increase in runoff (beyond the NOAA data), a valid technical basis would be warranted for such a combination in order to defend the data against potential future critics.

The Osborn/Renard research at Walnut Gulch is considered too limited to be applied on a generalized statewide basis at this time. As discussed previously, their research only addressed storm durations up to 1-hour. Accordingly, there is no guidance provided on possible errors associated with longer duration events, such as the 24-hour storm.

2. Spatial Distribution - Recommend that rainfall be evenly distributed over the entire contributing watershed. Point rainfall values should be reduced to simulate a storm size of 10 square miles. The areal reduction factor (ARF) should be taken from the NOAA Atlas, Figure 14. When compared to data in the Zehr/Myer article, the use of Figure 14 (NOAA) will provide a degree of conservatism for runoff calculations.

   The lower ARFs suggested in the Zehr/Myer report are not recommended because of their admission of considerable variation in the uncertainty of the results (on a statewide basis) due to sparse data in certain areas of the state.

   Adoption of the ARFs from the NOAA Atlas will also, to some extent, counteract the effects of the suggestion by Osborn and Renard that the NOAA rainfall data may be lower than that which actually occurs in various regions of Arizona.

3. Temporal Distribution - Recommend adoption of the 24-hour, SCS Type IIA storm. The 24-hour duration will provide suitable runoff volumes for detention/retention basin design while the Type IIA distribution incorporates a short burst of high intensity rainfall to simulate convective thunderstorm characteristics that are common to the project area. This intensity will provide the high peak discharges that are recommended for the design of any channel improvements. This distribution was reportedly developed on the basis of thunderstorm data collected from the Arizona - New Mexico area.

The HEC-1 hypothetical distribution was rejected because of a tendency to generate peak discharge values that are significantly higher than discharges obtained with other distributions and independent peak discharge calculations. This distribution makes no attempt to acknowledge regional rainfall

characteristics. The hypothetical distribution in HEC-1 also exhibits large variations in peak discharge when adjustments are made to reduce point rainfall as a function of the assumed areal extent of a given storm. For example, when adjusting the ARF from 10 square miles to 50 square miles, the HEC-1 hypothetical storm revealed an average decrease of 25% in peak discharge values, while under the same circumstances, the Type IIA storm only caused an average reduction of about 7% in peak discharge. As discussed previously, these differences are due to the way in which the ARF is applied to the rainfall values for the two storms.

It is believed that use of the HEC-1 ARF algorithm might lead future users of the HEC-1 models created for this study to unwittingly make adjustments for areal distribution of rainfall and end up with large variations in the peak discharges used for channel improvements in different parts of the watershed. As noted previously, changes in the NOAA ARF, with subsequent application to the Type IIA storm, has much less influence on peak discharge.

The City of Phoenix rainfall distribution was rejected on the basis that it provides substantially lower rainfall intensities, during the 1-hour burst of rainfall in the middle of the storm, than the Type II, IIA, and HEC-1 hypothetical distributions. For example, maximum rainfall intensities for these distributions (based on a 24-hour rainfall depth of 4.23" and a 5-minute computation interval) were found to be 2.76, 4.68, 4.68, and 8.04 in/hr for the City of Phoenix, Type II, Type IIA, and HEC-1 hypothetical distributions, respectively. The low intensity associated with the Phoenix distribution translates into a significant reduction in peak discharge (see Table 3.4).

3.6.2 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, i.e., 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 modeling used for this project. These procedures are listed as follows:

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 2, 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 study 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 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.

Procedure 3 (TR-55) is based on an SCS Type II rainfall distribution and uses a time of concentration that evaluates sheet flow, 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 three hours. Since this procedure is based on short duration storms, all SCS curve numbers taken from the HEC-1 model were converted to 1-hour values using Woodward's curves.

The large size of the project watershed dictated that only a small "test" section be selected for the model calibration/verification process. Accordingly, six sub-basins (varying in size from 0.04 square miles to 5.87 square miles) were selected for analysis by these four procedures.

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 3.4 through 3.9 graphically illustrate the data presented in Table 3.5.

When interpreting the results in Figures 3.4 through 3.9, 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 <u>average</u> 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 sheet

flow 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 3.4) - This sub-basin has both average soil infiltration characteristics and shape factor (7.11). Peak discharge correlation between HEC-1 and the independent procedures is good.

- CP 3135 (Figure 3.5) - This drainage area has relatively pervious soils (almost 100% Hydrologic Soil Group B). This is suspected as the primary reason why HEC-1, Pima County, and TR-55 (all curve number dependent) are noticeably lower than the regression equations, which are not curve number dependent. This sub-basin also exceeds the minimum drainage area size of 0.09 square miles associated with the Roeske equation.

  Given these facts, the correlation between HEC-1, Pima County, and TR-55 is considered good. The regression equations would undoubtedly provide better correlation if they were curve number dependent.

- CP 3150 (Figure 3.6) - This is a very long and narrow drainage area. The shape factor of 25.67 exceeds the maximum shape factor of 20.6 associated with the USGS (Eychaner) regression equation. The basin also has somewhat more pervious soils than average. The

shape factor is considered the primary reason why the two regression equations compare so poorly with the other three methods. A long, narrow basin produces long travel times which prevent peak runoff from upstream areas from combining with peak runoff from downstream areas.

HEC-1, Pima County, and TR-55 include input data to simulate this phenomenum. Accordingly, these three methods produce relatively good agreement.

- CP 3160 (Figure 3.7) - This sub-basin has both average infiltration characteristics and shape factor (7.26). As a result, there is good correlation among the procedures.

- CP 3400 (Figure 3.8) - This drainage area has very pervious soils (between HSG A and B) and is also very long and narrow (shape factor = 19.63, which is at the upper limit for the USGS Eychaner equation). The pervious soils are considered the main reason for the poor correlation with the USGS Eychaner equation, while both the soils and shape factor explain the poor correlation with the USGS Roeske equation. The HEC-1 results compare very well with Pima County and TR-55.

- CP 3490 (Figure 3.9) - This basin is also very long and narrow (shape factor = 34.37, which exceeds the upper limit for the USGS Eychaner equation), but has average soil infiltration characteristics. This drainage area seems to contradict the previous trend that showed the regression equations as over-predicting peaks when extremely large shape factors were involved. It may be that the

average soil infiltration characteristics are dominating the response in this basin, as far as the regression equations are concerned. With the exception of the Pima County procedure, the results of these calculations show good correlation with HEC-1. It should be noted that the time of concentration for this basin exceeds the maximum 3-hour limit defined for the Pima County procedure.

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

As a final step in the verification process, 17 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 3.10 and 3.11. Figure 3.10 utilizes sub-basins that lie within the northern half of the watershed where sheetflow is less prone to occur. Figure 3.11 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, along with a regression line fitted to the HEC-1 data points.

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, etc.) 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 3.10 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% to 100% 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 3.10 illustrates. At the other end of the spectrum, the two data points that plot significantly below the USGS line are composed nearly 100% 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 <u>above</u> the USGS regression line in Figure 3.10, it should be recalled from Section 3.2 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, i.e., 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 3.11 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% 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 3.10 and 3.11 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 107 square mile watershed, it can be justifiably concluded that the complete watershed model is providing output data that is reasonably representative of the rainfall/runoff response of the project study area.

4.0 DESIGN GUIDELINES FOR RETENTION BASIN ANALYSIS

Due to uncertainties in future land development patterns, it was not considered prudent for this conceptual study to recommend specific retention basin sites. If large regional retention basins are ultimately utilized in this study area, their location should be based on input from major land owners/developers, local residents, and the City of Phoenix.

However, as a result of this study, a very useful planning tool has been developed in the form of regression equations that provide quick answers to questions regarding retention basin storage requirements, right-of-way requirements, excavation volume, and construction costs.

Regression equations were used in lieu of graphs because of the numerous independent variables that have a significant influence on retention basin design. Three to five independent variables were required in the regression analysis to produce a standard error of estimate that was consistently less than five percent. These variables include:

1) ground slope at the basin location;

2) drainage area contributing runoff to the basin;

3) SCS curve number for the drainage area contributing runoff to the basin;

4) rainfall amount being used for the basin design; and

5) unit excavation cost for estimating basin construction cost.

Inclusion of this many variables in a graphical format would have required literally hundreds of graphs to adequately encompass the variation in the family of variables.

The equations developed from this regression analysis can be used to predict the following four parameters related to retention basin design:

1) required retention storage volume;

2) surface area required for retention basin construction;

3) excavation cost required for retention basin construction; and

4) excavation volume required for retention basin construction.

Table 4.1 presents the regression equations, along with a definition of each variable and the standard error of estimate associated with each equation. One standard error indicates that approximately 68% of the actual values (sample size) used in the regression are within the listed percentage of the value predicted by the regression equation. Two standard errors of estimate would envelope about 95% of the actual values used in the regression.

Twenty-two data sets were used to develop each regression equation. Logarithms of each variable were used to develop the multiple variable, power equations presented in Table 4.1. A complete listing of the regression data is provided in Appendix A.

Caution should be used in applying the regression equations beyond the extreme values listed in Table 4.2. The prediction error associated with the application of these equations beyond these limits is unknown. The data sets

used in the regression analysis were taken from concentration points lying within the watershed boundaries for the Paradise Valley Fan Terrace. Accordingly, the equations should enjoy wide application within these boundaries.

The limits (Table 4.2) associated with the rainfall variable were selected on the basis of providing design data for either the 100-year, 2-hour storm or the 100-year, 24-hour storm. The 2-hour storm duration was used to provide compatibility with retention criteria in the City's Storm Drain Design Manual - Subdivision Drainage Design, September, 1985. The 24-hour event was included to provide compatibility with the design storm specified for this concept drainage study.

Several important assumptions are incorporated into the development of the regression equations. The user should be familiar with these assumptions before applying the equations to a specific site. These assumptions are summarized as follows:

1) Retention basin side-slopes are 5H:1V.

2) Retention basin topwidth dimensions are based on a square basin shape projected onto a sloping ground surface. The surface area computed from these topwidth dimensions reflects 1 foot of freeboard above the design water surface depth and 5H:1V side-slopes.

3) One hundred percent (100%) of the required retention storage is provided below natural ground, i.e., no artificial levees or embankments are constructed to create retention storage.

4) The required storage volume is based on retaining 100 percent of the design event, i.e. no outflows from the basin are considered during the inflow period. Should a basin be designed with some type of regulated outflow system that would operate during the inflow period, the assumption of total retention will yield conservative basin design parameters.

5) None of the regression equations include any allowance for sediment storage.

6) All runoff calculations are based on land-use conditions that existed circa January 1989.

7) Regression equations were developed for minimum retention basin depths of 4 ft., 7 ft., and 10 ft. These minimum depths are referenced to natural ground elevation at the downstream end of the basin, and include 1 foot of freeboard over the actual water depth that would occur when the basin reaches its required storage volume capacity.

8) Due to the relatively steep slopes (1% - 2%) in the study area, there is considerable overburden excavation that must be removed before any storage volume can begin to be achieved. This overburden prism is accounted for by inclusion of the slope term in the equations for surface area, excavation cost, and excavation volume.

9) Excavation costs are varied as a function of the amount of excavation required. Excavation costs in the regression equations adhere to the following schedule:

1. < 10,000 c.y. -$5/c.y.

2. 10,001 c.y. to 100,000 c.y. -$4/c.y.

3. 101,000 c.y. to 500,000 c.y. -$3/c.y.

4. > 500,000 c.y. -$2/c.y.

This variation in unit excavation cost is reflected in the regression equations for basin excavation cost. Deviation from the above schedule of unit costs will cause a decrease in the accuracy of the excavation cost equations. If the user prefers a different unit excavation cost schedule, the basin excavation volume can be computed with the appropriate regression equation. This computed volume can then be applied to the desired unit excavation cost to get a total revised basin excavation cost.

10) The basin excavation cost equations only reflect excavation costs. They do not include any allowances for right-of-way acquisition, landscaping, outlet systems, or engineering design fees, contingencies, etc.. Right-of-way and landscaping costs can easily be estimated by using the appropriate regression equation to compute the required basin surface area and then applying the desired cost per acre to that area.

An approximation of an outlet system cost can be achieved by use of the following equation:

$$\text{Outlet Cost} = (HW) + (PC)\left(\frac{H(zS+1)}{S-0.0050}\right)$$

where: Outlet Cost = cost of underground pipe outlet (including inlet and outlet headwalls) extended on a slope of .0050 ft/ft from the retention basin invert to the intersection point of the pipe slope with the natural downstream land slope.

HW = cost of inlet headwall and outlet headwall.

PC = installed pipe cost per lineal foot.

H = depth below natural ground to pipe inlet (feet).

z = horizontal component of retention basin side-slope.

S = slope of natural ground along the pipe alignment (ft/ft).

Figure 4.1 presents a graphical illustration of the outlet system components.

11) When using the regression equations, it is important to remember that the curve number and precipitation variables should be representative of the entire contributing drainage area. This can

best be accomplished by computing an area-weighted average for these two parameters. Such a procedure will require that the drainage area be subdivided into smaller, homogeneous sub-basins. Plate 1 provides soils data and rainfall zones that can be used with this procedure.

5.0 DESIGN GUIDELINES FOR CHANNELIZATION ANALYSIS

From an engineering prospective, flood control channels are a viable drainage concept for any location within the study area. Although a detailed channelization plan is not being proposed for the study area (for the same reasons previously discussed for retention basins in Section 4.0), several channel alignments were evaluated to determine peak discharge requirements at various concentration points within the study area. These alignments are discussed in detail in Sections 6.1 and 6.4 of this report.

Section 5.0 will focus on the development of cost estimating guidelines for various channel cross-sections, cutoff walls, drop structures, bridges, and low-flow culverts.

5.1 LINED AND UNLINED CHANNELS

The purpose of the sub-section is to present channel design aides that can be used for quick analyses of channel excavation and lining costs and right-of-way requirements. These design aides are presented in the form of graphs relating channel discharge to both cost per lineal foot of channel length and to required right-of-way width. Channel slope is also included on the graphs as an independent variable.

Design curves are enclosed in Appendix B for the following channel cross-sections:

1) Fully lined concrete section (see Figure 5.1 for cross-section);

2) Partially lined section with soil-cement banks and earth bottom (see Figure 5.2 for cross-section); and

3) Completely unlined section with earth banks and bottom (see Figure 5.3 for cross-section).

An attempt was made to develop a second set of curves relating drainage area size to unit channel cost and right-of-way width for the 100-year, 24-hour storm. Unfortunately, there were too many other watershed variables (besides drainage area) that significantly influence peak discharge. Accordingly, this single variable relationship did not consistently produce accurate results. The use of regression equations (similar to those previously developed for retention basin analysis) was briefly considered but subsequently discarded because of the difficulty required to regress the many channel routing variables (bottomwidth, side-slope, roughness, bed-slope).

Prior to using any of the channel design curves, the user should be aware of the basic assumptions that were used to develop the curves. These assumptions are listed as follows:

1) All curves are based on the channel geometries illustrated in Figures 5.1, 5.2, and 5.3.

2) Concrete channels use an "n" value of 0.020, partially lined soil-cement channels use n = 0.025 and 0.035 (separate curves are provided for each value), and earth-lined sections use n = 0.050.

3) The soil-cement section incorporates a constant bank-lining toe-down (scour) depth of 5.0 feet.

4) All channel excavation costs are based on $3.0/cubic yard; in-place soil-cement is based on $24.00/cubic yard, and in-place concrete is priced at $180.00/cubic yard.

5) All channel design curves are based an "normal depth" assumptions, using Mannings Equation.

Two sets of design curves are provided in Appendix B. One set encompasses peak discharge values from 0 to 7,000 cfs, while the second set uses discharges from 7,000 to 14,000 cfs.

5.2 CHANNEL CUT-OFF WALLS

The channel construction cost curves presented in Section 5.1 do not include a cut-off wall along the top of the channel bank, i.e., the bank lining is simply extended to the top of the bank slope and terminated. Most channels will be designed with the intent of intercepting overland flows at any location along the channel bank-line. These lateral inflows may cause some erosion along the top of the bank. Such erosion may begin to create small rills along (parallel to) the top of the bank. These rills are capable of intercepting and concentrating small flows. These flows may cut sufficiently deep into the fill supporting the bank protection that structural problems may result.

To prevent this type of damage, cutoff walls are recommended along the top of the bank lining. These walls, which are illustrated in Figure 5.4, should be angled away from the top of the bank so that any downcutting along the top of the bank will not expose a vertical surface, which would tend to create a scour profile similar to that at a bridge pier. The angled wall surface will deflect the velocity vector up and over the top of the bank, rather than down under the bank lining.

The vertical depth of a cut-off wall is a decision left to the design engineer. Different sites may require different depths. However, a minimum cut-off wall depth of 2 feet would seem prudent.

Table 5.1 presents installation costs/L.F. of cut-off wall. These costs, which are presented for both concrete and soil-cement linings, are a function of cut-off wall depth and are based on the cut-off wall geometry illustrated in Figure 5.4.

5.3 DROP STRUCTURES

The use of completely earth-lined channel sections will always be accompanied by design criteria that specifies some maximum allowable (non-erodible) velocity. Even lined channel sections may have some maximum design velocity (or Froude Number) so as to confine all flows to a subcritical regime.

More often than not, channel velocity limits can only be achieved through an artificial reduction in channel bed-slope. Such slope reductions can be created with the use of drop structures.

Figure 5.5 presents conceptual cross-sectional geometry for a soil-cement drop structure. The scour depth associated with the plunge pool on the downstream side of the drop was estimated with the Veronese equation for vertical drops:

$$d_s = 1.32 H_T^{0.225} q^{0.54}$$

Where: d_s = maximum depth of scour below tailwater level (feet)

H_T = hydraulic head from the reservoir level to the tailwater level (feet) (for channel analysis, use the difference between the upstream and downstream flow depth, which is equivalent to the drop height).

= unit discharge (cfs/ft) in channel.

Both d_s and HT are dimensioned in Figure 5.5.

Figure 5.6 presents a series of curves that can be used to estimate the cost per lineal foot for the drop structure geometry illustrated in Figure 5.5. The user is cautioned to observe the notes on Figure 5.6 relative to flow depth, freeboard, side-slopes, and soil-cement costs.

Flows will accelerate towards critical velocity as they approach a drop structure. In order to prevent bank erosion in the vicinity upstream of the drop, channel bank-lining will be required. This bank-lining will also be required for some distance downstream of the drop to prevent erosion from the turbulent flow occurring in the plunge pool.

Figure 5.7 presents a series of curves that can be used to estimate bank-lining costs for each drop structure. These bank-lining costs are based on soil-cement being extended 58 feet upstream from the brink of the drop structure and 53 feet downstream from the brink of the drop structure. A constant toedown (scour) depth of 5 feet is included in the 58 foot upstream length of lining. For the 53 feet of bank lining extended downstream of the drop, the toedown dimension for the bank lining is based on y_s, as defined in Figure 5.5.

The lining costs presented in Figure 5.7 include both sides of the channel and are based on the same flow depth, freeboard and side-slopes used for the drop structure costs presented in Figure 5.6. The unit cost for soil-cement bank lining was reduced from that used for drop structures because of less difficulty in placing bank lining material versus drop structure material. The bank-lining costs in Figure 5.7 do not include provisions for a top-of-bank cut-off wall. Accordingly, should cut-off walls be desired, the engineer should refer to Table 5.1 for estimating such costs.

5.4 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 roadways. The cost of such bridges should be included in the overall cost for channelization.

Perhaps the most convenient method to estimate bridge cost is on the basis of a unit cost per square foot of deck area. This unit cost can be combined with the desired bridge width and channel geometry to produce the following equation for the total estimated bridge cost:

$$Cost = [(BW) + 2zD] * W * (UC)$$

where:
- Cost = total bridge cost (dollars)
- BW = channel bottomwidth (ft)
- Z = horizontal component of channel side-slope
- D = depth of channel (ft)
- W = bridge width (ft)
- UC = cost of bridge per unit deck area, i.e., dollars/square foot of deck area

5.5 LOW-FLOW CULVERTS

Depending upon directional alignment, the construction of man-made channels within the study area will have a high probability of intercepting the flow in natural washes. In order to support the natural vegetation community along these washes, it is recommended that man-made channels include low-flow outlets that will continue to feed water to the downstream remnants of these natural washes.

Figure 5.8 illustrates the intersection of a natural wash with a man-made channel, and shows the culvert length that would be required to supply water from the man-made channel to the downstream remnant of the natural wash. The following equation can be used to estimate the cost of each culvert installation required along the channel alignment

$$\text{Culvert Cost} = (HW) + (PC)\left(\frac{H(zS+1)}{S-0.0050}\right)$$

where: Culvert Cost = the total in-place cost of the pipe, with concrete headwalls at both inlet and outlet.

HW = cost of inlet headwall and outlet headwall.

PC = installed pipe cost per lineal foot.

H = depth (feet) from the invert of the man-made channel to the intersection point with the invert of the natural wash and the downstream side-slope of the man-made channel (see Figure 5.8)

z = horizontal component of side-slope of man-made channel

S = bed-slope ft/ft) of the natural wash being intercepted.

If the pipe is placed on a slope other than 0.0050 ft/ft, the actual pipe slope can be substituted in-place of the 0.0050 value in the preceding equation. The user should note that the slope of the intercepted wash (S) must be greater than the slope of the pipe. Otherwise this equation will yield <u>negative</u> values for the total pipe cost.

6.0 DISCUSSION OF FLOODPLAIN MANAGEMENT OPTIONS

Several options are available to the City of Phoenix for managing drainage problems within the Paradise Valley Fan Terrace study area. In addition to conventional structural solutions, such as channelization and retention basin construction, the City of Phoenix requested that this report include a discussion of drainage concepts previously proposed by other consultants for Tatum Ranch and Desert Ridge, as well as a discussion of the effectiveness of the FEMA Flood Insurance Study for this region, and a "no action" alternative.

A detailed discussion of each of these options is provided in the following sub-sections.

6.1 CHANNELIZATION

As previously stated in section 5.0 of this report, channelization is a viable alternative for any location within the watershed. Channels provide a very efficient mechanism for intercepting flood waters and safely conveying such water to a suitable outlet point.

However, if not properly designed, channels may be prone to bank erosion, scour problems, or even loss of capacity due to sediment deposition. Channel construction costs can also be expensive, especially when consideration is given to the need for bridge crossings that are required to restore vehicular access to areas severed by the channel alignment. Should channels be pursued as a public works project, they should be justified by a benefit:cost analysis.

It should be emphasized that all the channel concepts that were evaluated for this study would operate as interceptor channels, and in most cases, be aligned alont the upstream side of an existing or proposed roadway. In no instances were the interceptor channels aligned along an existing wash, i.e. there would be no excavation or enlargement of natural desert washes.

Interceptor channels operate on the concept of crossing a washershed at some angle to the natural drainage path. This intersection angle allows the channel to "intercept" sheetflows, as well as flows from natural washes, in order to prevent such flows from continuing along their natural drainage path and causing potential flood damage to downstream areas. The interceptor channels divert these flows to a suitable outfall where no flood damage will occur.

It is recognized that the man-made diversion of runoff from natural desert washes will adversely impact the native vegetation along such washes, i.e. the vegetation will be deprived of its natural source of water. To mitigate this problem, it is recommended that all interceptor channels be designed with low-flow outlets (culverts) at their intersection with existing washes (see Figure 5.8). These low-flow outlets will allow controlled rates of runoff to be distributed through the natural drainage network. However, as part of a flood control system, the low-flow outlets would be regulated so as not to release more water than could be contained within the bankfull capacity of the natural washes. More detailed information on the design of low-flow outlets is presented in Section 5.5 and 6.4.2.1 of this report.

An interceptor channel system based on these concepts should be in concert with the goals and policies outlined in the 1987 General Plan for Peripheral Areas C and D. This plan cited a policy of "preserving and protecting desert washes" while simultaneously recognizing the need for some form of drainage system to channel, collect, convey, and redistribute floodwaters.

Although a hydrologic model (HEC-1) was not created to analyze the required channel capacity for an integrated system of channels throughout the study area, Plate 3 illustrates a network of interceptor channels that could be constructed along the major roadway grid forecast for this area as part of the General Plan, Peripheral Areas C and D, City of Phoenix Planning Department, October, 1987.

The channel schematic shown on Plate 3 is very conceptual. Future planning efforts for this area may wish to consider a channel density that is either greater or less than that illustrated on Plate 3. However, any alterations to this channel schematic should pay special attention to possible diversions of flow from their natural outfall location. Watershed boundaries for the Cave Buttes Dam auxiliary dike and the CAP dikes are shown on Plate 3 in order to provide guidance for possible channel alignments. Flows should not be diverted across these boundaries.

During the early phases of this study, detailed HEC-1 models were created in an attempt to generate more detailed data on required channel capacities along specific alignments. These alignments include a major interceptor channel along the east side of Cave Creek Road, and smaller channels located along the north side of Lone Mountain Road, Dixileta Road, and Dynamite Road. A brief discussion of these alignments is presented in the following sub-sections. All analyses were based on a 100-year, 24-hour storm.

Section 6.4.2 of this report presents additional channel alignments that were analyzed along the northern and eastern periphery of the proposed Desert Ridge development.

6.1.1 Cave Creek Road Channel

As a major roadway through the watershed, Cave Creek Road draws immediate attention as a candidate for a parallel interceptor channel. A channel along the east side of this roadway would provide all-weather

access on Cave Creek Road, as well as creating a substantial reduction in peak discharge to those areas located west of the roadway.

Plate 4 depicts the probable alignment of this channel as extending from the Carefree Highway to the Cave Buttes Dam auxiliary dike located roughly halfway between Jomax Road and Happy Valley Road. The channel would outlet through a bridged crossing of Cave Creek Road and discharge to the Cave Buttes Dam reservoir area. By terminating the channel at the auxiliary dike, no additional drainage area is diverted to Cave Buttes Dam beyond that which naturally drains to this flood control reservoir.

This channel alignment was evaluated for both existing land-use conditions and future land-use conditions. Table 6.1 summarizes peak flow rates in the channel for both land-use conditions. Table 6.2 presents the reduction in downstream peak flow rates that are estimated to result from channel construction. The peak discharge values in both of these tables reflect the existence of some small flows being outlet to downstream washes at periodic intervals along the channel alignment. Concentration points are shown on Plate 4.

6.1.2 Lone Mountain, Dixileta & Dynamite Road Channels

A network of three channels was evaluated along the north side of Lone Mountain, Dixileta, and Dynamite Roads. The alignment of each channel is shown on Plate 5. The purpose of this analysis was threefold:

1) identify the peak 100-year discharge that would have to be conveyed by such channels;

2) determine the approximate size of channels required to convey the 100-year event; and

3) evaluate the reduction in peak discharge that would occur to areas downstream of such channels.

HEC-1 models were created for both existing and future land-use conditions. Table 6.3 summarizes peak flow rates in each channel (existing and future land-use conditions) for the 100-year, 24-hour event. Table 6.3 also presents approximate channel geometry, flow depths and velocities that would be expected if the channels were constructed on existing land-slopes.

Table 6.4 shows the reduction in peak discharge that would occur through selected areas downstream of the interceptor channels. The "with channel" discharges in Table 6.4 do not include any low-flow releases from the man-made channels into existing washes that are intercepted by the channel.

6.1.3 Channelization Summary

Channelization can provide very effective flood control anywhere within the study area. However, the efficiency of this solution must be weighed against the installation cost and the aesthetic impact that such a measure would have on the natural beauty of the desert environment. Details of additional channelization concepts for the study area are presented in Sections 6.3 and 6.4 of this report.

Channel design should address the following issues:

1. Location of a suitable outfall, so as not to discharge concentrated flows onto property that would not naturally receive such flows. Also, avoid unnatural diversion of water from one drainage area to another.

2. Conduct a thorough sediment transport analysis to prevent unacceptable erosion of channel banks and/or bed. Sediment transport analyses should also investigate the potential for aggradation (sediment deposits) to occur in the channel.

3. Identify drop structure requirements, in order to keep flow velocities within non-erodible limits.

4. Consider use of low-flow outlets to maintain small flows into natural washes that are intercepted by man-made channels. This is an important environmental consideration.

5. Consider stabilization of channel banks to prevent headcutting resulting from lateral inflows cascading over the channel bank.

6. Caution should be used when considering the construction of raised berms (levees) along the channel bank. Such berms will restrict lateral inflows and cause new floodplains to be created along the land-side of the berm.

7. Channel alignments should consider severance problems that may result from severing public access to specific tracts of land. Bridge construction, which may be required to cure such problems, should be considered an integral part of the channelization plan.

8. Different types of channel lining should be considered. Depending upon site-specific conditions and needs, fully-lined, partially-lined, or non-lined channels might be considered. Lined sections are generally more hydraulically efficient and require less right-of-way.

9. The channel design should project aesthetic appeal. This can be achieved through curvilinear alignments, partially vegetated (with natural desert species) bed and/or banks, and the use of color additives to any channel lining materials.

10. As with any prospective drainage solution, the cost and benefits associated with the channel design should be compared to the costs and benefits resulting from other alternatives, i.e., a benefit:cost analysis should be performed. However, the final selection of a drainage solution should not be based solely on technical and economical factors, i.e., environmental and social concerns should also be evaluated.

Within the Paradise Valley Fan Terrace study area, channelization might find more economic justification in the southern parts of the watershed, that are exposed to a higher sheetflow potential. Channel systems could eliminate, or substantially reduce, flooding over large areas inundated by sheetflow. Since the northern areas of the watershed are less prone to sheetflow, (i.e., more incised topography) they would appear to benefit less from channelization. However, if development in these northern areas is not confined to "high ground" that exists between washes, this assessment could change.

6.2 DETENTION/RETENTION BASINS

A reduction in downstream peak discharge can be achieved through the installation of detention and/or retention basins. Detention basins temporarily store floodwater and release it at a metered rate, which will not damage downstream areas. Retention basins operate on the concept of permanently storing floodwaters (no downstream releases) and letting it infiltrate into the ground.

Detention/retention concepts may take place on a small scale that might achieve floodwater storage on individual lots, or on a larger scale, such as would be associated with regional basins that would receive floodwaters from large portions of the upstream watershed.

Small-scale, on-site detention/retention could be effectively employed anywhere within the study area. This could be achieved through continued enforcement of the City's requirement for retention of the runoff from the 100-year, 2-hour storm that is generated within the boundaries of a proposed development. A collection and storage facility for the required runoff volume could be constructed simultaneously with the installation of the utility and street system for a specific development. The development (be it a single lot or large commercial tract) would be graded to direct all runoff to an on-site retention basin that would be excavated as part of the earthwork construction for the site.

The installation of large, regional detention/retention basins within the study area would have to be accompanied by the installation of a system of collector channels. The stream pattern within the study area is characterized by literally hundreds of small, parallel ephemeral washes. In order to capture the runoff from these washes, interceptor channels would have to be constructed to divert the flow in the washes to the regional detention/retention facility.

The lack of any deep valleys, or major incised watercourses, makes the construction of flood control reservoirs impossible within the study boundaries. However, portions of the contributing drainage area that lie within the City of Scottsdale (east of Scottsdale Road and north of Jomax Road) contain more incised topography that does provide the potential for achieving significant detention storage without the need for long collector dikes or interceptor channels. Several natural detention basin sites were

previously identified as part of the General Drainage Plan for North Scottsdale, Water Resources Associates, Inc. (WRA) 1989. Construction of detention basins at these Scottsdale sites would provide some reduction in flow rates through the City of Phoenix study area, although the exact amount of reduction is unknown.

Pursuit of either detention or retention basins (large or small scale) will be accompanied by two potential problems. The first problem will occur in response to the sediment trapping that will take place in these basins. The accumulation of sediment in these basins will deprive downstream washes of their natural sediment supply. This sediment deficit will induce a change in the fluvial equilibrium of these downstream washes. This change in equilibrium will occur in the form of streambed degradation, as the washes flatten their bed-slope to bring their sediment transport capacity into equilibrium with the reduced sediment supply. As these streambeds degrade (incise), bank sloughing will occur and lateral bank erosion will be accelerated. This could jeopardize any structures located near the banks of the washes. This problem was addressed in the City of Scottsdale detention basin concepts by using proportional weirs that would allow the bed-material load to pass through the bottom of the weir opening.

The second problem associated with detention basin concepts is related to the water release schedule. For a flood control system that might include multiple basins, it is important to ensure that the outflows from the basins are synchronized in a way that will minimize the potential for the outflows to combine in some manner that might actually cause a downstream peak discharge that is higher than that which would occur under natural conditions.

The effectiveness of detention/retention basin concepts was quantitatively analyzed for limited test portions of the study area. This analysis was completed with the use of reservoir routing operations in the HEC-1 models. Both retention and detention concepts were evaluated. The assumptions used for each case, and the modeling results are discussed in the following sub-sections:

6.2.1 Retention Basin Analysis

A quantitative analysis of retention basin concepts was based on the following assumptions:

1. Retention volume is based on 100% of the direct runoff (DRO) resulting from the 100-year, 2-hour rainfall. This precipitation is assumed to fall as part of the 100-year, 24-hour, Type IIA storm being used for this analysis. Accordingly, 24-hour curve numbers are used to compute the DRO that will be used to size the retention basin.

2. For concept analysis purposes, all basins are assumed to be 10-foot deep cubes. Surface area is determined by dividing total DRO (AF) by 10 ft.

3. Basin overflow is modeled as weir-flow over the top of a dam, using an ST card in the HEC-1 model. Overflow will commence when the DRO from the 100-year, 2-hour precipitation has filled the basin.

4. The floodwater retention that is being simulated in this analysis can occur as small-scale, on-site retention or large scale, regional retention. For the purpose of evaluating downstream reductions in peak discharge, the model simulates the retention at a single reservoir location, when in fact the retention could occur as on-site retention spread over the entire upstream watershed.

5. Future land-use conditions are assumed for the entire watershed.

6. Since major portions of the contributing watershed lie within the City of Scottsdale, the runoff response to the City of Phoenix area will be partially dependent upon enforcement of drainage ordinances by the City of Scottsdale. To approximate the impact of this situation, two cases were evaluated. Case 1 assumes retention of the runoff from the 100-year, 2-hour storm for the entire watershed. Case 2 assumes retention will occur on the City of Phoenix lands, but not on those within the City of Scottsdale.

The results of the test models used for the retention basin analysis are summarized in Table 6.5. All three of the retention basin test sites are located east of Cave Creek Road, and all have substantial portions of their contributing watershed located within the City of Scottsdale. Accordingly, the effectiveness of the retention concept is very dependent upon enforcement of retention policies by the City of Scottsdale. This is verified by a review of the last three columns of Table 6.5, which show dramatic differences in downstream peak discharge between the cases of retaining runoff just from City of Phoenix lands versus retention on both Phoenix and Scottsdale lands.

6.2.2 Detention Basin Analysis

The detention basin analysis utilized the same test sites (excluding Sub-Basin 3660) and basic assumptions used for the preceding retention basin analysis. However, the detention basin models also include the following additional assumptions:

1. Model T2A.24I: A constant flow-flow discharge of 69.5 cfs was added to the reservoir routing operation to empty the basin within 36 hours. The detention basin has a storage capacity equal to the

runoff from both Phoenix and Scottsdale lands from the 100-year, 2-hour storm. Any runoff in excess of this volume will be routed over the spillway in the reservoir simulation.

2. Model T2A1.24I: This is the same detention basin site as Model T2A.24I, except the available storage capacity has been reduced to match the runoff volume from only City of Phoenix lands. A constant discharge of 25.3 cfs is included to empty the basin within 36 hours.

3. Model T2A2.24I: This model uses a detention basin with sufficient capacity to detain the runoff from both Phoenix and Scottsdale lands from the 100-year, 24-hour storm. A constant discharge of 129.7 cfs is used to empty the basin within 36 hours.

4. Model T2C.24I: This model simulates a detention basin at the intersection of Cave Creek Road and the Lone Mountain Road interceptor channel discussed in Section 6.1.2 of this report. The basin has sufficient storage capacity to detain the entire 100-year, 24-hour inflow hydrograph, with a constant low-flow outlet discharge of 303 cfs.

The results of the test modeling for these detention basin scenarios are summarized in Table 6.6. As with the retention basin scenarios, a dramatic improvement in downstream flood peak reduction occurs when additional storage is provided for runoff from the City of Scottsdale portion of the upstream watershed.

The detention basin scenario provides slightly greater reduction in downstream peak discharge because of the constant low-flow discharge that reduces the magnitude of spillway discharges, i.e., the maximum reservoir water level is lower due to the continual low-flow release.

6.2.3 Detention/Retention Basin Summary

In summary, detention/retention basin concepts can be employed within the watershed to achieve substantial reductions in peak discharge to adjacent downstream sub-basins. However, due to the long, narrow shape of the majority of the watershed sub-basins, the downstream benefits from retention/detention basins usually begin to diminish within 1.5 to 2.0 miles below the basin location. This shape of watershed tends to generate peak flow rates from runoff relatively close to a measurement point, i.e., due to the long travel times, the downstream peaks are gone by the time the flood wave from the upper watershed arrives.

Achieving retention/detention goals through small on-site storage basins (scattered throughout the watershed) would probably be more cost effective and beneficial than attempting to provide the required storage capacity at large regional basins. The regional basin concept also tends to be somewhat impractical for the sheetflow environment that is characteristic of large portions of this study area, i.e., long diversion dikes or interceptor channels are required.

The lack of natural, deep valleys through the study area will also require that retention/detention storage capacity be achieved through "below ground" excavation. This type of storage can be expensive because of the 1 to 2 percent ground slopes in the study area that require substantial overburden excavation to reach level ground (see discussion in Section 4.0 of this report).

Because of these factors, regional storage basin concepts should be compared to other alternatives before a final drainage plan is adopted for a specific site.

6.3 TATUM RANCH

Tatum Ranch consists of nearly two and one-half square miles of planned community development that includes residential, commercial, and resort hotel land-uses. The development also provides designated acreage for both school and church facilities.

Located near Cave Creek Road, between Dynamite and Lone Mountain Roads, Tatum Ranch is situated on a pediment that has a more well-defined channel geometry than the adjacent (south of Dixileta Road) alluvial fan terrace. Although the sand-bed washes through Tatum Ranch are not deeply incised, they are generally wider and have more hydraulic capacity than the densely braided rill network on the adjacent fan terrace. Tatum Ranch also appears to have more topographic relief between washes than those areas on the adjacent fan terrace, i.e., Tatum Ranch might be described as a slightly more "hilly" or "rolling surface" than the flatter fan terrace. These topographic differences reduce the potential for widespread sheetflow through Tatum Ranch, although the occurrence of such a phenomenum should not be completely ruled out.

The Master Drainage Plan for Tatum Ranch was prepared by Coe and Van Loo Consulting Engineers, Inc. (CVL). The drainage concepts were developed with a basic objective of minimizing disruption of the desert landscape. This has been accomplished by utilizing several of the more prominent natural washes (passing through the development) as major collector channels for runoff that is generated within the development and diverted into the collectors. The capacity of these natural collector channels has been increased by constructing a raised earth berm along each side of the natural washes, i.e., the washes are essentially being leveed along each bank. To provide protection

against lateral bank erosion, a vertical wall (one foot wide) is constructed within the earth embankment (this wall is constructed as a trench, and is backfilled with concrete. These artificial berms are constructed so that the natural wash has a contained width of approximately 100 to 150 feet. In some locations, the natural washes are allowed to flow through golf course areas without the raised berms along each side of the wash. Apparently the golf courses are contoured so as to safely contain any overflow from the natural washes.

Flows being carried by these washes are routed through concrete box culverts at street crossings. The inlets to the box culverts consist of relatively long tapered concrete channels (roughly 100 to 350 feet long) that are flared to the full width of the bermed wash at the upstream end of each channel. After passing through the culvert, the water is continued through a lined channel section (approximately 100 to 350 feet long) that ultimately transitions to a flared outlet designed to match the width of the downstream wash. These culvert inlet and outlet channels are attractively designed and utilize a color additive to give the concrete a tan appearance that blends well with the native desert soils.

Application of the Tatum Ranch "wash improvement and stabilization concept" to other watershed locations should only be undertaken with careful consideration being given to possible adverse impacts that might be created by this concept. Such impacts might be summarized into the following categories of questions:

1. Will bermed washes be capable of capturing all upstream sheetflow?

2. Will the interception and unnatural diversion of on-site runoff into bermed washes change the equilibrium of these small fluvial systems?

3. Will the unnatural concentration of water and sediment into the bermed washes aggravate flooding and sediment problems at the outlet of such washes?

4. Will the construction of raised berms along the edge of natural washes create new miniature floodplains along the upstream side of such berms?

An expanded discussion of each of these issues is presented in the following paragraphs:.

Upstream Sheetflow

A field inspection of the Tatum Ranch development revealed that the man-made berms along the natural washes terminated at the upstream property boundary. The abrupt termination of these berms provides no mechanism to capture incoming sheetflow or overbank flow that might extend beyond the 100 to 150 foot wide swath of wash that is contained between the berms. Under such conditions, the larger flows that exceed the bankfull capacity of the natural washes will simply flow around the bermed section of the wash, since there is no collection dike to capture these flows and funnel them into the bermed cross-section.

This issue was discussed with representatives of CVL, who indicated that any sheetflow that bypassed the bermed sections of washes would be intercepted by man-made channels aligned approximately perpendicular to the natural path of sheetflow. Some of these man-made interceptor channels are located along the upstream property line of the development, while others are located through interior portions of the development.

Application of such a concept to intercept and control overland flow should be based on assurance that those areas exposed to overland flow, upstream of the interceptor channels, contain sufficient topographic relief (either natural or through raised building pads) to prevent inundation of finished floors. Furthermore, the design of the interceptor channels should ensure that such channels have sufficient sediment transport capacity to convey the incoming sediment load to the main collector channels.

Equally important in the design of the interceptor channels is the inclusion of a mechanism to prevent headcutting along the side of the channel that is receiving the lateral inflows. The construction of these channels will undoubtedly intercept numerous small washes. Field inspections of one of the major interceptor channels in Tatum Ranch provided graphic evidence of this phenomenum. In this particular case, the interceptor channel had a lower invert than that of the natural channel being intercepted. As a result, headcutting had occurred trough the bank of the interceptor channel as flow from the natural channel cascaded down the bank of the interceptor channel. Subsequent inspections of this site revealed that grouted riprap had been placed at these locations to prevent further headcutting problems.

A review of the concept drainage plans for Tatum Ranch indicates that the interceptor channels are to be fully concrete lined with a cutoff wall along the top of each bank. It is important that such channels be lined to prevent the headcutting problems discussed previously. The design of a cutoff wall should avoid a vertical wall in favor of an angled wall that can guide incoming flows up and over the bank of the interceptor channel. A vertical wall would create a very blunt obstruction to flow that would aggravate scour and undercutting along the channel lining at the top of the channel bank. "Weep holes" should also be included at the toe of the channel lining to alleviate hydrostatic pressure that might build-up behind the wall and cause cracking or structural failure of the concrete lining.

Disruption of Natural Equilibrium

The combination of artificial berms along the natural washes, and the diversion of overland flow into the bermed washes, will alter the natural hydraulic characteristics of these washes. These two unnatural phenomena will cause the washes to routinely carry more water than they would under natural conditions. Perhaps the most influential impact will be the increase in flow velocity that will accompany the concentration of water in the bermed washes. The higher velocities will be more erosive and possibly lead to the destruction of some native vegetation that could naturally withstand the shallower and slower flows that occur in un-bermed washes.

The increase in velocity will also cause an increase in sediment transport capacity through the bermed washes. Unless this increase in sediment transport capacity is accompanied by an equivalent increase in sediment supply, the channel bed-slopes will begin to flatten in order to reduce the sediment transport capacity back to a value that is in equilibrium with the sediment supply. The adjustment to a flatter channel slope will occur through vertical incisement of the channel bed. Severe equilibrium adjustments might cause bank instability problems and possible undercutting of the vertical concrete wall embedded in the earth berms on each side of the wash.

Should the additional sediment being delivered to the bermed washes (by the interceptor channels) exceed the transport capacity of the bermed washes, the opposite effect will occur, i.e., the washes will begin to aggrade in an effort to increase the channel slope to generate more sediment transport capacity. Such aggradation may reduce the design capacity of the bermed washes to the point where they could no longer carry the design discharge without overtopping the side berm.

In summary, it is very important to consider the potential impacts that unnatural interception and diversion of water will cause to both the vegetation community in the wash and to the hydraulic/sediment transport characteristics of the wash.

Concentration of Flow at Channel Inlet

As stated previously, the artificial berms will concentrate larger discharges within the bankfull boundaries of the desert washes. This confinement of flow will cause potential flooding and scour/sedimentation problems at the locations where the bermed washes outlet to a natural wash geometry. For example, a wash that might typically carry a bankfull discharge of 100 cfs, might now carry a bankfull discharge of 500 cfs with the artificial berms in-place. Obviously, such an increase in discharge has the potential to create unnatural flooding problems at the downstream point where the berms are terminated.

Probably the most common problem created at these outlet locations will be increased bank erosion and floodwater depths associated with the concentrated flows being directed into the shallow washes. Evidence of this phenomenum was observed at the outlet of one of the Tatum Ranch box culvert channels. This type of damage could easily serve as a cause for litigation by downstream land owners who feel their property is being subjected to unnatural flooding and erosion by these concentrated outflows.

Sedimentation might also occur in those downstream washes that are receiving more sediment inflows than they are capable of carrying. This could create sediment deposits near the channel outlets. Such a problem would be compounded by the reduction in velocity that would occur as the concentrated flows leave the bermed channels and begin to spread laterally. The velocity

reduction would cause more sediment deposition to occur. This might lead to localized channel blockage which would increase the potential for lateral flooding in those areas downstream of the bermed outlets.

Discussions with CVL personnel indicated that these problems had been considered in the development of the Master Drainage Plan for Tatum Ranch. However, no firm plan has yet been adopted to mitigate this potential problem. Some consideration has been given to designing some type of flow-splitter device to re-distribute the concentrated flows back to a more natural distribution.

Due to the litigation potential associated with this problem, the redistribution of concentrated flows should be given a high priority in the design of any drainage system.

Floodplains Created by Raised Berms

The raised berms that are constructed along the natural washes will undoubtedly block some of the overland flow that naturally drains to these washes. Depending upon the amount of upstream drainage area intercepted by these berms, there may or may not be sufficient water concentrated along the upstream side of these berms to cause flooding problems.

It is not anticipated that the flows being trapped by these berms would be sufficiently large to create a major problem. However, it is a phenomenum that should be investigated. Probably the easiest solution to mitigate this problem would be to reserve a drainage easement along the upstream side of these berms; the width of the easement would be based on site-specific hydrologic and hydraulic calculations.

Summary of Tatum Ranch Concept

It is not the purpose of this report to quantify the impacts that the Tatum Ranch "wash improvement and stabilization plan" might have on the natural desert environment. Instead, potential problems associated with this concept have been identified and discussed so that application of this concept to other locations might be undertaken with a more complete understanding of design requirements that will enhance the successful operation of such a drainage concept.

It would appear that the primary objective of the Tatum Ranch concept is to increase the hydraulic capacity of desert washes without extensive channelization that would destroy the native vegetation within the drainage corridors. Accordingly, this concept would have the most appeal in relatively flat areas of the watershed that contain washes with insufficient capacity to convey appreciable amounts of floodwater. However, the requirement of an overland-flow interceptor channel system might make this concept economically unfeasible for those flat areas where overland, or sheetflow, is the predominant flooding mechanism. The construction of man-made berms (levees) along the natural washes restricts the entry of sheetflow and creates miniature floodplains along the landside of such berms. Certainly, alternate solutions should be considered for a specific site in order to find the most economical and functional drainage system.

6.4 DESERT RIDGE

Similar to, but much larger than Tatum Ranch, Desert Ridge is another planned community development located within the project study area. Encompassing nearly 8.5 square miles, Desert Ridge is situated between Pinnacle Peak Road and the CAP Aqueduct and is bounded on the west by 32nd Street and on the east by a northern extension of 64th Street.

Desert Ridge is located on that portion of the alluvial plain (alluvial fan terrace) that is characterized by a dense network of small, braided, sand-bed washes. There is no single large channel that would serve as a major collector channel for this region. As a result of the limited capacity of these washes, this area is very prone to experiencing sheetflow.

6.4.1 Preliminary BRW Concept

Desert Ridge is still in the preliminary planning stages, i.e., no construction has yet taken place and there is no approved master drainage plan for the development. Site planning and drainage system design are being performed by BRW, Inc..

On December 8, 1989, BRW representatives provided a detailed briefing of the current drainage system concepts proposed for Desert Ridge. This concept consists of the construction of several, major drainage corridors that will parallel the natural drainage alignment through Desert Ridge. Due to the absence of any large-capacity natural washes through this area, the proposed drainage corridors will consist of mostly new channel excavation. However, the new channels will be 150 to 250 feet wide and will incorporate a maximum depth of approximately 5 feet. The channels are to be re-vegetated with natural desert species and will include a bike path.

The large width to depth ratio and emphasis on landscaping should provide a very natural and aesthetically appealing drainage system. The main channels do not include any raised berms along either bank. Accordingly, interior sheetflow will be free to enter the channel at any point.

Due to the flat topography in the Desert Ridge area, offsite sheetflow will be a major problem. This will require that some type of upstream collector system be constructed to intercept the offsite sheetflow and divert it into the designated drainage corridors passing through Desert Ridge. The September 1989 concept drawings prepared by BRW have addressed this problem by including provisions for both permanent and temporary "wing berms and channels" designed to intercept sheetflow and divert such flows into the designated drainage corridors. The final design of such a system will also require a careful evaluation of the same problems previously discussed for the drainage improvements at Tatum Ranch, i.e., disruption of natural equilibrium conditions and allowances for suitable outfall systems. As will be discussed in Section 6.4.2 of this report, another alternative is the construction of major interceptor channels around the northern and eastern periphery of Desert Ridge.

6.4.2 Major Interceptor Channel Concept

The Arizona State Land Department (SLD) has submitted an approximate alignment for two interceptor channels that would lie to the north and east of Desert Ridge. Both channels would start near the intersection of Scottsdale Road and Jomax Road, with one channel flowing in a westerly direction to an outfall near Cave Creek Road and the CAP, while the second channel would flow directly south (along the east or west side of Scottsdale Road) to the CAP. The east-west channel is approximately 7.3 miles long, while the north-south channel is about 5 miles long. The channel alignments are shown on Plate 6A.

A review of 7.5 minute quadrangle maps indicates the east-west interceptor channel will probably cause an un-natural diversion of runoff from portions of Sub-Basins 7, 8.1, 8.5, 9, 10, and 10.1 to the

CAP dike located west of Tatum Boulevard. Any possible adverse impacts of this diversion should be evaluated during final channel design.

It should be noted that the original SLD north-south concept channel alignment was located from one-quarter to one-half mile west of Scottsdale Road. In order to maximize the use of HEC-1 modeling data from the General Drainage Plan for North Scottsdale, Arizona (WRA 1989), the north-south channel was modeled along the east side of Scottsdale Road (a channelization model had previously been created for this alignment). This minor shift in alignment will have very little, if any, impact on the development of peak discharge values, channel hydraulics, and concept cost estimates for this reach of channel. However, if this north-south channel concept is ever pursued to final design, an exact HEC-1 model should be developed for the final alignment.

From a practical perspective, the alignment of the north-south channel along the east side of Scottsdale Road would provide substantially greater flood control benefits than an alignment to the west of the road. These additional benefits would be generated as a result of the flood control protection that would be provided to Scottsdale Road. This would be a major step towards enhancing the all-weather access along this major north-south transportation corridor. Such benefits should create adequate justification for the City of Scottsdale to cost-share in the construction of such a channel.

6.4.2.1 Channel Hydrology

Four HEC-1 models were created to determine the design discharges for specific reaches of the two channels. The models were based on existing land-use conditions and used the 100-year, 24-hour rainfall, in conjunction with the SCS Type IIA rainfall distribution.

Curve numbers, overland flow, and channel routing parameters were taken from the same database used for the large watershed models presented in Section 3.0 of this report.

Two models were created for each channel. The first model simulated the release of low-flows into the natural washes that were intersected by the proposed channels. To set an upper limit for the maximum channel discharge, the second model assumed that the low-flow outlets would be blocked and that 100 percent of the water would remain in the channel.

Channel routing operations (within the HEC-1 model) along the proposed channel alignments utilized an average roughness value of 0.025, existing ground slopes, and 2H:1V channel side-slopes. Channel bottomwidths varied from 25 feet to 75 feet. Should these channel concepts ever be pursued to final design, specific channel routing parameters should be chosen that match the type of channel being designed. The HEC-1 models should then be re-run with these revised parameters to get a final design discharge.

The HEC-1 models used for these interceptor channel analyses also model the reduction in peak discharge that would occur within Desert Ridge in response to construction of the interceptor channels. Due to the sheetflow potential through this area, the original "existing condition" HEC-1 models used very wide channel bottomwidths to simulate the movement of sheetflow through this region. With the proposed interceptor channels in-place, there was a possibility that these wide channel widths might create unreasonably low routing velocities for the reduced inflows through Desert Ridge. This phenomenon was evaluated by applying the original channel geometry to the computed HEC-1 discharges and computing normal depth velocities and Froude Numbers with Mannings' Equation.

With the exception of Sub-basins 28, 29, and 29.1, the original channel routing geometry was found to produce reasonable results for the lower discharges that occurred with the new channels in-place. Sub-basins 28, 29, and 29.1 had extremely wide channel widths (2500 ft. to 3750 ft.) that produced velocities on the order of 1 fps or less. Accordingly, the channel widths in these three sub-basins were reduced to values that produced velocities in the 3 to 4 fps range.

As stated previously, low-flow releases are included in two of the models to simulate small, non-damaging discharges into the natural washes intersected by the proposed channels. These releases are simulated in the HEC-1 model by the activation of divert routines at all interceptor channel concentration points.

Although actual low-flow release points would have to be determined during final channel design, the HEC-1 divert locations should provide a realistic simulation of this process. The divert ratios used in the model were based on the hydraulic capacity of a 36-inch RCP operating under inlet control through a flow depth ranging from 1.5 to 4.0 feet. This information was combined with the average rill density (taken from aerial photographs) for each sub-basin, and the angle formed by the intersection of the rills and the interceptor channels, to obtain an average length between washes, measured along the channel centerline. This calculation provided the number of low-flow outlet points required for specific reaches of the channels. The number of release points was then combined with the assumed discharge rating of a 36-inch RCP in order to arrive at a very rough estimate of the total discharge that would be diverted to low-flow washes from specific reaches of the interceptor channels. The 36-inch pipes were assumed to have a 60

percent discharge efficiency in order to reflect debris clogging and momentum forces associated with the direction of flow in the interceptor channel.

The investigation of low-flow outlets is very approximate at this concept level of analysis. The exact number, size, and hydraulic capacity of low-flow outlets would have to be determined during the final channel design. The ultimate assumptions used in the design of the low-flow outlets could have a significant impact on the channel design discharge and the discharge received by Desert Ridge.

As this report was in the final stages of preparation, a formal request was received from the Arizona Department of Transportation to evaluate an alternate east-west channel alignment along Pinnacle Peak Road. This alternate alignment is shown on Plate 6B. The purpose of this request was to examine the impact that the alternate alignment would have on reducing the peak discharge to be used for the design of an off-site drainage system for the Outer Loop Highway.

Although the alternate east-west channel alignment is located nearly 2 miles closer to the Outer Loop, it has a relatively minimal impact (about 25% reduction) on the peak discharge that occurs at the Outer Loop (see Concentration Points 6 and 10 in Tables 6.7 and 6.8). This minimal change is easily explained in terms of the long, narrow shape of the drainage areas intercepted by the Outer Loop. The peak discharge at locations in the downstream regions of these drainage areas is primarily a function of the runoff rate that occurs from sub-basins located near the peak discharge measurement point. In this case, the majority of the peak discharge for Sub-basins 6 and 10 comes from runoff within

these two basins. By the time additional runoff from distant upstream areas reach the Outer Loop, the runoff from Sub-basins 6 and 10 is essentially gone. Accordingly, these upstream sub-basins are not adding appreciable rates of runoff to the peak discharge seen at the Outer Loop. However, as the east-west channel is moved closer to the Outer Loop, correspondingly larger reductions in peak discharge would be noticed.

Table 6.7 summarizes the peak discharge data at major concentration points along the original east-west channel alignment beginning at Jomax Road. This table also shows the impact that the channel has on sub-basins within Desert Ridge and at the Outer Loop Highway. Table 6.8 presents the same information for the alternate east-west alignment along Pinnacle Peak Road. Table 6.9 provides similar information for the north-south channel near Scottsdale Road.

6.4.2.2 Channel Hydraulics

Using the hydrologic data presented in the preceding section, preliminary channel sizing calculations were performed for the following alternatives:

1. Completely lined (banks and bottom) concrete section placed on existing ground slope (Figure 5.1).

2. Partially lined section, with soil-cement banks and earth bottom, placed on existing ground slope (Figure 5.2).

3. Completely unlined section (earth banks and bottom), placed on a slope of 0.0067 ft/ft (Figure 5.3).

In order to preclude the analysis of an infinite number of channel geometry combinations, certain channel dimensions were fixed at constant values. Channel bottomwidth was selected as the only variable in determining the channel geometry. Table 6.10 summarizes the design parameters that were used in the hydraulic analysis. The data in Table 6.10 was input to Mannings' Equation in order to compute the channel bottomwidth, velocity, and Froude Number. These calculations were computerized in a Lotus 1-2-3 spreadsheet in order that excavation and channel lining quantities and construction costs could be automatically generated as part of the channel sizing computation. Detailed printouts of these spreadsheets, showing the results of all hydraulic calculations, are provided in Appendix C.

A brief discussion of each channel alternative is provided in the following paragraphs. Plates 7 and 8 provide centerline profiles of existing ground along the original east-west (Jomax Road) and north-south alignments, respectively.

Concrete-Lined Channel

A fully lined section (6-inch thick lining) was evaluated under the assumption that it would be constructed to match existing ground slopes, but have an invert 5.5 feet below existing ground. A cross-section of this channel is illustrated in Figure 5.1. This assumption, combined with the hydraulic parameters listed in Table 6.10, produced supercritical flow throughout the channel length for both the east-west and north-south channels. For the 100-year peak discharge, maximum velocities ranged from 13.3 to 25.4 fps through

various reaches of the channels. Along the original east-west alignment, channel topwidths (at a depth of 5.5 feet) varied from about 30 feet at the upstream end to approximately 100 feet at the downstream end of the channel. Topwidths along the north-south alignment ranged from 30 feet at the upstream end to 205 feet at the downstream channel terminus.

Fully lined concrete channels will minimize right-of-way requirements because of their hydraulic efficiency, but, as will be discussed in the following section, their construction cost may be prohibitive. From an aesthetic standpoint, concrete channels would probably not be favored by local residents because of the harsh, sterile lines associated with such channels.

Partially Lined, Soil-Cement Channel

This option, which would be the recommended alternative, provides a soil-cement bank to prevent lateral erosion, but leaves the channel bottom as a natural sand-bed. A cross-section of this channel is illustrated in Figure 5.2. This alternative was also evaluated along the existing ground-slope with an assumed channel invert 5.5 feet below natural ground. Analyses were performed with an "n" value of both .025 and .035. The lower "n" value would represent a well-maintained channel that is kept free of any noticeable vegetation. The higher roughness value would be more descriptive of a channel-bed in which some limited amount of vegetation would be allowed.

The aesthetic appeal that might be associated with a partially vegetated channel bed is achieved at a cost of having to acquire more right-of-way to accommodate the wider channel widths generated

by the higher channel roughness value. For example, with a design discharge of 4,478 cfs and a slope of .0154 ft/ft. the .025 "n" value produces a channel bottomwidth of 58.1 feet, while the .035 "n" value increases the bottomwidth to 82.4 feet.

Overall, the channel width along the east-west channel would easily fit into a 200-foot wide corridor. For an "n" value of .035, the channel topwidths ranged from approximately 32 feet to 163 feet. However, due to the much higher discharge being carried by the north-south channel, topwidths varied from about 38 feet to nearly 345 feet (n = .035).

Maximum velocities through either channel are approximately 20 fps (n = .025), which is non-damaging to a properly constructed soil-cement bank. Scour protection along the channel banks is provided by extending the soil-cement to a depth of 5 feet below the channel invert. This scour dimension is only an approximation used for generating preliminary cost estimates for this study. Actual scour depths would have to be determined as part of the final design process.

Based on the assumption of matching existing ground-slope, it does not appear (at this concept level of analysis) that grade control structures would be required for the soil-cement alternative. However, a final design sediment transport analysis might indicate a need for such structures at intermittent locations, although they should be very few in number.

Unlined Channel

The City of Phoenix requested that this alternative be evaluated with a Manning's "n" value of .050. Accordingly, this channel section could be constructed with a vegetation community that would closely resemble the natural desert washes in the area. A typical channel cross-section is presented in Figure 5.3. Unfortunately, to avoid exceeding the specified maximum velocity of 5 fps, the channels would have to be constructed on a slope no steeper than approximately .0067 ft/ft. This combination of high roughness value and flat slope produces extremely wide channel sections which would not seem practical for this area. For example, channel topwidths along the original east-west alignment reach nearly 350 feet, while those along the north-south alignment exceed 900 feet.

The practicality of this alternative is further jeopardized by the excessive number of drop structures that would be required along each alignment. Using the stated bed-slope of .0067 ft/ft, 3-foot high drop structures, and a maximum channel depth of 7.5 feet between drops, 75 drop structures would be required along the original east-west alignment, while 94 drop structures would be needed for the north-south channel. Installation of these drop structures would also require bank lining for a limited distance upstream and downstream of the drops.

Another major disadvantage of an unlined channel section is its inability to receive lateral inflows without headcutting beyond the channel bank. The channels through this area will be exposed to intercepting large swaths of overland sheetflow. As this sheetflow cascades over the channel bank, severe erosion will occur along the top of the channel bank. This erosion will propagate upstream (in the direction of the incoming sheetflow) in the form of a headcut.

This type of erosion will have the potential to completely destroy the upstream side of the channel bank, while simultaneously depositing large amounts of sediment (eroded bank material) in the channel bed. Even though the construction cost of unlined sections may, under certain conditions, be competitive with the partially lined soil-cement option, it is not a feasible concept to use in a sheet-flow environment where lateral channel inflows are expected to occur. Accordingly, a completely unlined channel section is not recommended for this project.

6.4.2.3 Cost Estimates

Detailed, reach by reach quantities and costs for excavation, channel lining, and right-of-way are presented on the spreadsheets included in Appendix C.

A complete summary of the Desert Ridge channel construction costs is presented in Tables 6.11 through 6.16.

The channel costs in Tables 6.11 through 6.16 do not include allowances for bridges. Due to the variation in channel width for the different lining alternatives, bridge costs would be lower for the concrete and soil-cement options than they would be for the unlined earth channel (assumes the same bridge locations would be selected for each alternative).

Cutoff walls would also be recommended along the top of the bank for the concrete and soil-cement lining options. Costs for cutoff walls are included in Tables 6.11 through 6.16 as a separate line item. These costs are based on 1-foot and 2-foot deep walls along the downstream and upstream sides of the channel, respectively. The configuration of both the concrete and soil-cement cutoff walls is shown in Figure 5.4.

The unit excavation costs used for the various channel options listed in Table 6.11 through 6.16 adhere to the excavation volume/cost schedule previously presented in Section 5.0 of this report. Soil-cement was estimated at $24 per cubic yard (in-place) and costs for 6-inch concrete lining were based on $30 per square foot (equivalent to $180/cubic yard at 6-inches thick). Landscaping costs of $7,500/acre were provided by the City of Phoenix. It should be noted that landscaping costs may be less than shown in the cost tables for the concrete-lined channels. This is due to the fact that a major portion of the channel right-of-way will be covered with concrete and, thus, not available for landscaping.

Table 6.16 presents a summary of the total construction cost for each of the Desert Ridge channel options. These total costs do not include right-of-way; they are simply a summary of the detailed costs presented in Tables 6.11 through 6.15.

Tables 6.17, 6.18 and 6.19 summarize right-of-way requirements for the concrete, soil-cement, and earth channel sections evaluated for the various Desert Ridge alignments. Three right-of-way scenarios are presented in each of these tables. As indicated by these tables, some channel alternatives will not fit within a constant drainage corridor width of 200 feet.

A review of the data in Tables 6.11 through 6.19 clearly indicates the partially lined, soil-cement channel is the most practical and economical alternative. This option could be designed to provide aesthetic appeal in addition to its economic advantages. For example, the soil-cement could be mixed with "Type C" fly-ash to provide both a savings in cement costs and to impart a more natural

"earthy" color to the soil-cement mixture. Using the higher "n" value of .035 would also allow some native vegetation to be placed within the channel cross-section. The top of the bank could be landscaped to any density desired. The channel could also be designed with some slight curvature to avoid the monotony of straight lines.

Comparing the two east-west alignments indicates that the Pinnacle Peak Road alignment would be the more economical alternative. This alternate alignment also provides a greater reduction in peak discharge through Desert Ridge and to the Outer Loop Highway. The alignment along Pinnacle Peak Road would also provide controlled outlet points (i.e., low-flow outlets to intercepted washes) for the capture and routing of water through the proposed Desert Ridge drainage corridors.

6.5 FEMA FLOOD INSURANCE STUDY

The Federal Emergency Management Agency (FEMA) has recently released "work maps" showing nearly all of the study area located south of Dixileta Road to be in a Special Flood Hazard Area (SFHA). The SFHA consists of both AO zones, which have numbered depth and velocity zones, and A zones, which do not have specified depths and velocities. These "work maps" do not represent final floodplain delineations. Preliminary Flood Insurance Study (FIS) maps are estimated to be published on October 16, 1989. A public review and appeal period will follow publication of these preliminary FIS maps.

The SFHA for this area has been classified by FEMA as "alluvial fan flooding". This is a very significant classification, because FEMA has very strict requirements regarding removal of the SFHA designation from alluvial fan areas. These requirements are specified under paragraph 65.13, Federal

Register, Vol. 54, No. 156, August 15, 1989. Perhaps the most stringent requirement in these rules is that a structure, or parcel of land, cannot be removed from the SFHA by elevating on fill. Only major structural flood control measures will be considered by FEMA for removing an area on a designated alluvial fan from the SFHA. The design of such flood control measures is subject to very strict design criteria, one of which is the requirement to identify the impact of the measure on all other areas of the fan.

Accordingly, unless individual homeowners, developers, or governmental agencies undertake construction of major flood control projects, any new development in this region will have to include adequate flood protection in the proposed design and include the purchase of flood insurance as part of the operating cost of the structure.

Because of the recent disclosure of the alluvial fan SFHA designation for this area, there is opportunity for confusion regarding what may be required as suitable flood protection for new development in the study area.

Problems may occur because of confusion over where to reference the finished floor elevation of structures located in the SFHA. In numbered AO zones (with depth/velocity designations), minimum federal regulations will be met if the lowest finished floor (including basements) is elevated to the designated depth above the highest adjacent grade (44 CRF, Sub-section 60.3, C, 7). Discussions with representatives from FEMA Region IX, indicated that "highest adjacent grade" would be defined as the highest natural ground elevation under the proposed finished floor boundaries. There are presently no federal requirements to address impacts due to the velocity designation in a numbered AO zone.

For A zones, where no depth is specified, minimum federal regulations require that new construction have the lowest finished floor (including basements) elevated at least two feet above the highest adjacent grade.

There are presently no federal regulations for flood-proofing requirements in X zones (areas of moderate or minimal flood hazards).

Although the preceding paragraphs define minimum federal regulations for development in designated SFHA alluvial fans, their application should be tempered with considerable engineering judgement. The methodology that was employed by FEMA to establish the depth/velocity designations within the SFHA does not acknowledge the topographic variation that can occur across the fan surface (the term "fan" is used in this discussion even though the author does not consider the regions in this study to be true alluvial fans). The depth/velocity designations simply imply that the specified depth and velocity of flow could occur anywhere within the designated zone boundary during a 100-year event. In reality, there may be defined washes or swales through the area where the flow will be mostly concentrated. Areas within the same zone that are naturally elevated on ridges or hills, may have very little exposure (possibly none) to floodwater.

When locating new structures within either AO or A zones, the engineer should identify the probable flow areas and conduct a hydrologic/hydraulic analysis to delineate the probable floodplain through the area. Structures should then be located well away from these flood-prone areas, yet still incorporate flood protection measures that will comply with minimum federal or local regulations, whichever may govern. These flood protection measures should also address potential erosion/scour problems. Some counties in California require that all building footings be extended to specified depths below ground, in order to protect against scour. It is reported that Kern County, California requires footings to a depth of three feet <u>plus</u> two times the flow depth.

There may be situations where substantially elevated ridges on hills exist in the SFHA zones. Such topographic features may be sufficiently elevated so there is no practical risk of flooding. It would appear that such areas could easily be proven to be out of any practical flood-prone boundaries. Documentation to this effect could be provided to FEMA, along with a request to remove the parcel, or elevated portion thereof, from the SFHA.

In summary, the engineer should make a concerted effort to identify the floodplain characteristics of any site located within SFHA alluvial fan zones. The engineer should not arbitrarily locate a structure any place within a numbered depth/velocity zone and assume that compliance with prescribed finished floor elevation requirements will be satisfactory for preventing flood damage. A site-specific hydrologic/hydraulic analysis should be considered mandatory.

6.6 "NO ACTION" ALTERNATIVE

Drainage requirements within the City of Phoenix are outlined in the Storm Drain Design Manual, Subdivision Drainage Design, City of Phoenix, September 1985. The primary drainage requirement, presented in Section VI of that document states: "All developments shall make provisions to retain the runoff of a 100-year, 2-hour duration storm falling within the boundaries of the development, unless the drainage can be directly carried to a major channel or natural drainageways whose capacity is adequate from the development to a major drainage outfall such as the Salt River. This requirement may be waived by the City Engineer for isolated developments under one-half acre in areas when there will be no critical drainage problem created by the additional runoff from the proposed development".

Drainage requirements for multi-lot developments retain the reference to a 100-year, 2-hour storm, but also add a statement that: "The design shall provide the calculated peak discharge after development does not exceed the peak discharge prior to development".

For the purpose of this discussion, a "No Action" alternative will be defined as maintaining compliance with current City drainage policies (as outlined in the previously referenced 1985 manual), as well as compliance with FEMA regulations associated with the proposed designation of large parts of the study area as a Special Flood Hazard Area (see Section 6.5 of this report). If properly enforced, these policies should prevent new development from being exposed to a high risk of flood damage. However, as discussed in Section 6.5, a major factor in the success of such a program will be dependent upon engineers' exercising sound judgement in the design of drainage systems for new developments in this area. It may be necessary for the City to develop supplemental drainage design standards in order to achieve this goal.

As previously stated, the defined "No Action" alternative is considered an effective floodplain management tool for the entire study area. However, it is recommended that no waivers be granted for isolated developments under one-half acre in size. The cumulative impact of several such waivers, in the same watershed, may ultimately cause flooding problems.

The advantages and disadvantages of a "No Action" alternative are summarized as follows:

<u>Advantages</u>

1. Other than administrative costs to the City for technical review (which should already be in-place), there will be no large expenditures of tax funds for publicly funded flood control projects, i.e., the cost of drainage improvements on specific tracts of land would be funded by the owner.

2. The authority to implement this plan is already in-place.

3. If properly enforced, this alternative will provide an effective floodplain management tool for new structures.

4. By definition, this alternative precludes the construction of man-made channels, which are considered by many groups to be a scar on the natural aesthetics of the area. Accordingly, pursuit of this option will probably generate a less negative aesthetic impact on the land.

Disadvantages

1. The elimination of major structural flood control improvements may severely limit the extent of new development in the area, i.e., the majority of the area is proposed to be classified as a Special Flood Hazard Area. For local residents, this may be viewed as an advantage, since it will probably create a lower development density for the area.

2. Because of the difficulties in analyzing and designing effective flood control systems for such a unique area (i.e., scour, sedimentation, acceptable hydrologic and hydraulic analyses), the City may have to develop detailed design standards and procedures in order to force some standardization, or level of continuity, in the engineering analyses that will accompany new development in the area.

3. It may be difficult to enforce and/or police on-site retention. Experience in other areas of metropolitan Phoenix has shown that residents will frequently "fill-in" areas reserved for floodwater retention after the City inspectors leave.

In summary, the adoption of a "No Action" alternative will not reduce the flooding potential in the watershed beyond what it is under present day conditions. i.e., the area will still be exposed to a possible Special Flood Hazard Area (SFHA) classification. This statement is based on: 1) the City's retention requirements; 2) the City's stipulation that post-development peak discharge cannot exceed pre-development peak discharge; and 3) FEMA's requirement that only major structural flood control improvements can remove a SFHA classification from designated alluvial fan flooding areas. This plan will, however, ensure that new development in the watershed is constructed in a manner that acknowledges the flooding potential of the area and incorporates adequate drainage/scour features into the design and location of the structural components of the development, thus mitigating the potential for flood damage to future development.

7.0 STORM DRAIN ANALYSES

The purpose of this section is to present conceptual storm drain designs for use in future land development planning. A detailed storm drain plan has been prepared for the study area. The plan was designed assuming that there will be no channelization or detention/retention facilities constructed in the watershed. Unfortunately, this plan will probably need to be modified once channel alignments and/or detention/retention basin locations have been finalized. The storm drain plan is discussed in detail in Section 7.1 through 7.3 of this report.

7.1 HYDROLOGIC ASSUMPTIONS

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 were used to design the storm drain system.

The study boundaries for the project contain approximately 56 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 modelling process, it is necessary to sub-divide the watershed into much smaller sub-basins of relatively homogeneous hydrologic characteristics. The number and size of sub-basins is also dictated by the number of locations at which hydrologic output data is desired, i.e, storm drain junctions, etc.

The watershed was divided into two basins. The first basin consists of all areas contributing runoff to Cave Creek Wash. The second basin consists of the remainder of the watershed area which flows to the CAP Aqueduct. These two basins were further subdivided according to proposed street alignments provided by the City of Phoenix staff.

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. Rainfall values from the NOAA Atlas were distributed over a 24-hour duration using the SCS Type IIA rainfall distribution. The distribution is graphically depicted in Figure 3.2.

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 the 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. The storm drain analyses were based on existing watershed conditions. A detailed discussion of the selection of appropriate hydrologic parameters was presented in Section 3.2 of this report.

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, shape, boundary roughness, length and slope of channel elements. Due to the fact that subdivision street patterns have not been established, natural watercourses were used to route storm runoff through the sub-basins to a major arterial street. The HEC-1 pipe routing option was then used to route the runoff to the ultimate outfall.

7.2 CONCEPT DESIGNS

The conceptual storm drain system was sized from hydrographs generated by the HEC-1 computer model. The following assumptions were used to design the system:

1. Minimum pipe size = 3 feet (possible sediment concerns)

2. Maximum pipe size = 8 feet

3. Minimum pipe velocity = 5 feet/second

4. Maximum pipe velocity = 10 feet/second

5. Mannings "n" value - 0.012 (concrete pipe)

6. Bed-slope equal to existing ground-slope (unless pipe criteria have been violated)

7. Design storm is the existing condition 2-year, 24-hour event (Type IIA storm distribution with a 10 mi^2 areal reduction factor applied).

8. Watershed is divided into two basins -

    1) areas that flow to Cave Creek Wash
    2) areas that flow to Central Arizona Project Aqueduct.

9. Curve numbers determined from the SCS hydrologic soil map

10. Overland flow lengths, slopes, and roughnesses were determined by averaging sub-basin characteristics for the two basins.

11. Routing parameters are from the existing condition hydrologic model (assuming natural washes through sections).

12. Pipes flowing full.

Pipe sizings determined by the HEC-1 computer model were verified using normal depth calculations.

Plate 9 presents an illustration of the drainage patterns, sub-basin boundaries, and concentration points used to model the system. Complete listings of the input and output data associated with the HEC-1 models developed for the storm drain analyses are presented in the appendix "HEC-1 Models for the Paradise Valley fan Terrace Concept Drainage Study".

7.3 COST ESTIMATES

Construction cost estimates were developed for the design presented in this report. Since this is a concept report, based on preliminary information, a contingency factor of 25% was added to the compu- ted costs. This factor is included for unknown problems and engineering costs that may be identified in the advanced stages of the drainage system design.

Unit costs used for developing cost estimates are listed in Table 7.1. These costs were provided by City of Phoenix staff.

Table 7.2 summarizes costs for the storm drain system presented in this report.

8.0 CONCLUSIONS AND RECOMMENDATIONS

This report presents the results of an initial study to examine the flooding problems in the Paradise Valley Fan Terrace watershed. The hydrology component of this study concluded with the development of computerized rainfall/runoff models (HEC-1) for the entire watershed. These models, which were created for both existing and projected future land use conditions, provide a valuable tool for the analysis of flood control problems and solutions within any region of the study area.

Although detailed hydrologic and hydraulic analyses were conducted for specific drainage concepts within certain regions of the area, it is not the intent of this study to recommend a final drainage plan for the study area. In consideration of all the unknown development plans for the watershed, it would be premature to adopt a specific plan at this early stage of watershed development. Adoption of a specific plan at this time would provide no flexibility to accommodate probable land-use changes that will undoubtedly occur as the watershed evolves into an urbanized setting.

This report presents considerable technical data and analytical tools that can be used to provide quick analysis of several flood control concepts. Accordingly, this report should be used as a comprehensive data base to assist the City of Phoenix in arriving at a final solution to the drainage issues in this area. This process will undoubtedly take several years and considerable debate enroute to the acceptance and construction of a final master drainage plan. The evolution of this final plan will be dependent upon: 1) the way in which development actually occurs; 2) public discussion and acceptance of specific solutions; 3) some type of funding plan; and 4) continued review of drainage requirements and aesthetic factors by the City of Phoenix.

In addition to the detailed technical data that accompanies this report, several general conclusions are summarized as follows:

1. The study area presently contains regions of substantial flood damage potential, primarily on the alluvial fan terrace (pediment, alluvial plain) south of Dynamite Road. Major portions of the watershed have recently been classified by FEMA as Special Flood Hazard Areas subject to alluvial fan flooding.

2. Future development in the project watershed must recognize the potential for widespread sheetflow across the alluvial fan terraces and pediments. Development in these areas should consider substantial elevation of building pads, armoring of raised building pads, lateral setbacks from natural washes, and avoidance of any development within low-lying areas.

3. The forecast land-uses for this area will intensify the flooding potential if preventive action is not taken. Preventive action may take the form of continued enforcement of City drainage policies, and/or, the construction of major structural flood control systems (e.g. channels). The development of flood control solutions for the watershed should use an approach that considers impacts to adjacent and downstream properties. Failure to do so may simply transfer a flooding problem from one location to another.

4. The HEC-1 models developed for this study are very comprehensive and consideration should be given to using these models as a basis for the evaluation and development of any flood control alternatives for the study area.

5. Large regional retention/detention basins are not considered practical within the study boundaries because of the relatively flat topography which would require the construction of long collector dikes or channels to intercept and divert water to the retention/detention facility. However, smaller basins within a subdivision might be a practical way to meet City retention requirements. The grading plan and street alignment for a subdivision could be designed to function as the collector system for diverting water to the retention/detention basin.

   For low-density residential areas, the use of on-site retention basins (for each lot) is the preferred approach for achieving floodwater retention goals.

6. Man-made channels provide an effective flood control solution anywhere within the study area. However, channels do not enjoy widespread acceptance by City planners and local residents of the area. The anticipated rejection of this alternative is based on aesthetic considerations associated with the desire to preserve the natural appearance of the watershed. However, it is the opinion of the author that channels could be constructed and landscaped in a manner that would not create an adverse visual impact to the area.

7. Should public sentiment indicate majority support for maintaining the natural setting of the area, structural flood control solutions should not be pursued. Instead the pending Special Flood Hazard Area (SFHA) designation should be maintained and rigidly enforced by the City of Phoenix. Area residents should be made aware that mainten-ance of the rural lifestyle means coping with occasional flooding problems. These problems will occur in the form of restricted access due to washed out roads, possible loss of property

due to bank erosion of natural washes, and perhaps inundation of existing structures that may have been built in flood-prone areas. New development will be exposed to very strict drainage design criteria intended to minimize the potential for additional flood damage.

8. Underground storm drain systems are not considered a practical alternative for this region. First, storm drains designed for a 2-year event will do nothing to eliminate the SFHA classification for this area. Second, unless a major structural flood control system is approved by area residents, there will be no intermediate outfall for the storm drains, i.e., the pipes will have to be extended to Cave Creek or to the CAP dikes. Third, the desert landscape of this area will generate substantial sediment loads which may cause serious blockage of the pipes. Several roads within the region will be maintained with a natural dirt surface. The placement of catch basins within such an erodible environment is not practical.

The following approach is recommended for achieving resolution of the drainage issues that will accompany continued urbanization of this watershed.

1. Schedule a series of public meetings within the study boundaries to discuss available drainage options (i.e., channelization, on-site retention, no action, etc.) with local land owners. Use these meetings to gather input on the type of solutions that would be acceptable to area residents and determine their desire to participate in the funding of such projects (i.e. through a special district tax).

2. Based on input from the public meetings, develop specific drainage system alternatives and cost estimates. The HEC-1 models and channelization/retention basin design tools presented in this report can be used to perform these analyses with minimal man-hour effort.

3. Conduct a second series of public meetings for the purpose of presenting the drainage system data developed under Step 2. Use this series of meetings to secure approval from the public on a specific course of action, i.e., adopt a specific plan and identify a funding source and installation schedule.

4. Proceed with final design and construction.

In summary, this report is considered a first step in exploring the drainage issues and options that are associated with the Paradise Valley Fan Terrace. A detailed hydrologic model of the watershed is now operational, several drainage concepts have been evaluated at various levels of detail, and design tools have been developed to expedite the analysis of additional drainage system scenarios. This information should now be used to formulate a functional plan that is acceptable to the public.

9.0 PUBLIC MEETING

A public meeting was held at 6:30 p.m. on October 18, 1989 in order to present the results of this study to area residents. Notice of the meeting was published in the Arizona Republic, the Business Gazette, and the Paradise Valley Independent. The meeting location was at the Paradise Park West Mobile Home Park, 3901 East Pinnacle Peak Road.

The meeting was convened by Mr. Paul Kienow, Floodplain Management Engineer, City of Phoenix. Mr. Kienow presented a brief overview of the General Plan for Peripheral Areas C and D and summarized the complex drainage issues that will accompany urbanization of this area. Mr. Kienow then introduced Mr. Robert Ward and Mr. Jeff Holzmeister of Water Resources Associates, Inc. (WRA).

Mr. Ward presented an overview of the WRA drainage study and then initiated specific discussions of the drainage concepts that were evaluated as part of the study. These concepts included:

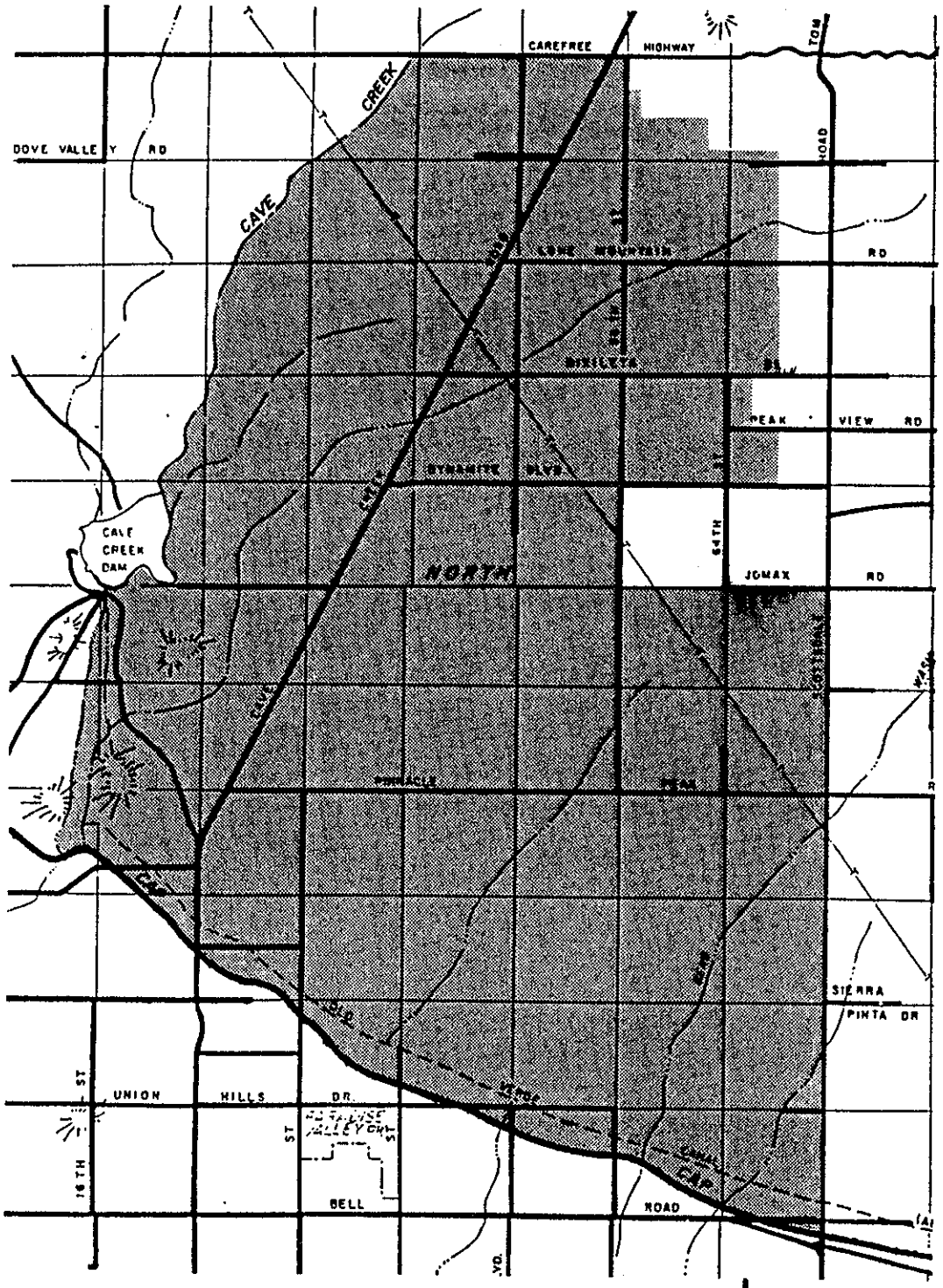
1. Retention/Detention Basins
2. Flood Control Channels
3. Tatum Ranch Concept
4. Desert Ridge Concept
5. FEMA Flood Insurance Study
6. No Action

Mr. Holzmeister concluded the presentation with a discussion of storm drain systems.

The purpose of this meeting was not to solicit approval from the attendees for a specific drainage system plan. Instead, the meeting objective was to inform the public of what concepts were being evaluated and to discuss the advantages and disadvantages of applying the concepts to different regions of the study area. The general response of those in attendance focused on a concern for preservation of the natural beauty of the area and how any drainage system improvements would be funded, i.e., would there be a possible increase in tax on local residents.

At the conclusion of the meeting, Mr. Kienow reminded all attendees that the City would welcome written comments on the drainage issues in the study area. As of January 3, 1990, the City has received no written comments in response to the public meeting. A mailing list of Peripheral Area C & D Planning Advisory Committee members and attendees of the Public Meeting is attached as Appendix D.

FIGURES



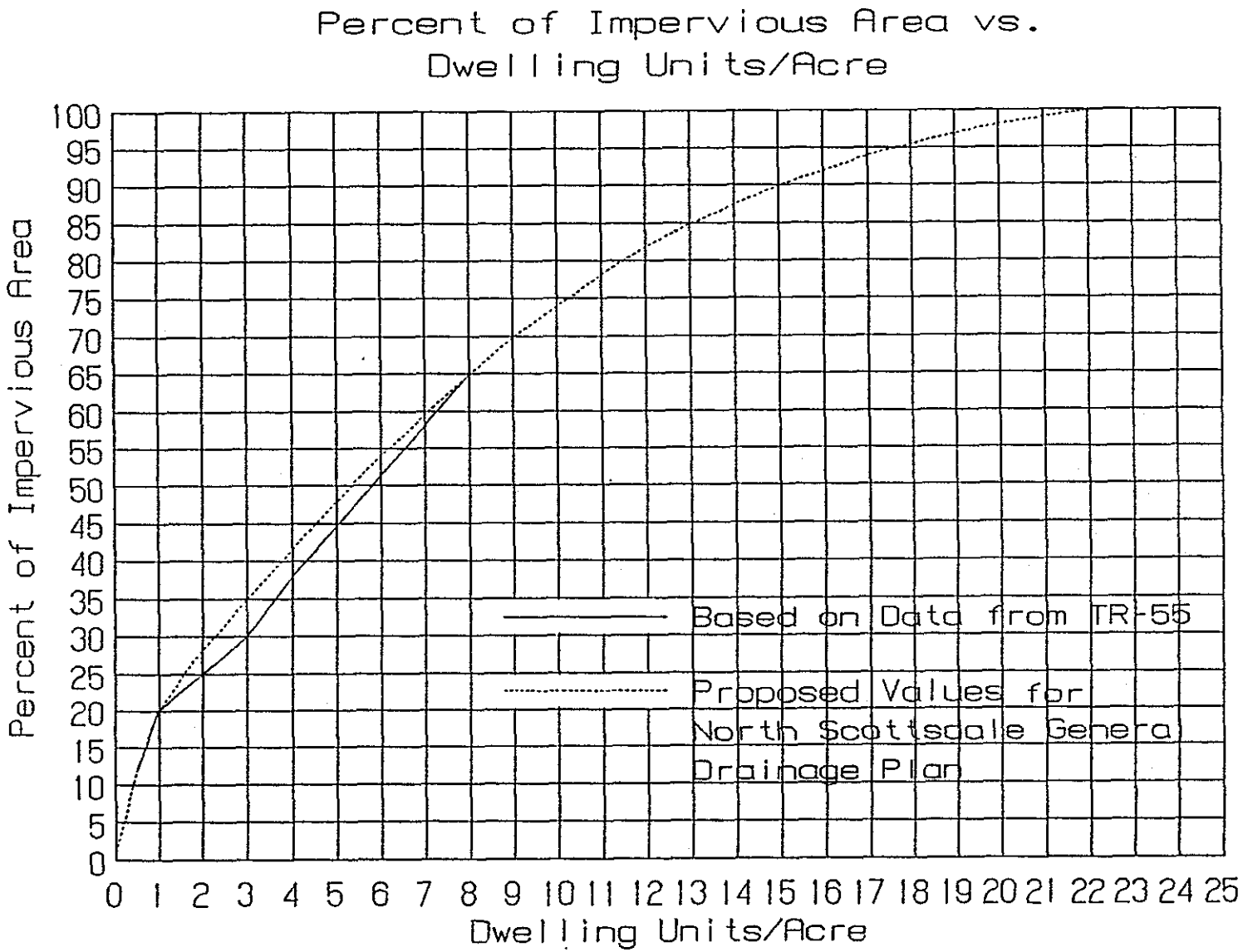
STUDY AREA

N

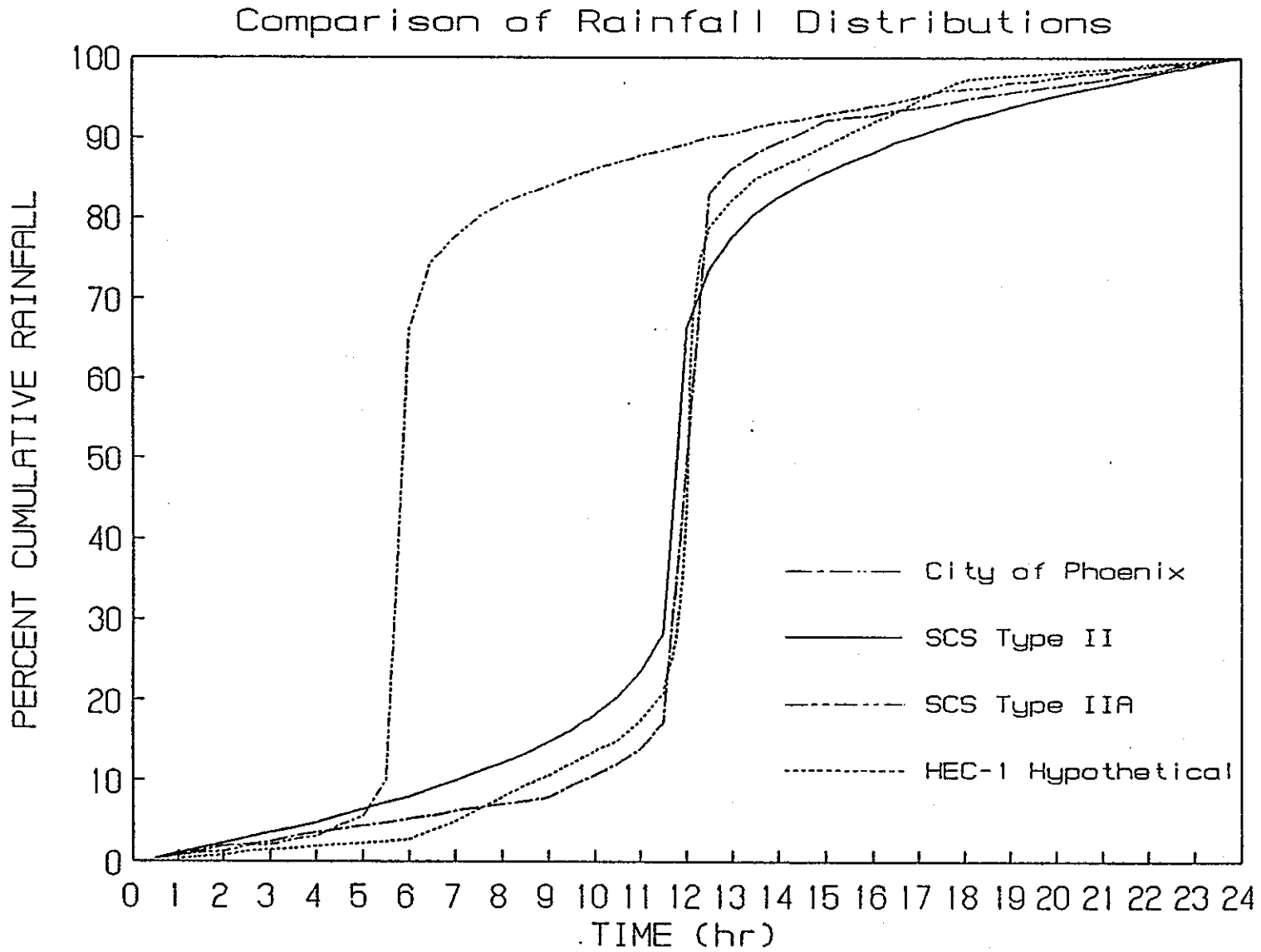
1 0 1 MILE

BASE MAP : Maricopa County Planning And Development

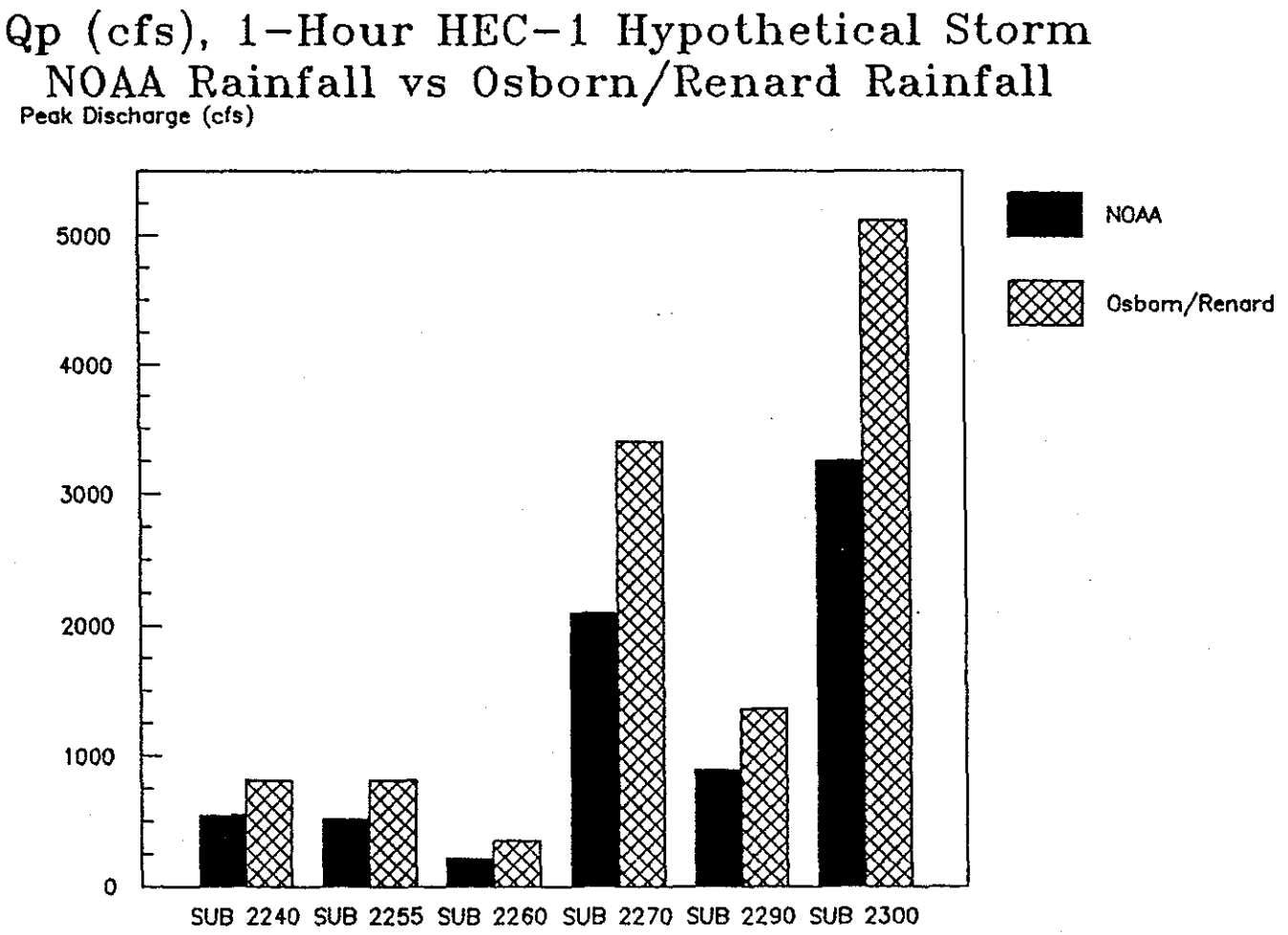
Water Resources Associates, Inc.	STUDY BOUNDARIES	FIGURE 1.1
PROJECT NO. AC00060 DRAWN BY KMA		
DATE 7/27/88 CHECKED BY SDN		



Water Resources Associates, Inc.	PERCENT OF IMPERVIOUS AREA VS. DWELLING UNITS/ACRE	FIGURE 3.1
PROJECT NO AC00060 DRAWN BY HLW DATE 9/25/89 CHECKED BY JJH		



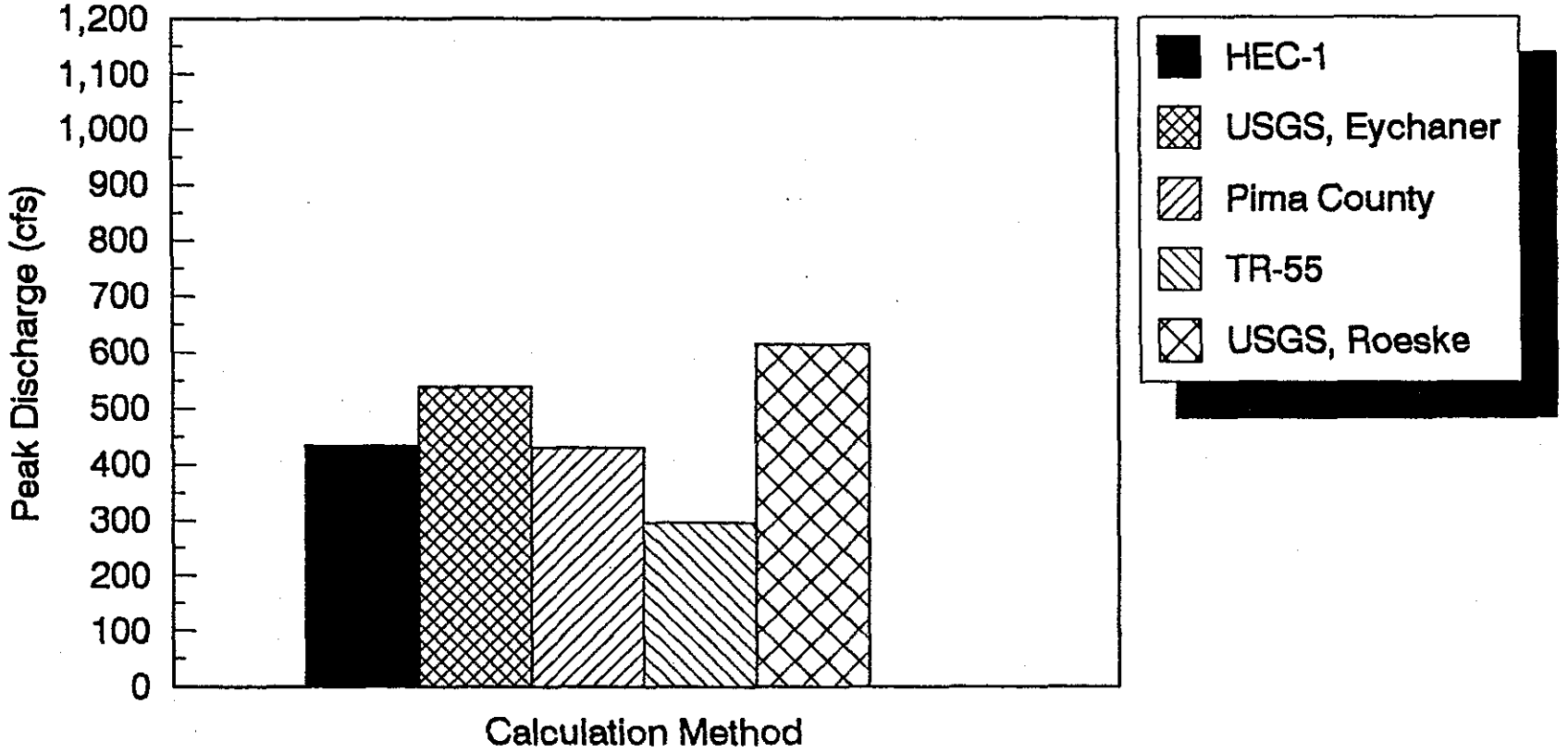
Water Resources Associates, Inc.	COMPARISON OF RAINFALL DISTRIBUTIONS	FIGURE 3.2
PROJECT NO AC00060 DRAWN BY RLW DATE 9/25/89 CHECKED BY JJH		



Water Resources Associates, Inc. PROJECT NO AC00060 DRAWN BY HLW DATE 9/25/89 CHECKED BY JJH	COMPARISON OF PEAK DISCHARGE FROM NOAA VS. OSBORN/RENARD RAINFALL	FIGURE 3.3

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3020

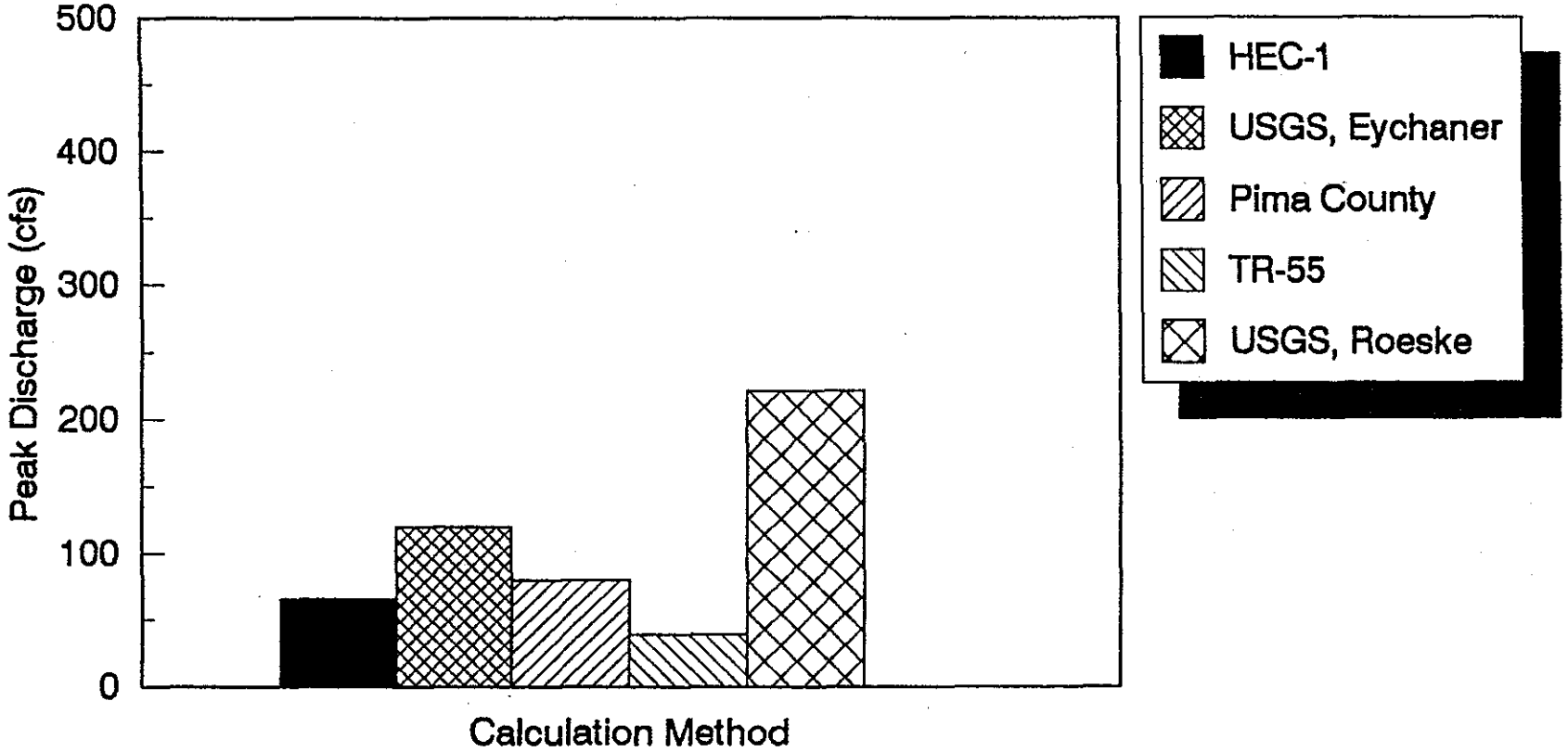


Drainage Area = 0.31 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II

Water Resources Associates, Inc.	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3020	FIGURE 3.4
PROJECT NO AC00060 DRAWN BY RLW		
DATE 9/25/89 CHECKED BY JJH		

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3135

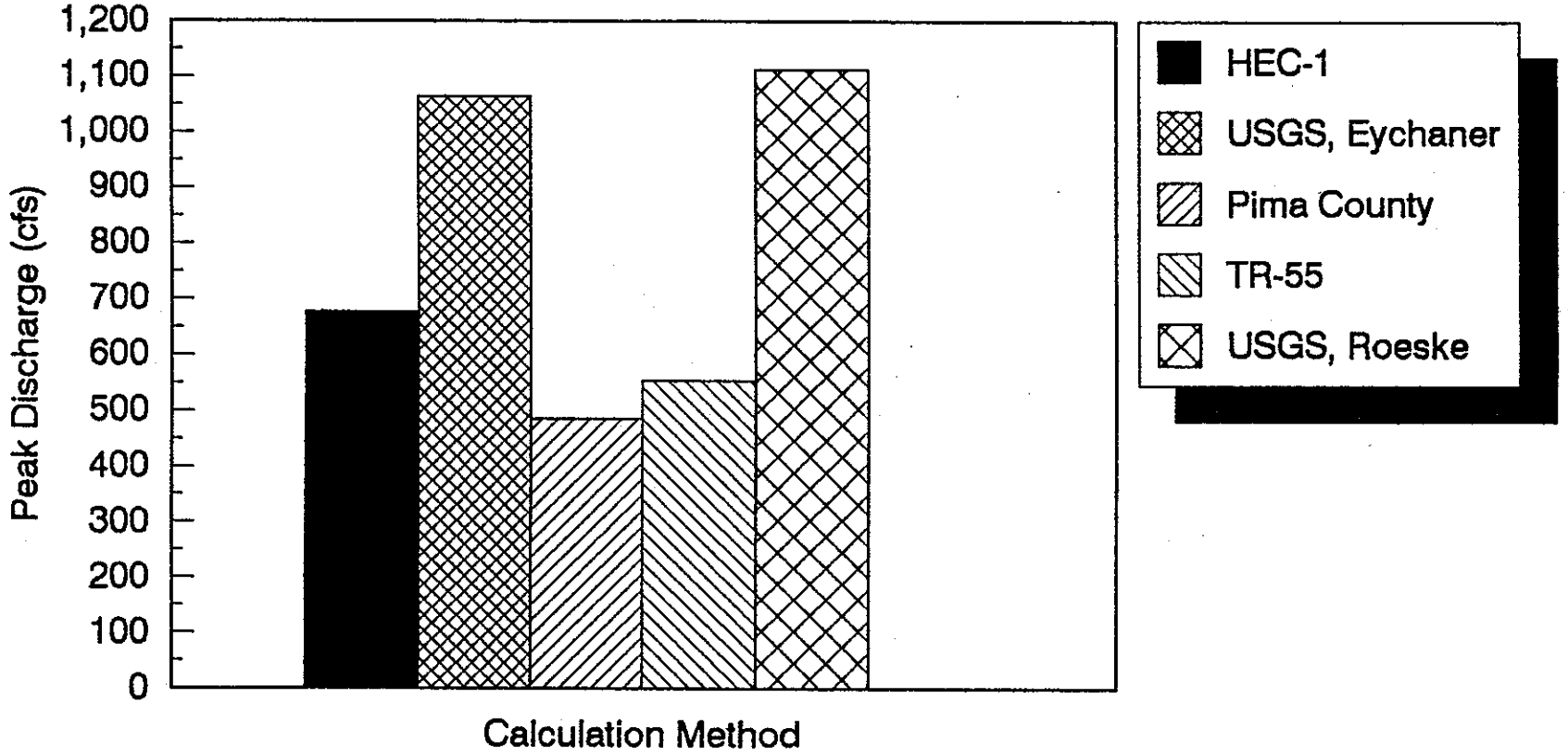


Drainage Area = 0.04 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II

Water Resources Associates, Inc.	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3135	FIGURE 3.5
PROJECT NO. AC00060 DRAWN BY RLW		
DATE: 9/25/89 CHECKED BY JJH		

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3150

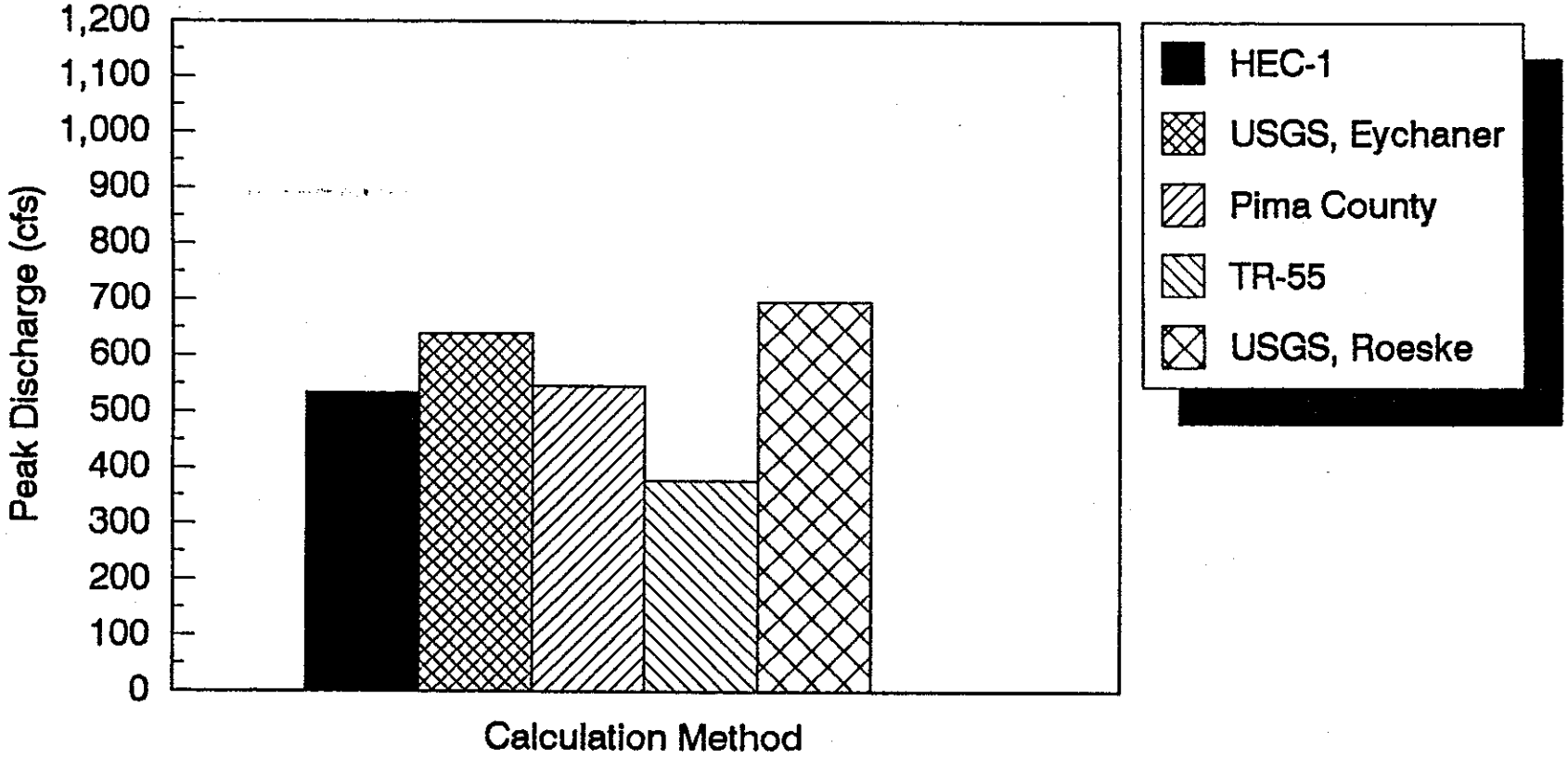


Drainage Area = 1.02 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II

Water Resources Associates, Inc. PROJECT NO AC00060 DRAWN BY RLW DATE 9/25/89 CHECKED BY JJH	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3150	FIGURE 3.6

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3160

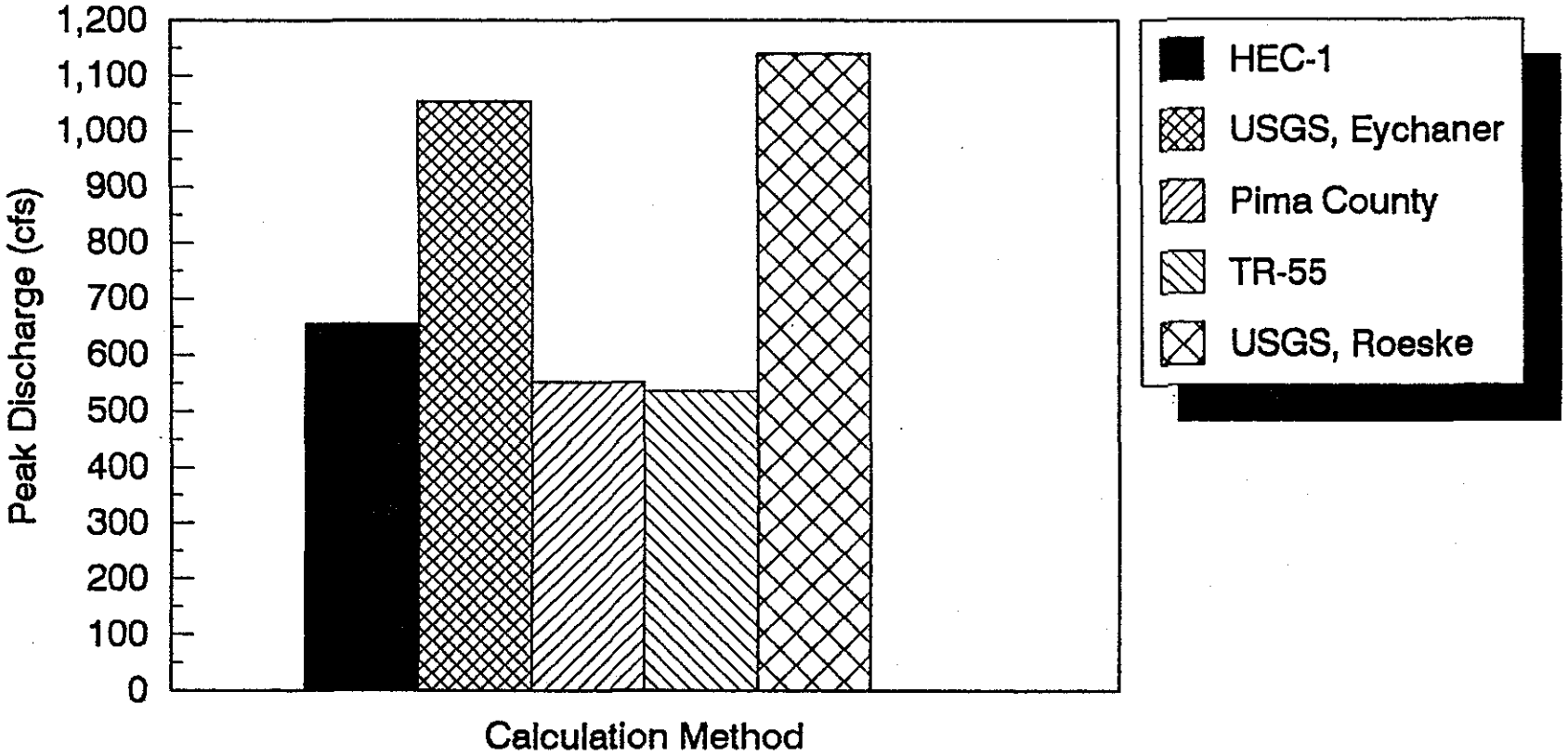


Drainage Area = 0.40 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II

Water Resources Associates, Inc.	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3160	FIGURE 3.7
PROJECT NO AC00060 DRAWN BY RLW DATE 9/25/89 CHECKED BY JJH		

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3400

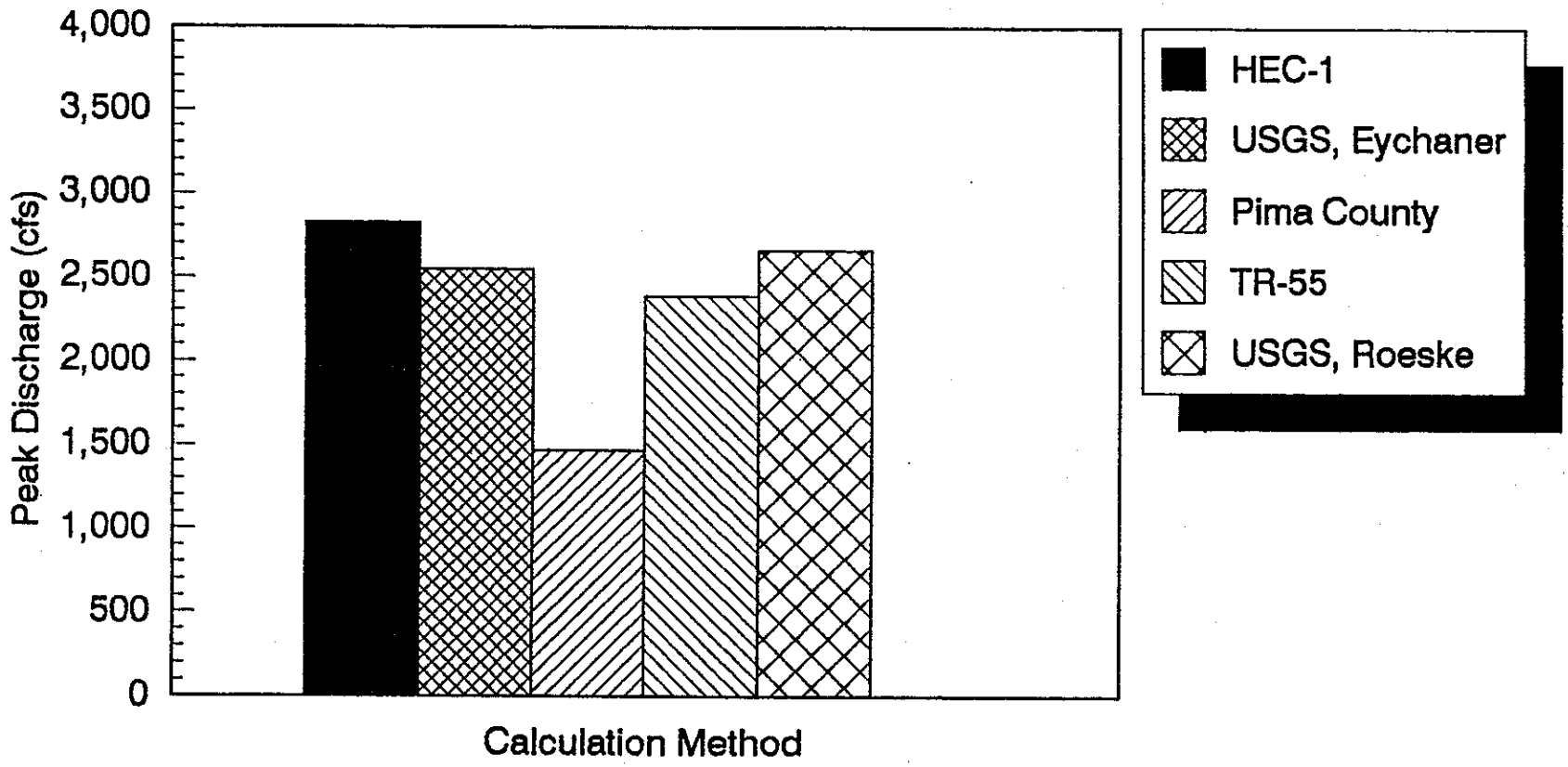


Drainage Area = 1.07 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II

Water Resources Associates, Inc.	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3400	FIGURE 3.8
PROJECT NO AC00060 DRAWN BY RLW		
DATE 9/25/89 CHECKED BY JJH		

HEC-1 Peak Discharge Verification

City of Phoenix, Paradise Valley Fan Terrace
Concentration Point 3490

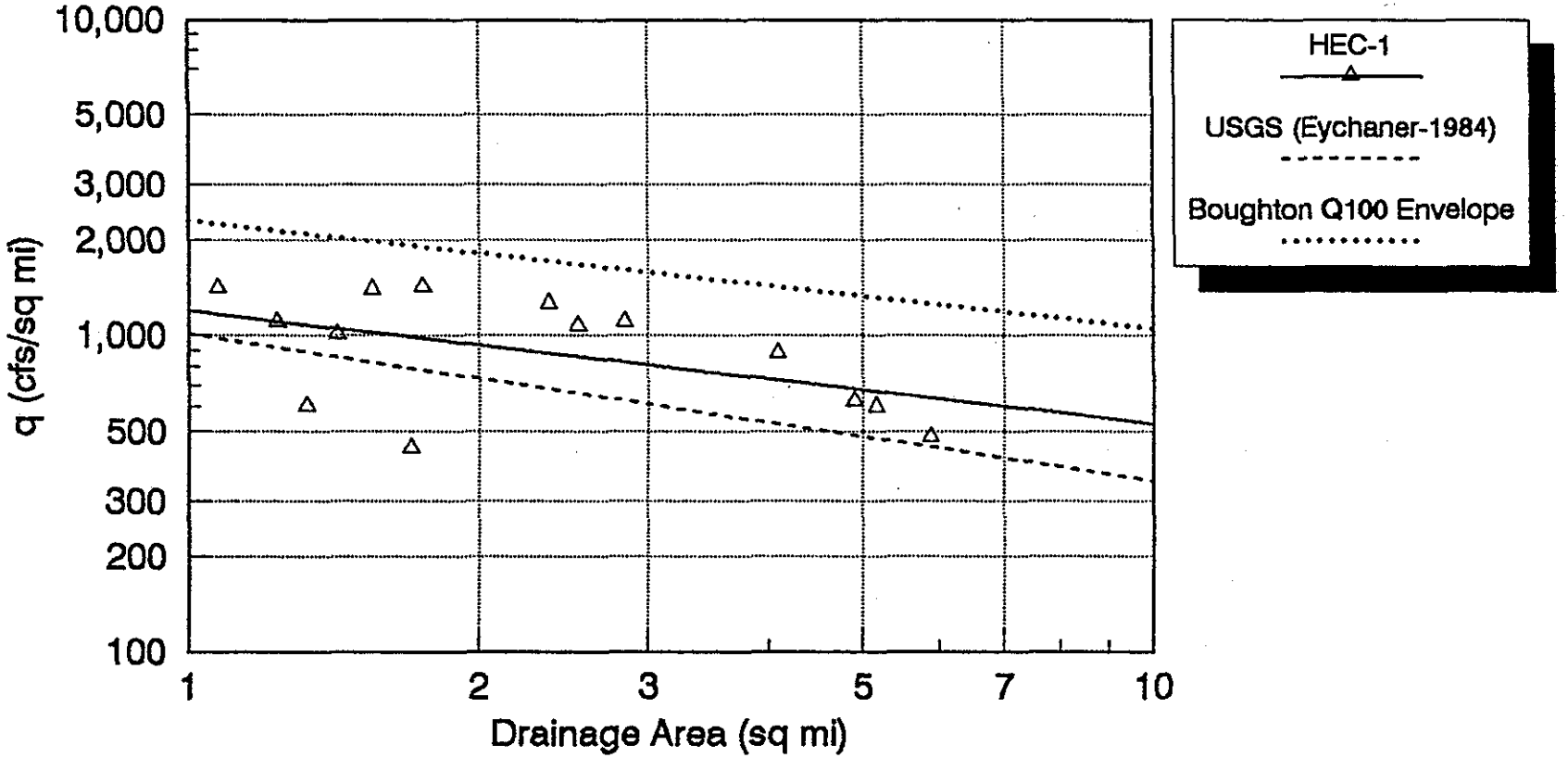


Drainage Area = 5.87 square miles
100-Year, 24-Hour, SCS Type IIA Storm
AMC II

Water Resources Associates, Inc.	GRAPHICAL PLOT OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR CONCENTRATION POINT 3490	FIGURE 3.9
PROJECT NO AC00060 DRAWN BY RLW		
DATE 9/25/89 CHECKED BY JJH		

Unit Discharge vs Drainage Area

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

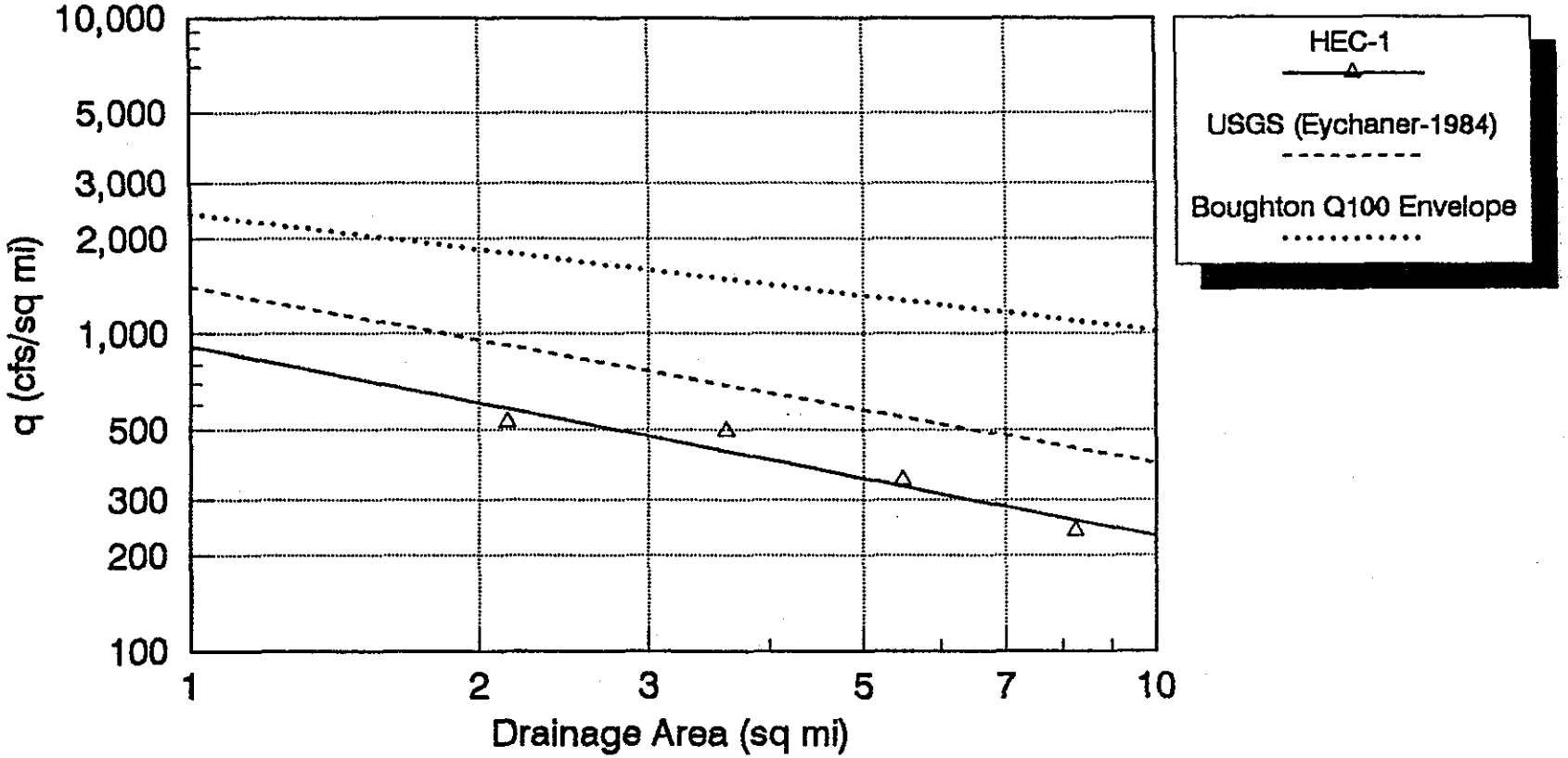


USGS Data from Primary Equations (Table 1)
HEC-1 data based on 24-hour, SCS Type IIA Storm
AMC II

Water Resources Associates, Inc.	UNIT DISCHARGE VS. DRAINAGE AREA 100-YEAR EVENT NON-SHEETFLOW AREAS	FIGURE 3.10
PROJECT NO AC00060 DRAWN BY RLW DATE 9/25/89 CHECKED BY JJH		

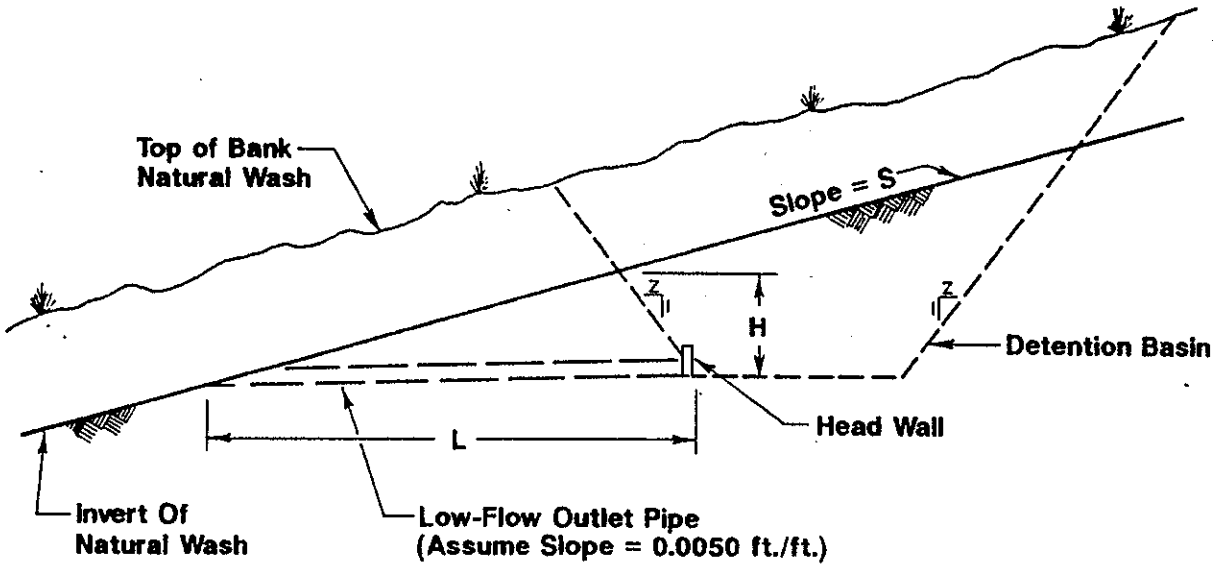
Unit Discharge vs Drainage Area

100-Year Event, Sheet-Flow Areas
City of Phoenix - Areas C & D



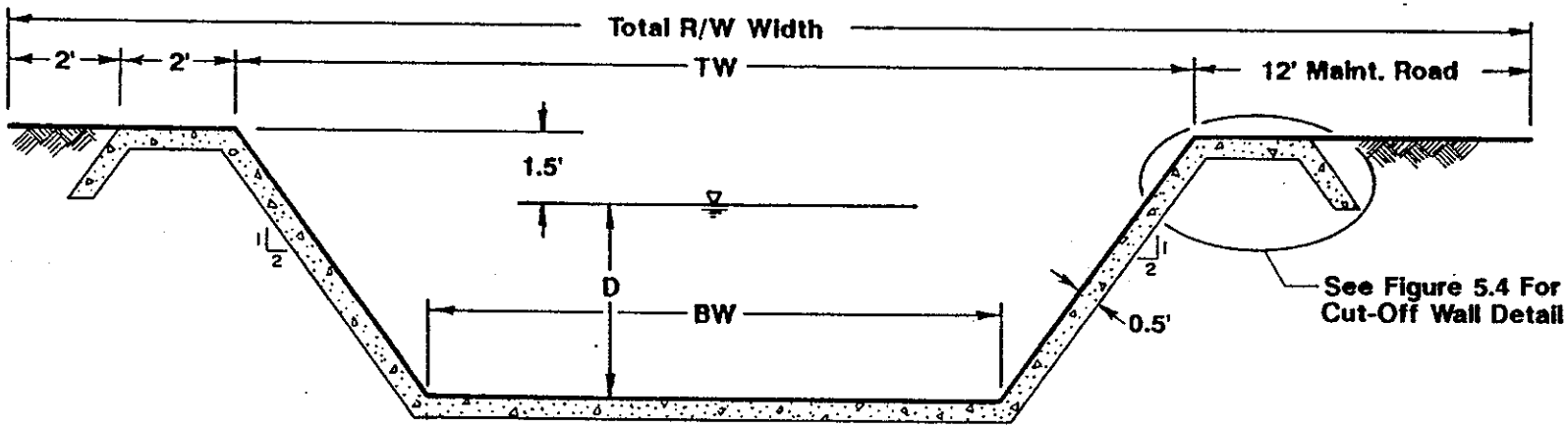
USGS Data from Primary Equations (Table 1)
HEC-1 data based on 24-hour, SCS Type IIA Storm
AMC II

Water Resources Associates, Inc.	UNIT DISCHARGE VS. DRAINAGE AREA 100-YEAR EVENT SHEETFLOW AREAS	FIGURE 3.11
PROJECT NO AC00060 DRAWN BY RLW DATE 9/25/89 CHECKED BY JJH		



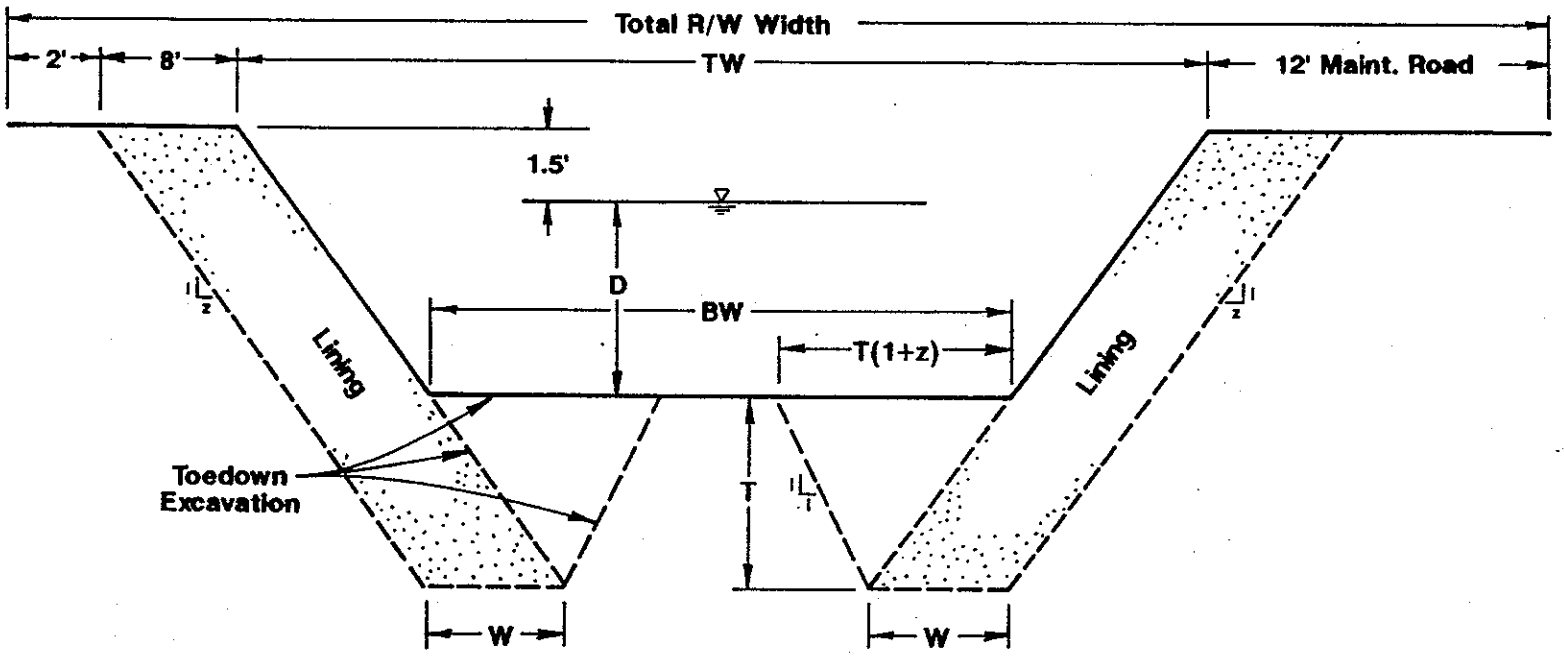
NOT TO SCALE

Water Resources Associates, Inc.	CROSS-SECTION OF TYPICAL DETENTION BASIN OUTLET SYSTEM	FIGURE 4.1
PROJECT NO. AC00060 DRAWN BY KMA		
DATE 9/25/89 CHECKED BY BLW		



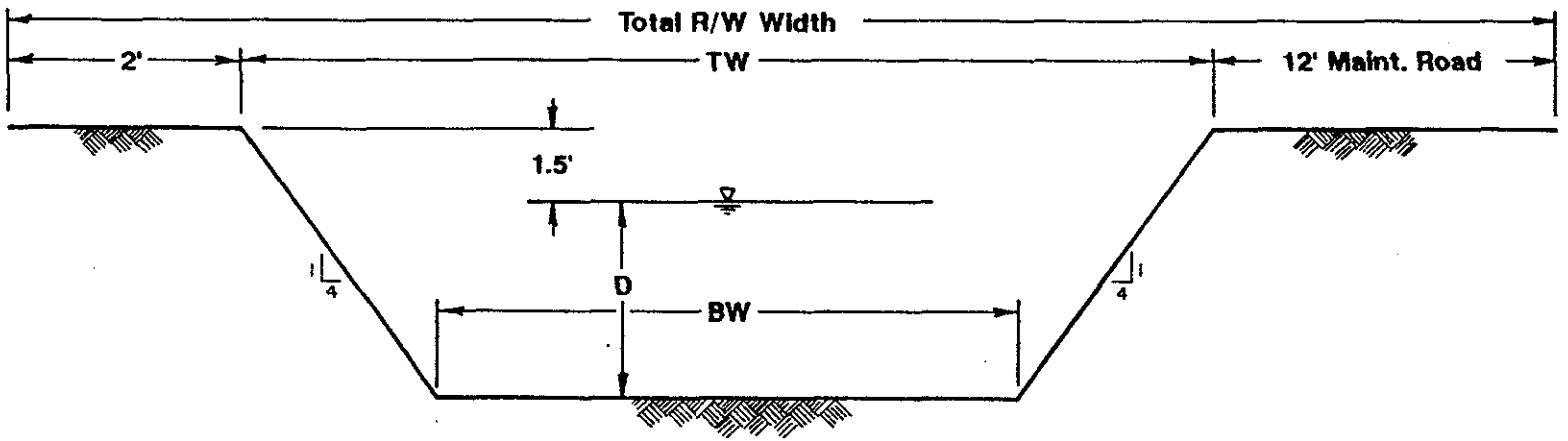
NOT TO SCALE

Water Resources Associates, Inc.	CONCRETE-LINED SECTION	FIGURE 5.1
PROJECT NO AC00060 DRAWN BY KMA DATE 9/6/89 CHECKED BY RLW		



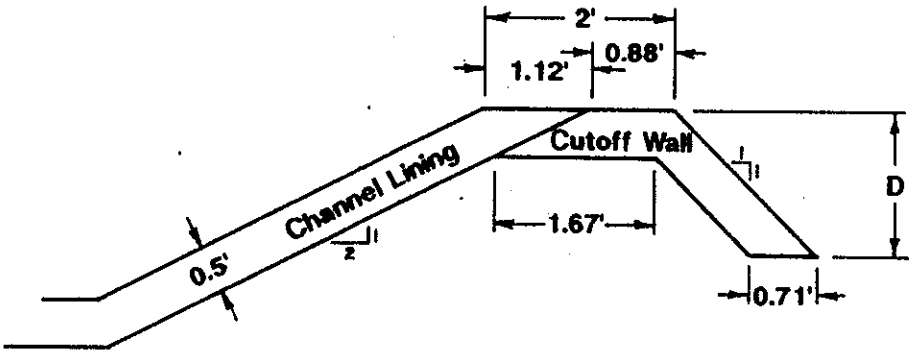
NOT TO SCALE

Water Resources Associates, Inc.	SOIL-CEMENT BANK EARTH BOTTOM CHANNEL SECTION	FIGURE 5.2
PROJECT NO AC00060 / DRAWN BY KMA		
DATE 9/6/89 / CHECKED BY RLW		

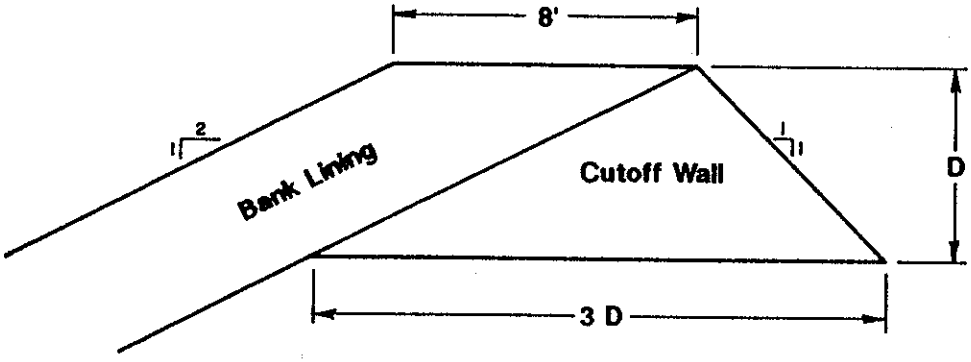


NOT TO SCALE

Water Resources Associates, Inc.	EARTH CHANNEL SECTION	FIGURE 5.3
PROJECT NO. AC00060 DRAWN BY KMA		
DATE 9/6/89 CHECKED BY RLW		

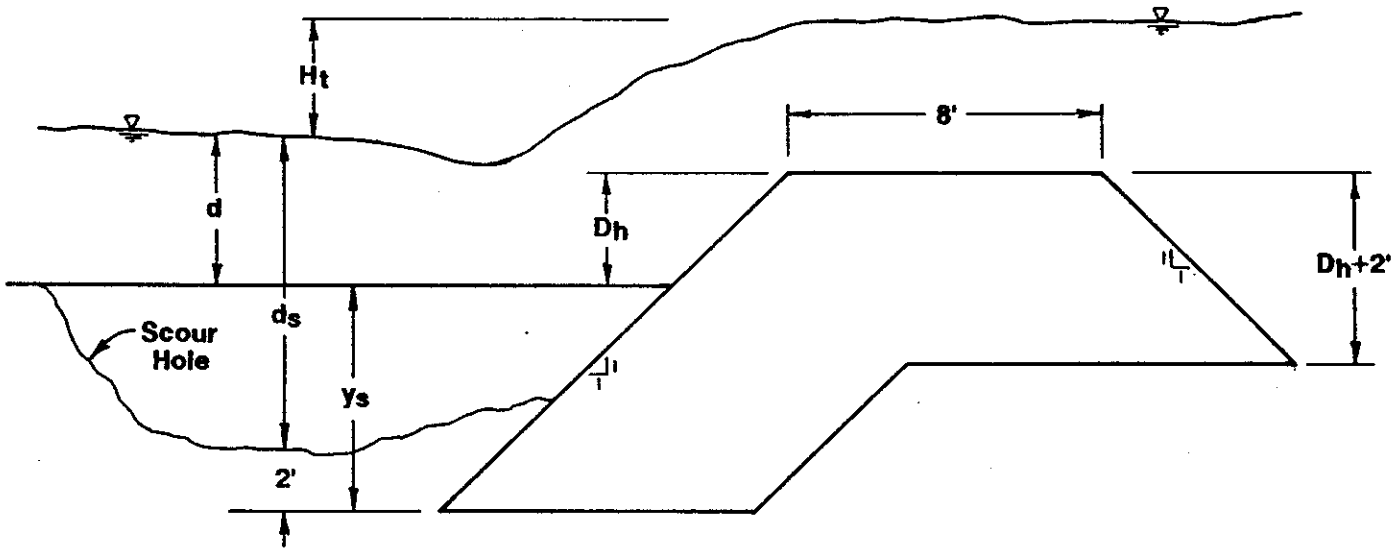


CONCRETE LINING
Not To Scale



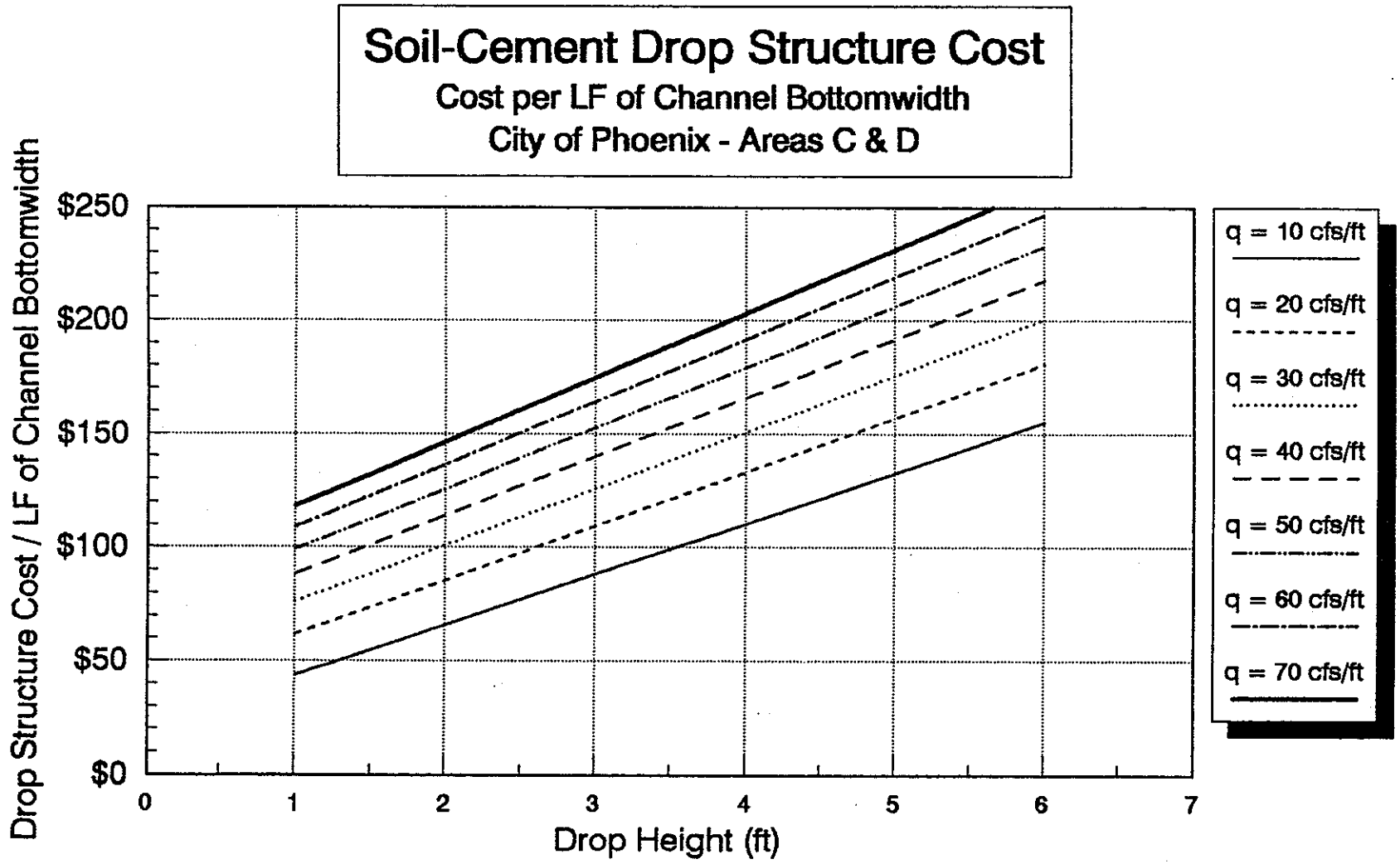
SOIL-CEMENT
Not To Scale

Water Resources Associates, Inc.	CHANNEL LINING CUT-OFF WALL	FIGURE 5.4
PROJECT NO AC00060 DRAWN BY KMA		
DATE 9/6/89 CHECKED BY RLW		



NOT TO SCALE

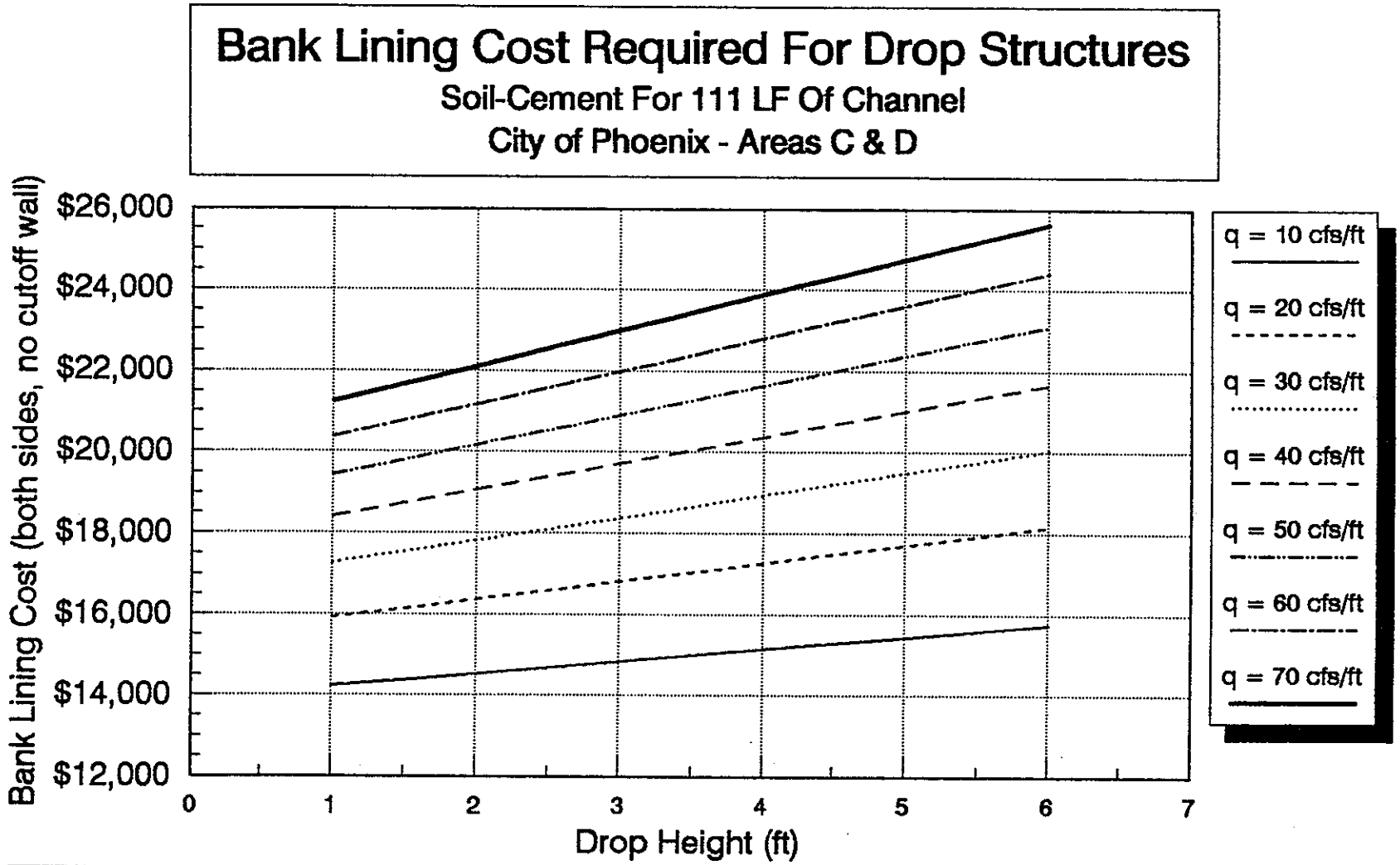
Water Resources Associates, Inc.	TYPICAL SOIL-CEMENT DROP STRUCTURE GEOMETRY	FIGURE 5.5
PROJECT NO AC00060 DRAWN BY KMA		
DATE 9/6/89 CHECKED BY HLW		



Flow depth = 3 ft, FB = 1.5 ft
z = 2H:1V; Costs Do Not Include Bank Lining
Soil Cement @ $27/cy

Figure 5.6

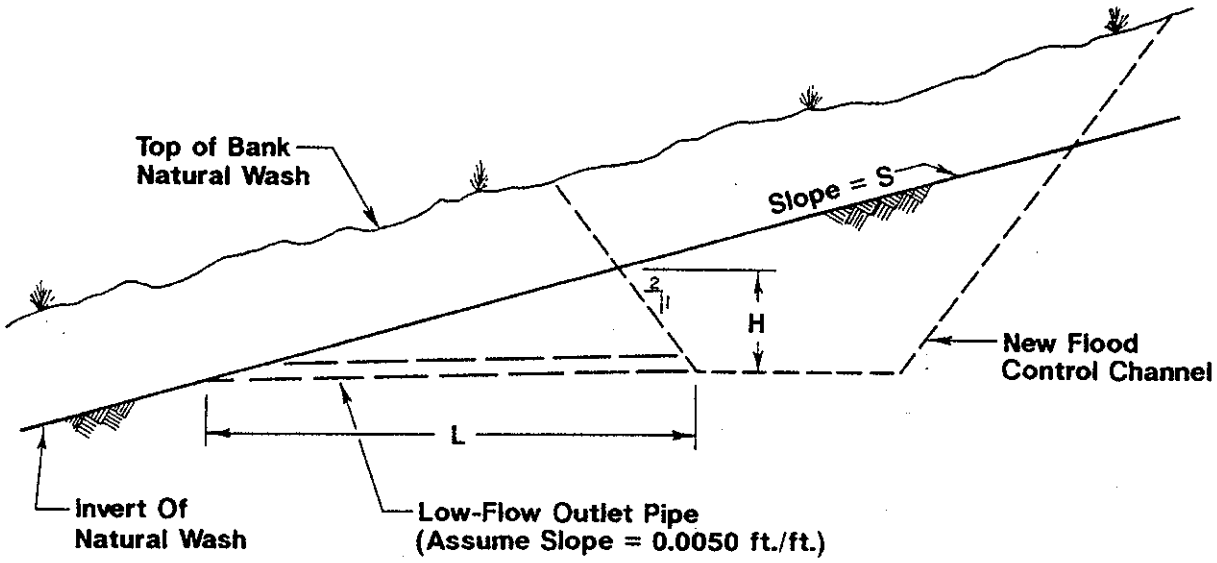
Water Resources Associates, Inc.



Flow depth = 3 ft, FB = 1.5 ft
z = 2H:1V; Soil-Cement @ $24/cy
Scour depth varies with drop height & q(cfs/LF)

Figure 5.7

Water Resources Associates, Inc.



NOT TO SCALE

Water Resources Associates, Inc.	CROSS-SECTION OF LOW-FLOW CULVERT OUTLETS FROM MAN MADE CHANNELS	FIGURE 5.8
PROJECT NO AC00060 DRAWN BY KMA		
DATE 9/25/89 CHECKED BY BLW		

TABLES

TABLE 2.1

SUMMARY OF HYDROLOGIC SOIL GROUP
ASSIGNED TO SOIL COMPLEX/ASSOCIATIONS

Map Symbol	Soil Complex/ Association	Hydrologic Soil Group	Comments
3 & 4	Antho- Carrizo- Maripo Complex	B A B	35% Antho 30% Carrizo 20% Maripo 15% Brios (A), Gilman (B), Vint (B), Denure (B), Momoli (B), Carrizo (A) Use <u>HSG B</u> for Map Symbols 3 & 4.
6 & 7	Anthony- Arizo Complex	B A	40% Anthony 40% Arizo 20% of the group is sandy soils similar to Arizo Use <u>50% HSG B</u> & <u>50% HSG A</u> for Map Symbols 6 & 7.
39	Eba- Nickel- Cave Association	C B D	30% Eba 25% Nickel 25% Cave 20% Rock outcrop Use <u>HSG C</u> for Map Symbol 39.
40,41, 42,43	Eba- Pinaleno Complex	C B	45% Eba 35% Pinaleno 20% Arizo (A), Anthony (B), Contenental (C), Ohaco (C), Greyeagle (D), Nickel (B), Vado (B), Tres Hermanos (B). Use <u>50% HSG B</u> & <u>50% HSG C</u> for Map Symbols 40,41,42,43.

TABLE 2.1 (CONTINUED)

SUMMARY OF HYDROLOGIC SOIL GROUP
ASSIGNED TO SOIL COMPLEX/ASSOCIATIONS

Map Symbol	Soil Complex/ Association	Hydrologic Soil Group	Comments
91 & 92	Momoli- Carrizo Complex	B A	45% Momoli 35% Carrizo 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.
93 & 94	Nickel- Cave Complex	B D	50% Nickel 35% Cave 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 2 is assumed.

TABLE 3.2

SUMMARY OF OVERLAND FLOW ROUGHNESS VALUES
CONCEPT DRAINAGE STUDY
PARADISE VALLEY FAN TERRACE
CITY OF PHOENIX

	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 3.3

SUMMARY OF AREAL RAINFALL VALUES
(in inches)

PARADISE VALLEY FAN TERRACE
CITY OF PHOENIX

Return Interval	Sub-Basin Series 1000 (inches)	Sub-Basin Series 1-53 (inches)	Sub-Basin Series 3000 (inches)
100-year	4.71	4.32	4.22
10-year	3.19	2.92	2.75
2-year	2.13	1.92	1.72

NOTE:

All rainfall values on this page are based on a 24-hour storm duration and a 10 square mile areal reduction factor.

TABLE 3.4

SUMMARY OF SENSITIVITY ANALYSES
FOR RAINFALL PARAMETERS
SUB-BASINS 2240 - 2340
(all discharge values in cfs)

Concentration Point or Sub-Basin No.	Total Basin Area (sq. mi.)	24-Hour Hypothetical, NOAA Rainfall 10 Sq. Mi. *NOAA ARF	24-Hour Hypothetical, NOAA Rainfall 50 Sq. Mi. *NOAA ARF	24-Hour Hypothetical, NOAA/PHX Rainfall 10 Sq. Mi. *NOAA ARF	24-Hour Type IIA, NOAA Rainfall 10 Sq. Mi. NOAA ARF	24-Hour Type IIA, NOAA Rainfall 10 Sq. Mi. HYDRO-40 ARF	24-Hour Type IIA, NOAA Rainfall 50 Sq. Mi. NOAA ARF	24-Hour Type IIA, NOAA Rainfall 50 Sq. Mi. HYDRO-40 ARF	24-Hour PHX Distribution, NOAA Rainfall 10 Sq. Mi. NOAA ARF	24-Hour Type II, NOAA Rainfall 10 Sq. Mi. NOAA ARF	1-Hour Hypothetical NOAA Rainfall 10 Sq. Mi. *NOAA ARF	1-Hour Hypothetical Osborn/Renard Rainfall 10 Sq. Mi. *NOAA ARF
2240	.60	712	505	771	664	535	546	431	665	649	543	811
2260	.26	290	219	313	273	241	246	193	285	262	219	352
2255	.30	565	439	607	491	456	462	400	381	480	512	812
2270	.97	2064	1564	2238	1763	1633	1655	1430	1282	1664	2096	3404
2271	1.27	2619	1969	2844	2249	2075	2104	1830	1663	2144	2573	4142
2250	1.49	2850	2160	3089	2610	2410	2443	2072	1942	2369	2893	4940
2251	1.76	3104	2351	3361	2832	2600	2638	2223	2227	2581	3026	5150
2280	1.94	3111	2447	3359	2859	2644	2682	2265	2392	2648	3345	5433
2290	.60	1332	987	1448	1130	1015	1034	841	812	1009	889	1363
2300	1.80	4814	3427	5179	3975	3722	3764	3123	2592	3560	3252	5122
2302	DIVERT	2407	1713	2589	1987	1861	1882	1562	1296	1780	1626	2561
2310	DIVERT	2536	1928	2701	2229	2046	2066	2040	1671	2001	1630	2592
2320	.37	857	665	943	706	655	664	539	500	700	563	876
2330	.48	1211	937	1299	1015	947	959	843	692	993	1054	1576
2340	.50	960	748	1039	847	777	789	670	630	779	756	1233

* These ARFs were actually computed with the HEC-1 algorithm and applied to each of the rainfall values on the PH card.

TABLE 3.5

SUMMARY OF INDEPENDENT PEAK DISCHARGE CALCULATIONS FOR VERIFICATION OF HEC-1 MODEL RESULTS

Concentration Point	Q_{100} (cfs) HEC-1	USGS[2] Eychaner	Pima Cty. Peak	TR-55 Type II	USGS[1] (Roeske)
3020 D.A. = 0.31 mi.^{2}	433	538	429	295	613 / 1321
3135 D.A. = 0.04 mi.^{2}	66	120	80	39	221 / 406
3150 D.A. = 1.02 mi.^{2}	670	1064	485	552	1111 / 3295
3160 D.A. = 0.40 mi.^{2}	532	639	546	378	696 / 1505
3400 D.A. = 1.07 mi.^{2}	655	1053	551	534	1138 / 3037
3490 D.A. = 5.87 mi.^{2}	2824	2543	1464	2382	2660 / 6384

NOTES:

[1] Region 2 vs. Region 3

[2] Uses Primary Estimating Equations in Table 1 (pg. 6 of 1984 USGS reference)

Mean Annual Precipitation: 16-inches

Average Basin Elevation from Eychaner Calculations

TABLE 4.1

REGRESSION EQUATIONS FOR ESTIMATING
RETENTION BASIN SIZE AND COST

Equation		One Standard Error of Estimate (percent)
Basin Storage Volume		
	$STOR = 10^{-4.900} A^{1.001} (CN)^{3.139} P^{1.624}$	2.6
Basin Surface Area		
Minimum Depth (ft)		
4.0	$AREA = 10^{-4.835} S^{0.176} A^{0.980} (CN)^{3.079} P^{1.593}$	2.5
7.0	$AREA = 10^{-5.005} S^{0.171} A^{0.957} (CN)^{3.028} P^{1.560}$	2.4
10.0	$AREA = 10^{-5.027} S^{0.165} A^{0.930} (CN)^{2.968} P^{1.523}$	2.4
Basin Excavation Cost		
4.0	$COST = 10^{-2.749} S^{0.785} A^{1.351} (CN)^{4.623} P^{2.271} C^{1.045}$	3.6
7.0	$COST = 10^{-3.113} S^{0.586} A^{1.269} (CN)^{4.487} P^{2.155} C^{1.137}$	3.8
10.0	$COST = 10^{-2.909} S^{0.467} A^{1.201} (CN)^{4.237} P^{2.041} C^{1.136}$	3.6
Basin Excavation Volume		
4.0	$EXCV = 10^{-2.705} S^{0.778} A^{1.344} (CN)^{4.605} P^{2.262}$	3.5
7.0	$EXCV = 10^{-2.798} S^{0.567} A^{1.239} (CN)^{4.347} P^{2.112}$	3.9
10.0	$EXCV = 10^{-2.595} S^{0.448} A^{1.171} (CN)^{4.097} P^{1.998}$	3.8

Table 4.1

Summary of Regression Equations for Estimating Retention Basin Size & Cost

STOR = required storage volume (acre-feet), assuming 100% retention of the design storm runoff.

AREA = surface area (acres) of retention basin with 1 foot of Freeboard and 5H:1V sideslopes.

COST = excavation cost (dollars) of retention basin with 1 foot of Freeboard and 5H:1V sideslopes.

EXCV = volume of excavation (cubic yards) required to construct retention basin to achieve required runoff storage (STOR) with 1 foot of Freeboard and 5H:1V sideslopes.

S = ground slope (ft/ft) at retention basin location.

A = drainage area (square miles) contributing runoff to retention basin.

CN = area-weighted curve number (Soil Conservation Service) associated with contributing drainage area (A).

P = total rainfall (inches) for the design event, adjusted for areal reduction, if appropriate.

C = unit cost of excavation (dollars/cubic yard).

TABLE 4.2

SUMMARY OF EXTREME VALUES IN THE DATA SETS USED FOR RETENTION BASIN REGRESSION EQUATIONS

Independent Variable	Extremes	
	Maximum	Minimum
Ground Slope (S)	0.0192 ft/ft	0.0100 ft/ft
Drainage Area (A)	5.94 sq. mi.	0.39 sq. mi.
Curve Number (CN)	86.4	74.0
Rainfall (P)	4.63 in.	2.65 in.
Unit Excavation Cost (C)	$4.00/cy	$2.00/cy

TABLE 5.1

SUMMARY OF CONSTRUCTION COSTS FOR
CUTOFF WALL ALONG TOP OF CHANNEL BANK

Cutoff Wall Depth (ft) (below top of bank)	Cost per Lineal Foot (one side of channel)	
	Concrete	Soil-Cement
1	$ 7.74	$ 1.34
2	12.60	5.33
3	17.28	12.00
4	21.96	21.33

NOTE:

All costs are based on cutoff wall geometry illustrrated in Figure 5.4.
Concrete (6-inches thick) is priced at $180/c.y. (in-place).
Soil-cement is priced at $24/c.y. (in-place).

TABLE 6.1

PEAK DISCHARGE SUMMARY FOR
CAVE CREEK ROAD CHANNEL
100-YEAR, 24-HOUR EVENT

HEC-1 Concentration Point	Peak Discharge (cfs)	
	Existing Land-Use	Future Land-Use
3261	80	143
3023	2199	2252
3134	5826	7159
3173	5838	7240
3243	7779	10,227
3343	9507	12,347
3403	9790	12,837
3453	11,131	14,262
3524	12,712	16,462

TABLE 6.2

REDUCTION IN PEAK DISCHARGE FOR AREAS DOWNSTREAM OF
CAVE CREEK ROAD CHANNEL
100-YEAR, 24-HOUR EVENT

Future Conditions

Concentration Point	Peak Discharge (cfs) Without Channel	With Channel	Percent Reduction
3031	2746	674	75
3060	2702	585	78
3090	3201	688	78
3140	635	629	1 (small basin)
3221	4320	1825	58
3250	1265	594	53
3371	4281	3185	26
3410	1638	755	54
3501 (channel outlet)	10,754	19,810	-84 (increase)

Existing Conditions

3031	2533	557	78
3060	2282	380	83
3090	2561	447	82
3140	372	368	1
3221	2760	838	70
3250	756	276	63
3371	2663	1104	58
3410	812	406	50
3501 (channel outlet)	8112	14,733	- 81 (increase)

TABLE 6.3

SUMMARY OF PEAK DISCHARGE VALUES AND CHANNEL HYDRAULICS
FOR LONE MOUNTAIN, DIXILETA, AND DYNAMITE ROAD CHANNELS
100-YEAR, 24-HOUR EVENT

	Existing Land-Use Conditions						Future Land-Use Conditions					
Concentration Point	Q_{100} (cfs)	BW (ft)	Existing Slope (ft/ft)	Flow Depth (ft)	Velocity (fps)	Froude Number	Q_{100} (cfs)	BW (ft)	Existing Slope (ft/ft)	Flow Depth (ft)	Velocity (fps)	Froude Number
Lone Mountain Road												
3330 A2	2599	43.6	.0175	4.0	12.6	1.19	2982	50.5	.0175	4.0	12.8	1.20
3200 A2	4981	86.1	.0175	4.0	13.2	1.22	5957	103.4	.0175	4.0	13.4	1.22
Dixileta Road												
3420 A2	5201	93.9	.0161	4.0	12.8	1.17	6599	119.7	.01651	4.0	12.9	1.17
3340 A2	5745	103.9	.0161	4.0	12.8	1.17	7301	132.7	.0161	4.0	13.0	1.18
Dynamite Road												
3482	989	15.5	.0154	4.0	10.5	1.07	1494	25.4	.0154	4.0	11.2	1.10
3400 B2	1753	42.6	.0083	4.0	8.7	0.82	2956	73.8	.0083	4.0	9.0	0.83

Notes:

n = 0.035
Z = 2H:IV

TABLE 6.4

REDUCTION IN PEAK DISCHARGE FOR AREAS
DOWNSTREAM OF LONE MOUNTAIN, DIXILETA,
AND DYNAMIDE ROAD CHANNELS
100-YEAR, 24-HOUR EVENT

Existing Land-Use Conditions

Concentration Point (without channel/with channel)	Location	Peak Discharge (cfs) Without Channel	Peak Discharge (cfs) With Channel	Percent Reduction
3330 / 3330B	between Lone Mt. and Dixileta	1230	210	83
3390 / 3390B		380	336	11
3430 / 3430B	between Dixileta and Dynamite	308	46	85
3420 / 3420B		1457	166	89
3470 / 3470B		3052	461	85
3510 / 3510B	along Dynamite	1495	494	67
3480 / 3480		3083	558	82
3440 / 3440A		1820	649	64

TABLE 6.5

SUMMARY OF RETENTION BASIN ANALYSIS
(Retention based on runoff from 100-year, 2-hour rainfall)

					Peak Discharge Summary (100-year, 24-hour Type II A) (cfs)					
						Retention Basin Outflow		Peak Discharge (cfs) Through Next Downstream Sub-Basin		
Model Number	Retention Basin Location	Basin Depth (ft)	Basin Surface Area (ac)	Basin Storage Volume (af)	Retention Basin Inflow	Retention for Phoenix Land Only	Retention for both Phoenix and Scottsdale Lands	Without Upstream Retention	With Retention for Phoenix Land Only	With Upstream Retention for both Phoenix and Scottsdale Lands
T1A.24I	Outlet of Sub. 3200	10	20.68	206.8	4011	N/A	761	Sub. 3210 - 2699	N/A	535
								Sub. 3240 - 1166		234
T1A1.24I	Outlet of Sub. 3200	10	7.53	75.3	4011	4511*	N/A	Sub. 3210 - 2699	2332	N/A
								Sub. 3240 - 1166	966	
T1B.24I	Outlet of Sub. 3660	10	17.47	174.7	1709	N/A	637	Sub. 3670 - 1832	N/A	1686
T1B1.24I	Outlet of Sub. 3660	10	4.98	49.8	1709	1810*	N/A	Sub. 3670 - 1832	1686	N/A
T1C.24I	CP 3200 A2 Lone Mt. Rd. Channel	10	46.96	469.6	5957	N/A	1473	N/A	N/A	N/A
T1C1.24I	CP 3200 A2 Lone Mt. Rd. Channel	10	14.94	149.4	5957	6039*	N/A	N/A	N/A	N/A

* NOTE: In reality, the basin outflow cannot exceed the maximum inflow. This anomaly is occuring because of computational limitations related to the 5-minute time interval

TABLE 6.6

SUMMARY OF DETENTION BASIN ANALYSIS

					Peak Discharge Summary (100-year, 24-hour Type II A) (cfs)					
						Detention Basin Outflow		Peak Discharge (cfs) Through Next Downstream Sub-Basin		
Model Number	Detention Basin Location	Basin Depth (ft)	Basin Surface Area (ac)	Basin Storage Volume (af)	Detention Basin Inflow	Detention for Phoenix Land Only	Detention for both Phoenix and Scotts-dale Lands	Without Upstream Detention	With Detention for Phoenix Land Only	With Upstream Detention for both Phoenix and Scottsdale Lands
T2A.24I	Outlet of Sub. 3200	10	20.68	206.8[1]	4011	N/A	549[2]	Sub. 3210 - 2699	N/A	391
								Sub. 3240 - 1166		171
T2A1.24I	Outlet of Sub. 3200	10	7.53	75.3[1]	4011	3770[3]	N/A	Sub. 3210 - 2699	2222	N/A
								Sub. 3240 - 1166	969	
T2A2.24I	Outlet of Sub. 3200	9.32[4]	27.57	254	4011	N/A	130	Sub. 3210 - 2699	N/A	197
								Sub. 3240 - 1166		133
T2C.24I	Cave Creek Rd. & Lone Mt. Rd.	6.51[5]	90.17	587	5957	N/A	303	Not Evaluated		

NOTES:
1. Based on direct runoff from the 100-year, 2-hour rainfall.
2. Includes low-flow discharge of 69.5 cfs.
3. Includes low-flow discharge of 25.3 cfs.
4. Maximum reservoir depth during flood routing, also reflects constant low-flow discharge of 129.7 cfs.
5. Maximum reservoir depth during flood routing, also reflects constant low-flow discharge of 303 cfs.

TABLE 6.7

PEAK DISCHARGE SUMMARY FOR ORIGINAL EAST-WEST DESERT RIDGE CHANNEL, BEGINNING AT JOMAX ROAD

HEC-1 Concentration Point	Comparison of Q_{100} (cfs) With and Without Low-Flow Diverts		
	With Divert	Without Divert	With No Channel
8.2	502	752	N/A
5.3	1856	2527	N/A
3668	2546	3846	N/A
3644	2782	4478	N/A
3653	2910	4616	N/A
3655A	2899	4633	N/A
3659	2892	4621	N/A
8.5	509	380	-
9	996	934	1118
10	1334	1295	1388
10.1	1502	1484	1527
5.5	1222	1218	1900 (Sub.5)
6	1440	1385	1971
6.1	1509	1462	2014
3675	1084	768	1428 (Sub.3670)
3685	1701	1576	1710 (Sub.3680)
3690	1986	1861	2006
3649	509	387	-
3657	696	610	-

TABLE 6.8

PEAK DISCHARGE SUMMARY FOR EAST-WEST DESERT RIDGE ALONG ALONG PINNACLE PEAK ROAD

HEC-1 Concentration Point	Comparison of Q_{100} (cfs) With and Without Low-Flow Diverts		
	With Divert	Without Divert	With No Channel
9.1	780	1103	N/A
6.6	2273	2972	N/A
3683	3314	4465	N/A
3643	3906	5352	N/A
3655	3955	5403	N/A
3659	3954	5401	N/A
10	1018*(1445)	1128*(1256)	1388
10.1	1344 (1602)	1392 (1581)	1527
6	1070*(1544)	1160*(1315)	1971
6.1	1201*(1606)	1287*(1426)	2014
3685	1309	929	1710 (Sub.3680)
3690	1550	1366	2006
3657	553	550	N/A

* These results are physically impossible, i.e., the peak downstream discharge cannot increase if the low-flow diverts are cut-off, the peaks would have to go down. This is apparently caused by some kind of error in the kinematic wave algorithm contained in the 1985 version of HEC-1. The input data is identical for both the "with and without" low-flow divert models.

The numbers in parentheses represent Q_p at the same location when the models were re-run under the 1988 version of HEC-1. Although the overall peaks are higher, the termination of low-flow diverts causes a reduction in downstream peaks, as would be expected.

TABLE 6.9

PEAK DISCHARGE SUMMARY
FOR NORTH-SOUTH DESERT RIDGE CHANNEL

HEC-1 Concentration Point	Comparison of Q_{100} (cfs) With and Without Low-Flow Diverts		
	With Divert	Without Divert	With No Channel
12.2	775	1106	N/A
28A3	10,636	10,986	N/A
33A3	11,775	13,069	N/A
33.4	10,691	13,012	N/A
13	755*	800*	1278
14	868	857	1238
14.1	919	903	1263
28	1060	761	8731
29	1292	1238	6577
29.1	1562	1531	6360
33	1116	897	2089
33.1	1310	1298	2162

* These results are physically impossible, i.e., the peak downstream discharge cannot increase if the low-flow diverts are cut-off, the peaks would have to go down. This is apparently caused by some kind of error in the kinematic wave algorithm contained in the 1985 version of HEC-1. The input data is identical for both the "with and without" low-flow divert models.

The numbers in parentheses represent Q_p at the same location when the models were re-run under the 1988 version of HEC-1. Although the overall peaks are higher, the termination of low-flow diverts causes a reduction in downstream peaks, as would be expected.

TABLE 6.10

SUMMARY OF HYDRAULIC PARAMETERS
FOR DESERT RIDGE CHANNEL ANALYSIS

Channel Alternative	Flow Depth (ft)	Channel Depth with Freeboard (ft)	Side-Slope	Bed Slope (ft/ft)	Scour Depth (ft)	Maximum Velocity (fps)	Manning's Roughness
1. Complete Concrete Lining	4.0	5.5	2H:1V	Existing Ground	N/A	20 ±	.020
2. Soil Cement Banks, Earth Bottom	4.0	5.5	2H:1V	Existing Ground	5.0	20 ±	.025 - .035
3. Earth Banks and Bottom	3.0	4.5	4H:1V	.0067	N/A	5	.050

TABLE 6.11

SUMMARY OF CHANNEL COSTS, NO LOW-FLOW CULVERTS
DESERT RIDGE
ORIGINAL EAST-WEST ALIGNMENT BEGINNING AT JOMAX ROAD

Construction Item	Total Concrete Lining	Soil-Cement Banks Earth Bottom n = .025	Soil-Cement Banks Earth Bottom n = .035	Unlined (Earth) Channel
1. Excavation	$ 1,433,000	$1,560,000	$1,925,000	$4,489,000
2. Lining Material	8,913,000	5,734,000	5,734,000	-0-
3. Cutoff Wall (2'U/S, 1'DS)	781,000	256,000	256,000	-0-
4. Drop Structures	-0-	-0-	-0-	2,898,000
5. Landscaping (@ $7,500/acre)	535,000	658,000	812,000	1,991,000
Sub-Total	11,662,000	8,208,000	8,727,000	9,378,000
6. Engineering, design, & inspection at 10% of Items 1 through 4	1,166,000	821,000	873,000	938,000
7. GRAND TOTAL:	$12,828,000	$9,029,000	$9,600,000	$10,316,000
* Right-of-Way Requirements (acres)	71.3	87.7	108.2	265.4

* Based on right-of-way width shown on Figures 5.1, 5.2 and 5.3

TABLE 6.12

SUMMARY OF CHANNEL COSTS, WITH LOW-FLOW CULVERTS
DESERT RIDGE
ORIGINAL EAST-WEST ALIGNMENT BEGINNING AT JOMAX ROAD

Construction Item	Total Concrete Lining	Soil-Cement Banks Earth Bottom n = .035	Unlined (Earth) Channel
1. Excavation	$ 1,005,000	$ 1,472,000	$3,025,000
2. Lining Material	6,772,000	5,734,000	-0-
3. Cutoff Wall (2'U/S, 1'D/S)	781,000	256,000	-0-
4. Drop Structures	-0-	-0-	2,334,000
5. Low-Flow Culverts	3,190,000	3,190,000	3,190,000
6. Landscaping (@ $7,500/acre)	424,000	620,000	1,424,000
Sub-Total	12,172,000	11,272,000	9,973,000
7. Engineering, design, and inspection at 10% of Items 1 through 5	1,217,000	1,127,000	997,000
8. GRAND TOTAL:	$13,389,000	$12,399,000	$10,970,000
* Right-of-Way Requirements (acres)	56.5	82.7	189.8

* Based on right-of-way width shown on Figures 5.1, 5.2 and 5.3

TABLE 6.13

SUMMARY OF CHANNEL COSTS
DESERT RIDGE
EAST/WEST PINNACLE PEAK ROAD ALIGNMENT

Construction Item	Soil-Cement Banks, Earth Bottom, n = .035	
	Without Low-Flow Culverts	With Low-Flow Culverts
1. Excavation	$1,779,000	$1,447,000
2. Lining Material	4,468,000	4,468,000
3. Cutoff Wall	200,000	200,000
4. Drop Structures	-0-	-0-
5. Low-Flow Culverts	-0-	2,320,000
6. Landscaping* (@ $7,500/acre)	751,000	611,000
Sub-Total	7,198,000	9,046,000
7. Engineering, design, and inspection at 10% of Items 1 through 5	720,000	905,000
8. **GRAND TOTAL:**	**$7,918,000**	**$9,951,000**
* Right-of-Way Requirements (acres)	100.1	81.4

* Based on right-of-way width shown on Figures 5.1, 5.2 and 5.3

TABLE 6.14

SUMMARY OF CHANNEL COSTS, NO LOW-FLOW CULVERTS
DESERT RIDGE
NORTH-SOUTH ALIGNMENT

Construction Item	Total Concrete Lining	Soil-Cement Banks Earth Bottom n = .025	Soil-Cement Banks Earth Bottom n = .035	Unlined (Earth) Channel
1. Excavation	$ 1,604,000	$2,307,000	$3,013,000	$ 9,429,000
2. Lining Material	14,036,000	4,729,000	4,729,000	-0-
3. Cutoff wall (2'U/S, 1'D/S)	644,000	211,000	211,000	-0-
4. Drop structures	-0-	-0-	-0-	7,206,000
5. Landscaping (@ $7,500/acre)	786,000	974,000	1,272,000	3,860,000
Sub-Total	17,070,000	8,221,000	9,225,000	20,495,000
6. Engineering, design and inspection at 10% of Items 1 through 4	1,707,000	822,000	923,000	2,050,000
7. GRAND TOTAL:	$18,777,000	$9,043,000	$10,148,000	$22,545,000
* Right-of-Way Requirements (acres)	104.8	129.8	169.6	514.7

* Based on right-of-way width shown on Figures 5.1, 5.2 and 5.3

TABLE 6.15

SUMMARY OF CHANNEL COSTS, WITH LOW-FLOW CULVERTS
DESERT RIDGE
NORTH-SOUTH ALIGNMENT

Construction Item	Total Concrete Lining	Soil-Cement Banks Earth Bottom n= .035	Unlined (Earth) Channel
1. Excavation	$ 1,488,000	$2,817,000	$ 8,717,000
2. Lining Material	13,114,000	4,729,000	-0-
3. Cutoff Wall	644,000	211,000	-0-
4. Drop Structures	-0-	-0-	6,835,000
5. Low-Flow Culverts	2,481,000	2,481,000	2,481,000
6. Landscaping (@ $7,500/acre)	739,000	1,189,000	3,584,000
Sub-Total	18,466,000	11,427,000	21,617,000
7. Engineering, Design, and Inspection at 10% of Items 1 through 5	1,847,000	1,143,000	2,162,000
8. GRAND TOTAL:	**$20,313,000**	**$12,570,000**	**$23,779,000**
* Right-of-Way Requirements (acres)	98.5	158.5	477.9

* Based on right-of-way width shown on Figures 5.1, 5.2 and 5.3

TABLE 6.16

SUMMARY OF TOTAL CHANNEL CONSTRUCTION COSTS
DESERT RIDGE CHANNEL ALIGNMENTS

Channel Alignment	Full Concrete Lining	Soil-Cement Bank Section n = .025	Soil-Cement Bank Section n = .035	Earth Section
With Low-Flow Culverts				
North-South Channel Near Scottsdale Rd.	$20,313,000	Not Evaluated	$12,570,000	$23,779,000
Original East-West Alignment, Starting at Jomax Road	13,389,000	Not Evaluated	12,399,000	10,970,000
Alternate East-West Alignment Along Pinnacle Peak Road	N/A	N/A	9,951,000	N/A
Without Low-Flow Culverts				
North-South Channel Near Scottsdale Rd.	$18,777,000	$9,043,000	$10,148,000	$22,545,000
Original East-West Alignment, Starting at Jomax Road	12,828,000	9,029,000	9,600,000	10,316,000
Alternate East-West Alignment Along Pinnacle Peak Road	N/A	N/A	7,918,000	N/A

NOTE: Costs do not include right-of-way.
Costs do include allowances for final design and engineering, landscaping and construction inspection.

TABLE 6.17

CHANNEL RIGHT-OF-WAY REQUIREMENTS
CONCRETE LINED SECTION (n = .020)
100-YEAR, 24-HOUR EVENT

RIGHT-OF-WAY REQUIREMENTS (ACRES)

Channel Right of Way Option	Without Low-Flow Culverts			With Low-Flow Culverts		
	N-S Channel	Original E-W Channel Alignment	Pinnacle Peak Road E-W Channel Alignment	N-S Channel	Original E-W Channel Alignment	Pinnacle Peak Road E-W Channel Alignment
1. 200-foot wide corridor for entire channel length.	145.4	176.3	N/A	145.4	176.3	N/A
2. Right-of-Way width per dimensions shown on Figures 5.1, 5.2, or 5.3	104.8	71.3	N/A	98.5	56.5	N/A
3. Right-of-Way width per dimensions shown on Figures 5.1, 5.2, or 5.3 plus an additional 50 feet	141.2	115.4	N/A	134.9	100.6	N/A

TABLE 6.18

RIGHT-OF-WAY SUMMARY
FOR DESERT RIDGE CHANNELS
SOIL-CEMENT SECTION (n = .035)
100-YEAR, 24-HOUR EVENT

Channel Right of Way Option	RIGHT-OF-WAY REQUIREMENTS (ACRES) Without Low-Flow Culverts: N-S Channel	Original E-W Channel Alignment	Pinnacle Peak Road E-W Channel Alignment	With Low-Flow Culverts: N-S Channel	Original E-W Channel Alignment	Pinnacle Peak Road E-W Channel Alignment
1. 200-foot wide corridor for entire channel length	145.4	176.3	137.4	145.4	176.3	137.4
2. Right-of-Way width per dimensions shown on Figures 5.1, 5.2, or 5.3	169.6	108.2	100.1	158.5	82.7	81.4
3. Right-of-Way width per dimensions shown on Figures 5.1, 5.2, or 5.3 plus an additional 50 feet	206.0	152.3	134.4	194.9	126.8	115.7

TABLE 6.19

RIGHT-OF-WAY SUMMARY
FOR DESERT RIDGE CHANNELS
EARTH CHANNEL SECTION (n = .050)
100-YEAR, 24-HOUR EVENT

Channel Right of Way Option	RIGHT-OF-WAY REQUIREMENTS (ACRES) Without Low-Flow Culverts: N-S Channel	Original E-W Channel Alignment	Pinnacle Peak Road E-W Channel Alignment	With Low-Flow Culverts: N-S Channel	Original E-W Channel Alignment	Pinnacle Peak Road E-W Channel Alignment
1. 200-foot wide corridor for entire channel length	145.4	176.3	N/A	145.4	176.3	N/A
2. Right-of-Way width per dimensions shown on Figures 5.1, 5.2, or 5.3	514.7	265.4	N/A	477.9	189.8	N/A
3. Right-of-Way width per dimensions shown on Figures 5.1, 5.2, or 5.3 plus an additional 50 feet	551.1	309.5	N/A	514.3	233.9	N/A

TABLE 7.1

COST FOR MAINLINE PIPE
STORM DRAINS
COST PER LIN. FT.

	DEPTH TO SUBGRADE-FEET								
PIPE SIZE IN.	6	7	8	9	10	11	12	14	PIPE SIZE IN.
12	50.44	52.38	53.35	55.29	56.26	57.23	58.20	60.14	12
15	52.38	53.35	54.32	56.26	58.20	59.17	60.14	62.08	15
18	55.29	56.26	58.20	60.14	61.11	61.11	63.05	64.99	18
21	56.26	57.23	59.17	61.11	62.08	63.05	65.96	68.87	21
24	59.17	60.14	62.08	63.05	64.02	65.96	68.87	72.75	24
27	62.08	63.05	64.99	66.93	67.90	69.84	71.78	76.63	27
30	66.93	67.90	69.84	70.81	72.75	75.66	78.57	83.42	30
33	71.78	73.72	75.66	77.60	79.54	84.39	86.33	91.18	33
36	78.57	80.51	83.42	85.36	87.30	90.21	92.15	97.97	36
39	87.30	89.24	92.15	94.09	96.03	98.94	101.85	106.70	39
42	96.03	97.97	100.88	102.82	105.73	108.64	111.55	117.37	42
45	104.76	106.70	109.61	111.55	114.46	117.37	121.25	127.07	45
48	113.49	115.43	118.34	121.25	124.16	127.07	130.95	136.77	48
51	122.22	125.13	128.04	130.95	133.86	136.77	140.65	146.47	51
54	130.95	133.86	137.74	140.65	143.56	146.47	150.35	157.14	54
57	139.68	142.59	146.47	149.38	153.26	156.17	160.05	166.84	57
60	149.38	153.26	157.14	160.05	163.93	166.84	169.75	177.51	60
63		162.96	166.84	170.72	174.60	178.48	182.36	189.15	63
66		172.66	177.51	181.39	185.27	189.15	193.03	199.82	66
69		186.24	191.09	195.94	199.82	202.73	206.61	212.43	69
72			199.82	203.70	208.55	212.43	216.31	224.07	72
75			211.46	215.34	220.19	224.07	228.92	236.68	75
78			224.07	227.95	232.80	237.65	242.50	251.23	78
81			237.65	241.53	246.38	251.23	257.05	266.75	81
84				257.05	261.90	266.75	271.60	282.27	84
87				272.57	278.39	284.21	291.00	302.64	87
90				288.09	293.91	300.70	308.46	322.04	90
93					310.40	319.13	327.86	344.35	93
96					329.80	336.59	343.38	356.96	96
102						356.96	365.69	382.18	102
108						376.36	385.09	401.58	108
114							402.55	419.04	114
120							422.92	442.32	120

TABLE 7.1 (continued)

COST FOR MAINLINE PIPE
STORM DRAINS
COST PER LIN. FT.

PIPE SIZE IN.	DEPTH TO SUBGRADE-FEET 14	16	18	20	22	24	26	28	PIPE SIZE IN.
24	72.75	78.57	84.39	90.21					24
27	76.63	82.45	88.27	93.12	97.97				27
30	83.42	89.24	94.09	98.94	103.79	108.64			30
33	91.18	95.06	100.88	106.70	115.43	123.19	130.95		33
36	97.97	103.79	109.61	116.40	122.22	131.92	141.62	155.17	36
39	106.70	113.49	120.28	128.04	135.80	143.55	155.17	168.78	39
42	117.37	124.16	130.95	138.71	147.44	155.20	167.81	180.42	42
45	127.07	134.83	141.62	149.38	158.11	166.84	180.42	194.00	45
48	136.77	145.50	154.23	161.99	170.72	179.45	193.03	206.51	48
51	146.47	155.20	163.93	172.66	179.45	192.06	205.64	219.22	51
54	157.14	162.95	172.66	185.27	194.97	203.70	218.25	232.80	54
57	166.84	178.48	190.12	200.79	208.55	216.31	230.86	245.41	57
60	177.51	186.24	195.91	208.55	218.25	228.92	247.47	258.99	60
63	193.03	199.82	210.49	221.16	231.83	242.50	257.05	272.57	63
66	199.82	211.46	222.13	233.77	244.44	255.08	271.60	287.12	66
69	212.43	224.07	235.71	247.35	256.08	269.66	286.15	302.64	69
72	224.07	236.68	249.29	260.93	272.57	286.15	302.64	318.16	72
75	236.68	249.29	261.90	273.54	288.09	300.70	318.16	334.65	75
78	251.23	263.84	277.42	290.03	303.61	316.22	334.65	352.11	78
81	266.75	279.35	292.94	305.55	320.10	333.68	352.11	371.51	81
84	282.27	293.91	306.52	322.04	335.62	352.11	370.54	391.88	84
87	302.54	316.22	325.92	343.38	359.87	376.36	397.70	419.04	87
90	322.04	337.55	353.08	368.60	385.09	401.58	423.89	446.20	90
93	344.35	360.84	377.33	393.82	414.19	433.59	455.90	477.24	93
96	356.96	375.39	393.82	413.22	434.56	455.90	480.15	504.40	96
102	382.18	401.58	420.98	441.35	463.66	487.91	515.07	544.17	102
108	401.58	420.98	432.62	459.78	484.03	508.28	538.35	569.39	108
114	419.04	430.68	451.05	479.18	503.43	531.56	562.60	593.64	114
120	442.32	457.84	479.18	503.43	529.62	558.72	589.76	620.80	120

TABLE 7.2

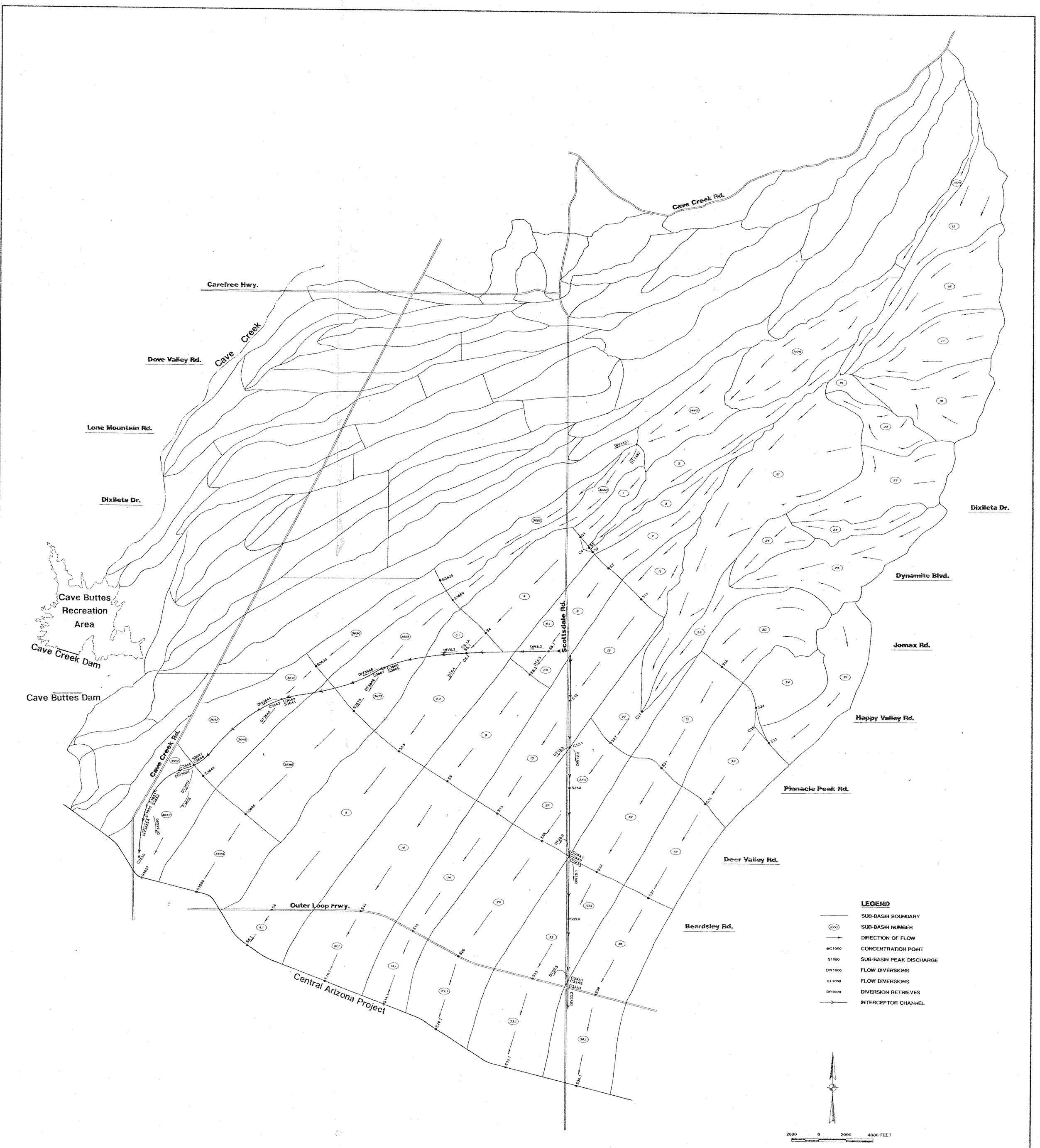
SUMMARY OF CONSTRUCTION COST ESTIMATES
FOR STORM DRAINS

PIPELINE	30" LENGTH	INVERT DEPTH	COST	3' LENGTH	INVERT DEPTH	COST	4' LENTH	INVERT DEPTH	COST	5' LENGTH	INVERT DEPTH	COST	TOTAL COST
5000	19,700	8	$ 1,375,800	-	-	-	6,100	16	$ 887,600	-	-	-	$ 2,263,400
5020	37,100	8	2,591,100	6,760	14	$ 662,300	5,300	16	771,200	11,600	16	$ 2,160,400	6,185,000
5080	40,400	8	2,821,500	10,700	14	1,048,300	14,600	16	2,124,300	-	-	-	5,994,100
5150	37,700	8	2,633,000	17,400	14	1,704,700	5,300	16	771,200	6,000	16	1,117,400	6,226,300
5230	51,600	8	3,603,700	23,400	14	2,292,500	12,300	16	1,789,700	4,500	16	838,100	8,524,000
6000	63,100	8	4,406,900	23,800	14	2,331,700	5,300	16	771,200	15,300	16	2,849,500	10,359,300
6100	45,400	8	3,170,700	25,600	14	2,508,000	10,300	16	1,498,700	2,500	16	456,600	7,643,000
6200	58,600	8	4,092,600	22,900	14	2,243,500	8,800	16	1,280,400	7,000	16	1,303,700	8,920,200
6300	22,400	8	1,564,400	25,200	14	2,468,800	-	-	-	-	-	-	4,033,200
6400	31,600	8	2,206,900	22,500	14	2,204,300	7,500	16	1,091,300	-	-	-	5,502,500
6500	19,200	8	1,340,900	17,600	14	1,724,300	-	-	-	-	-	-	3,065,200
6600	21,200	8	1,480,600	12,000	14	1,175,600	-	-	-	-	-	-	2,656,200
6700	22,000	8	1,536,500	18,100	14	1,773,300	-	-		-	-		3,309,800
			32,824,600			22,137,300			10,985,600			8,734,700	74,682,200

Note: Mainline storm drain top of pipe elevations located approximately 10 feet below natural ground surface to minimize utility conflicts (per City of Phoenix personnel)

30" laterals top of pipe elevation located approximately 5 feet below natural ground

PLATES



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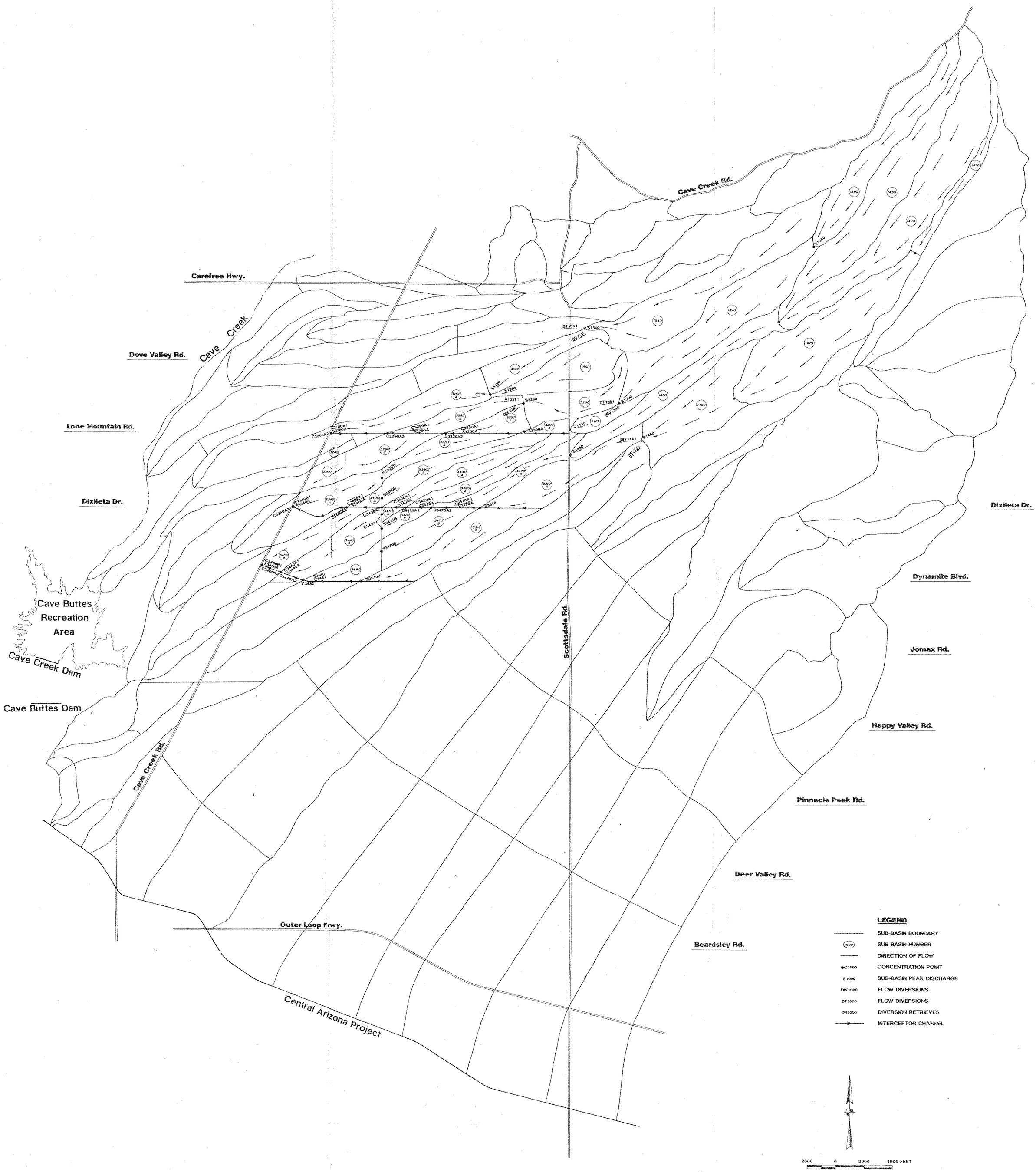
DATE 10/2/89 | CHECKED BY RLW

HEC-1 ROUTING SCHEMATIC
DESERT RIDGE INTERCEPTOR CHANNELS
JOMAX & SCOTTSDALE ROAD ALIGNMENTS

PLATE 6A



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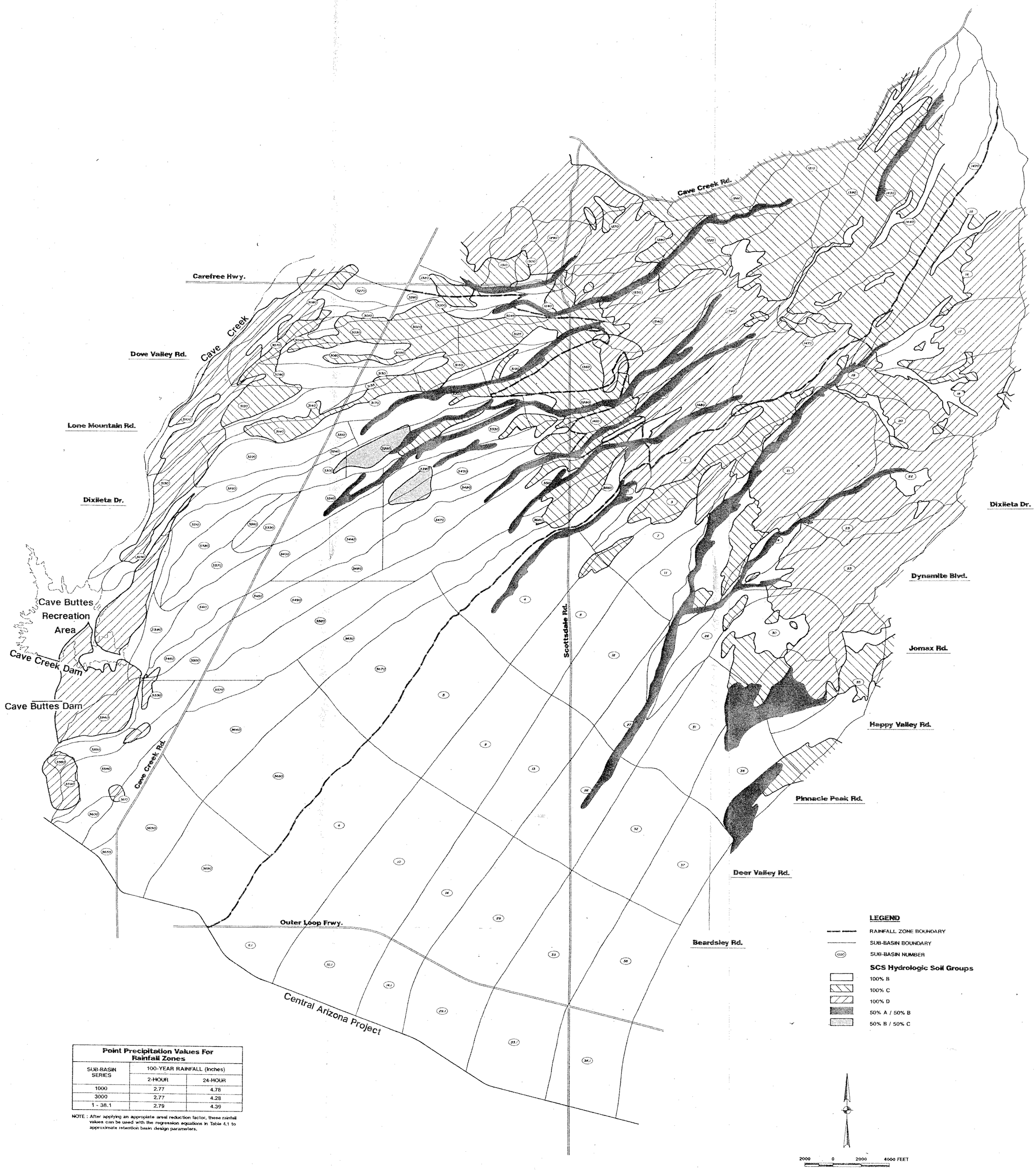
HEC-1 ROUTING SCHEMATIC FOR LONE MOUNTAIN, DIXILETA, AND DYNAMITE ROAD CHANNELS

PLATE 5



Point Precipitation Values For Rainfall Zones

SUB-BASIN SERIES	100-YEAR RAINFALL (Inches)	
	2-HOUR	24-HOUR
1000	2.77	4.78
3000	2.77	4.28
1 - 38.1	2.79	4.39

NOTE : After applying an appropiate areal reduction factor, these rainfall values can be used with the regression equations in Table 4.1 to approximate retention basin design parameters.

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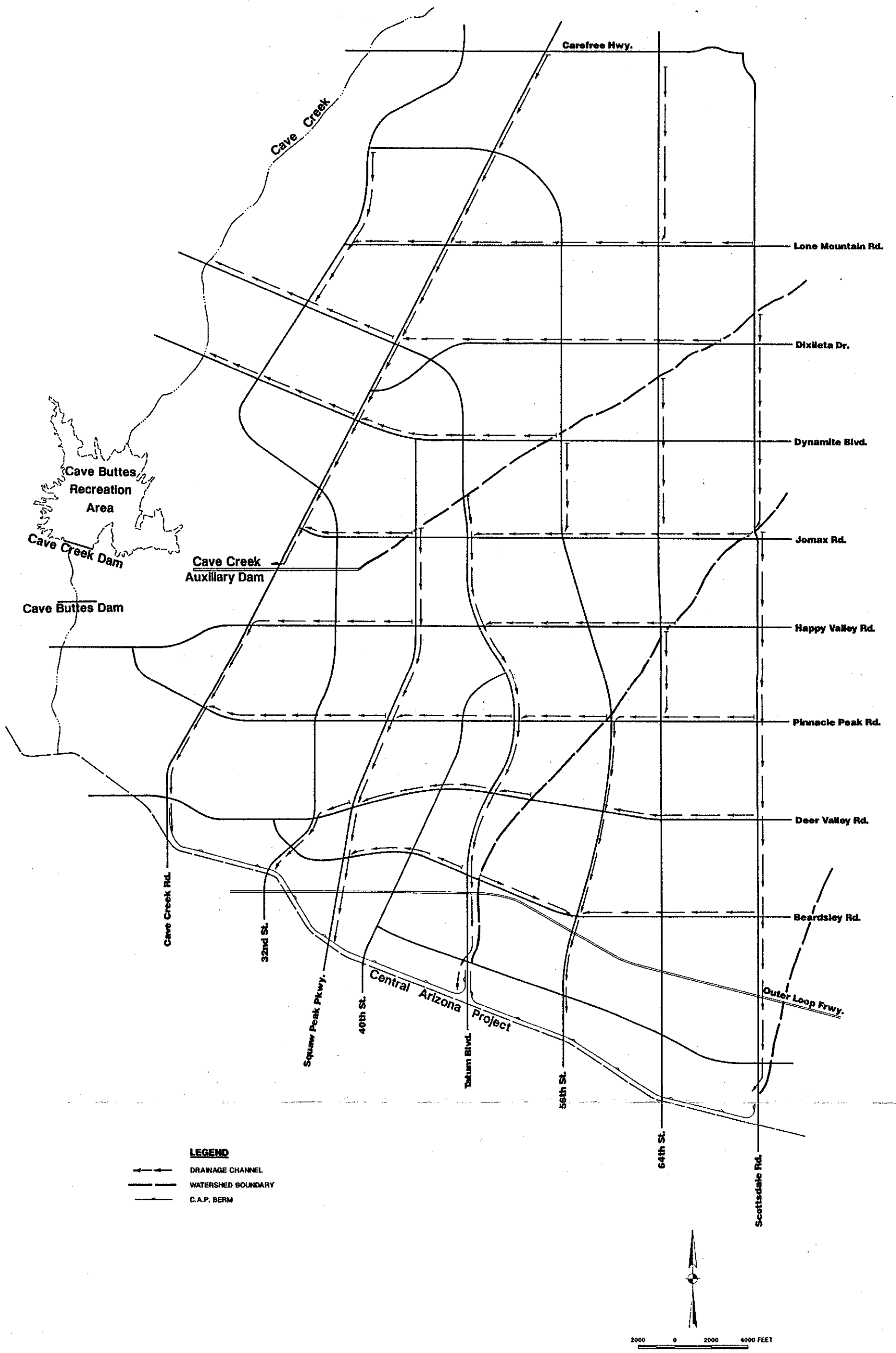
HYDROLOGIC SOIL GROUP AND RAINFALL ZONES

PLATE 1



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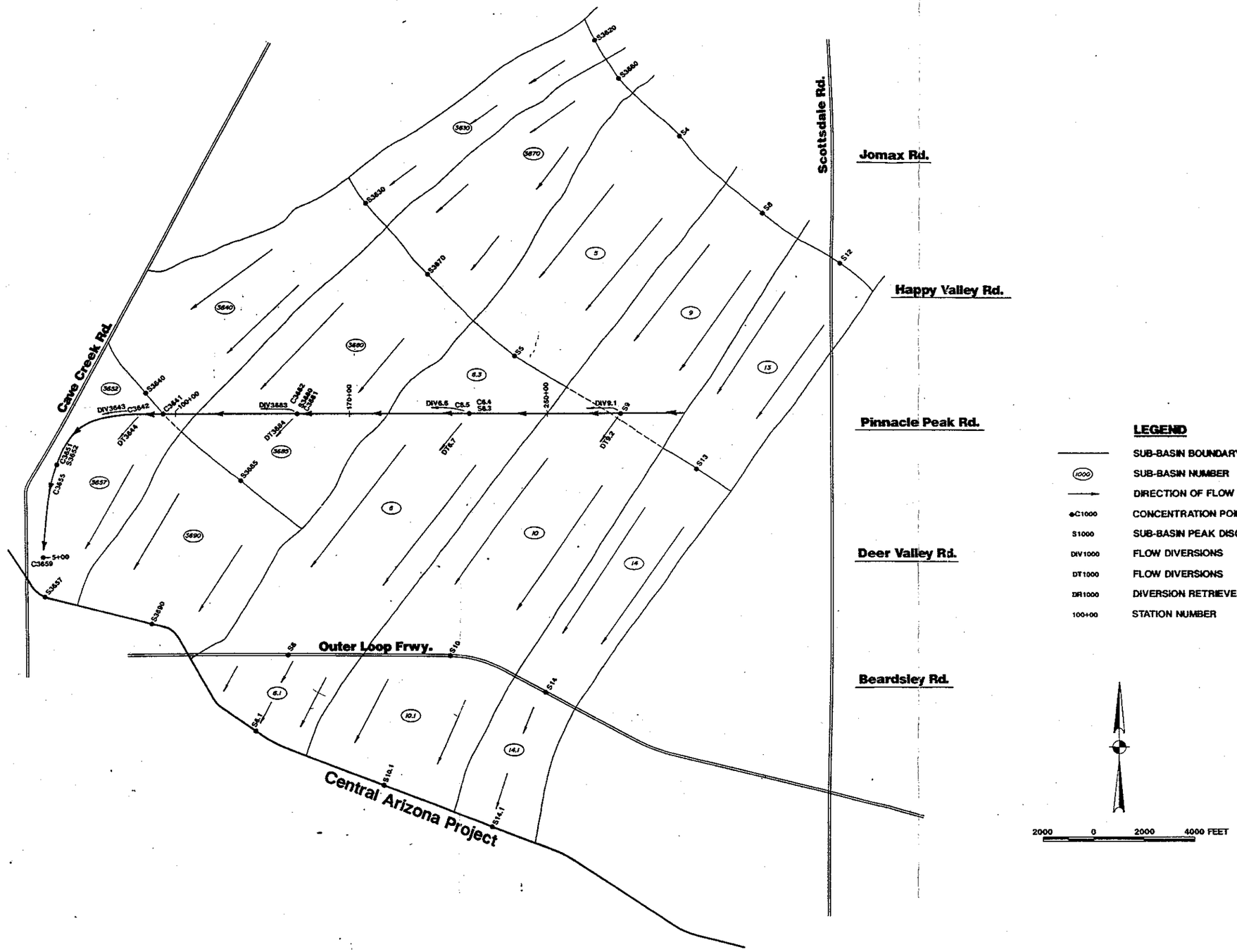
SCHEMATIC DIAGRAM FOR AREA-WIDE CHANNEL SYSTEM

PLATE 3



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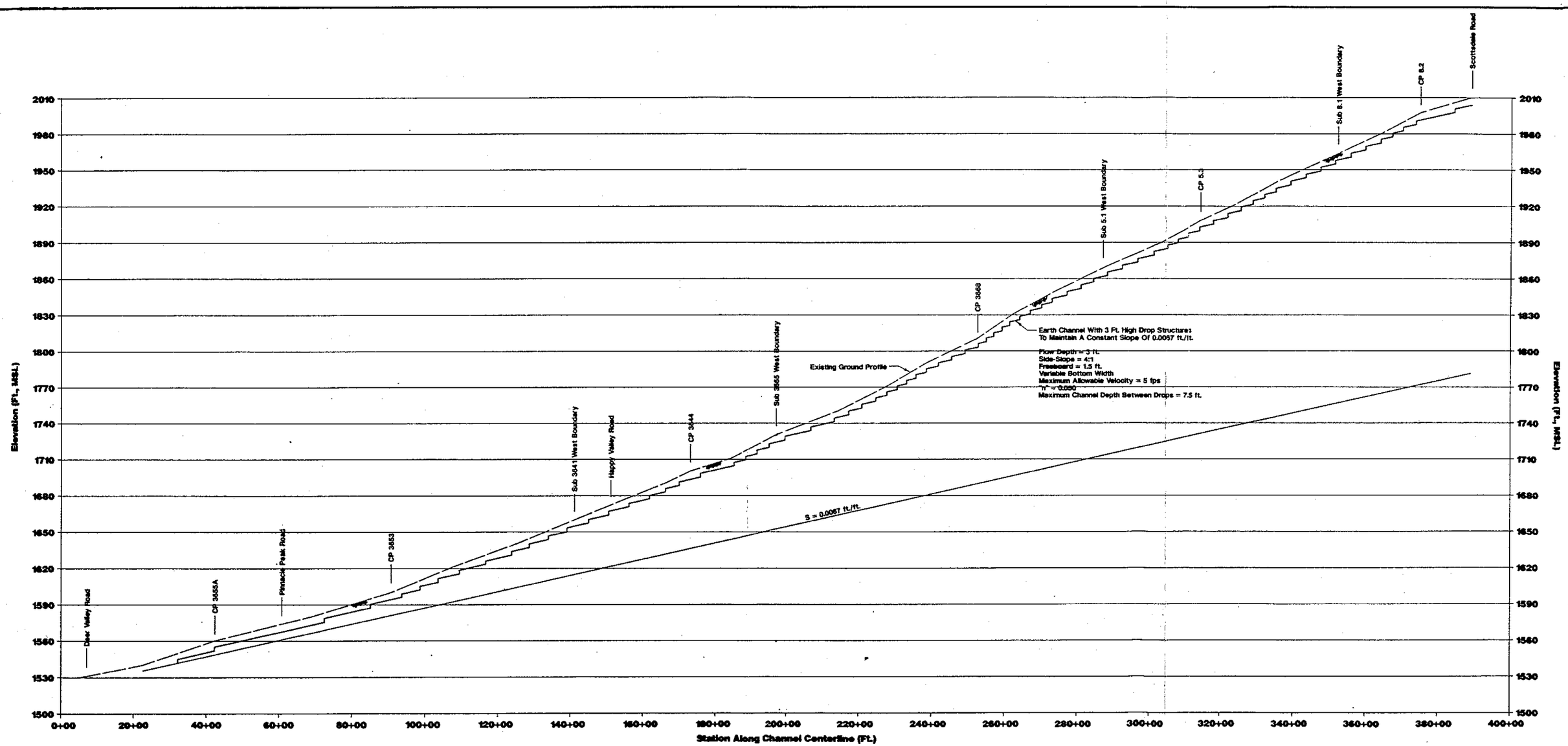
HEC-1 ROUTING SCHEMATIC
DESERT RIDGE ALTERNATE
EAST-WEST CHANNEL ALONG
PINNACLE PEAK ROAD

PLATE 6B



SCALE

Horiz. - 1 In. = 1000 Ft.
Vert. - 1 In. = 30 Ft.

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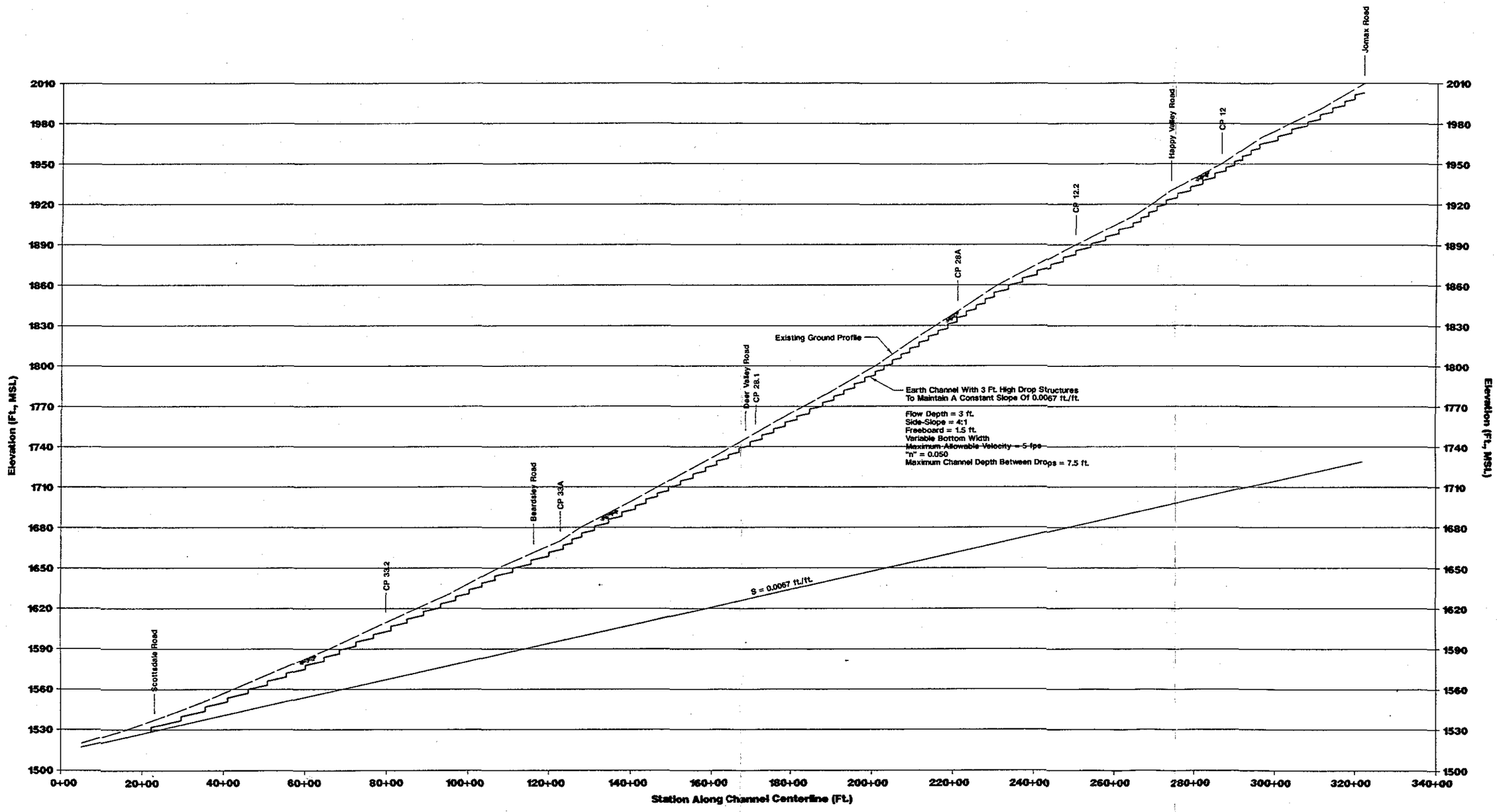
PROJECT NO. AC90060 DRAWN BY KMA

DATE 10/2/99 CHECKED BY RJW

CENTERLINE CHANNEL PROFILE FOR ORIGINAL EAST-WEST DESERT RIDGE CHANNEL

PLATE 7



SCALE
Horiz. - 1 in. = 1000 Ft.
Vert. - 1 in. = 30 Ft.

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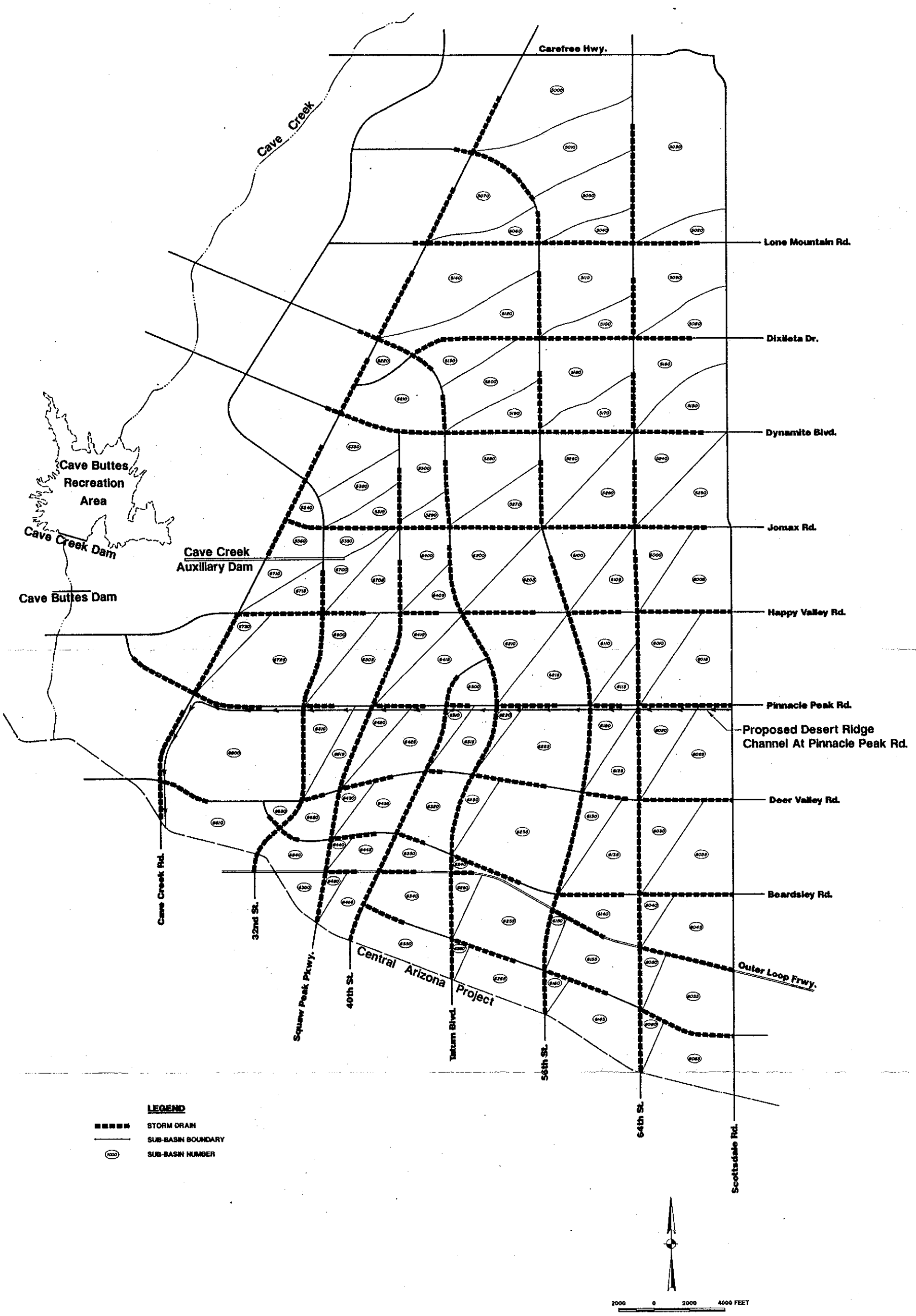
DATE 10/2/89 CHECKED BY RLW

CENTERLINE CHANNEL PROFILE FOR NORTH-SOUTH DESERT RIDGE CHANNEL

PLATE 8



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PROJECT NO. AC00060 DRAWN BY KMA
DATE 10/2/89 CHECKED BY RLW

SCHEMATIC DIAGRAM FOR STORM DRAIN SYSTEM

PLATE 9

APPENDICES

APPENDIX A

DATA BASE FOR RETENTION BASIN DESIGN EQUATIONS

Table A.1	Required Basin Storage Volume
Table A.2	Basin Surface Area (4-foot depth)
Table A.3	Basin Surface Area (7-foot depth)
Table A.4	Basin Surface Area (10-foot depth)
Table A.5	Basin excavation Cost (4-foot depth)
Table A.6	Basin Excavation Cost (7-foot depth)
Table A.7	Basin Excavation Cost (10-foot depth)
Table A.8	Basin Excavation Volume (4-foot depth)
Table A.9	Basin Excavation Volume (7-foot depth)
Table A.10	Basin Excavation Volume (10-foot depth)

Table A.1

Required Storage Volume For Retention Basin Design
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events
Curve Numbers Are Adjusted For Storm Duration (per Woodward)
Precipitation From NOAA Atlas 2, Arizona
City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Required Storage Volume (ac-ft)	Computed Storage Volume f(DA,CN,P) (ac-ft)	Percent Difference From Actual Volume (± 2)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	Y				
0.0148	2.53	79.6	4.50	336.6	340.0	1.0%	2.6%	5.3%
0.0192	2.25	81.1	4.55	317.5	326.3	2.7%		
0.0143	1.07	74.5	4.22	108.9	105.1	-3.6%		
0.0190	4.89	79.5	4.63	671.0	686.1	2.2%		
0.0182	0.39	75.0	4.22	40.4	39.1	-3.4%		
0.0192	0.69	77.0	4.22	75.6	75.1	-0.6%		
0.0185	3.21	74.8	4.32	331.1	332.2	0.3%		
0.0118	0.50	74.0	4.22	47.1	48.0	2.0%		
0.0100	0.48	76.2	4.22	50.9	50.6	-0.7%		
0.0125	5.94	76.7	4.34	676.5	670.6	-0.9%		
0.0158	4.01	78.0	4.40	488.9	487.7	-0.2%		
0.0148	2.53	85.3	2.65	183.4	178.8	-2.6%		
0.0192	2.25	86.4	2.65	171.4	165.5	-3.6%		
0.0143	1.07	81.5	2.65	65.7	65.5	-0.4%		
0.0190	4.89	84.9	2.65	342.5	340.8	-0.5%		
0.0182	0.39	82.0	2.65	24.6	24.3	-1.3%		
0.0192	0.69	83.0	2.65	44.8	44.7	-0.3%		
0.0185	3.21	81.6	2.67	192.8	199.9	3.5%		
0.0118	0.50	81.0	2.65	28.1	30.0	6.3%		
0.0100	0.48	82.2	2.65	29.9	30.1	0.8%		
0.0125	5.94	83.1	2.65	386.5	387.1	0.2%		
0.0158	4.01	84.2	2.65	276.4	272.2	-1.5%		

	log X2	log X3	log X4	log Y
	0.4031	1.9009	0.6532	2.5271
	0.3522	1.9090	0.6580	2.5017
	0.0294	1.8722	0.6253	2.0370
	0.6893	1.9004	0.6656	2.8267
	-0.4089	1.8751	0.6253	1.6064
	-0.1612	1.8865	0.6253	1.8785
	0.5065	1.8739	0.6355	2.5200
	-0.3010	1.8692	0.6253	1.6730
	-0.3188	1.8820	0.6253	1.7067
	0.7738	1.8848	0.6375	2.8303
	0.6031	1.8921	0.6435	2.6892
	0.4031	1.9309	0.4232	2.2634
	0.3522	1.9365	0.4232	2.2340
	0.0294	1.9112	0.4232	1.8176
	0.6893	1.9289	0.4232	2.5347
	-0.4089	1.9138	0.4232	1.3909
	-0.1612	1.9191	0.4232	1.6513
	0.5065	1.9117	0.4265	2.2851
	-0.3010	1.9085	0.4232	1.4487
	-0.3188	1.9149	0.4232	1.4757
	0.7738	1.9196	0.4232	2.5871
	0.6031	1.9253	0.4232	2.4415

Regression Output:

Constant		-4.9001	
Std Err of Y Est		0.0113	
R Squared		0.9995	
No. of Observations		22.0000	
Degrees of Freedom		18.0000	
X Coefficient(s)	1.0013	3.1392	1.6236
Std Err of Coef.	0.0069	0.2393	0.0430
	X2	X3	X4

Table A.2

Surface Area Required For Retention Basin Construction
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events
Minimum Basin Depth = 4 Ft (includes 1 Ft Freeboard)
Curve Numbers Are Adjusted For Storm Duration (per Woodward)
Precipitation From NOAA Atlas 2, Arizona
City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

File: R4AREA.WK1

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Actual Surface Area (acres)	Computed Surface Area f(S,DA,CN,P) (acres)	Percent Difference From Actual Area (± 2)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	Y				
0.0148	2.53	79.6	4.50	133.8	135.5	1.3%	2.5%	5.0%
0.0192	2.25	81.1	4.55	132.5	136.4	2.8%		
0.0143	1.07	74.5	4.22	43.8	42.7	-2.6%		
0.0190	4.89	79.5	4.63	277.5	281.5	1.4%		
0.0182	0.39	75.0	4.22	17.4	16.9	-2.7%		
0.0192	0.69	77.0	4.22	32.3	32.4	0.2%		
0.0185	3.21	74.8	4.32	137.1	137.7	0.5%		
0.0118	0.50	74.0	4.22	18.8	19.2	1.9%		
0.0100	0.48	76.2	4.22	19.9	19.6	-1.6%		
0.0125	5.94	76.7	4.34	260.7	255.7	-2.0%		
0.0158	4.01	78.0	4.40	195.7	195.1	-0.3%		
0.0148	2.53	85.3	2.65	73.5	72.2	-1.9%		
0.0192	2.25	86.4	2.65	72.1	70.0	-3.0%		
0.0143	1.07	81.5	2.65	26.7	26.8	0.4%		
0.0190	4.89	84.9	2.65	142.5	141.7	-0.6%		
0.0182	0.39	82.0	2.65	10.8	10.6	-1.5%		
0.0192	0.69	83.0	2.65	19.4	19.4	0.1%		
0.0185	3.21	81.6	2.67	80.4	83.6	3.9%		
0.0118	0.50	81.0	2.65	11.4	12.1	5.4%		
0.0100	0.48	82.2	2.65	11.9	11.8	-0.9%		
0.0125	5.94	83.1	2.65	149.7	149.1	-0.4%		
0.0158	4.01	84.2	2.65	111.3	110.1	-1.1%		

log X1	log X2	log X3	log X4	log Y
-1.8297	0.4031	1.9009	0.6532	2.1265
-1.7167	0.3522	1.9090	0.6580	2.1223
-1.8447	0.0294	1.8722	0.6253	1.6414
-1.7212	0.6893	1.9004	0.6656	2.4432
-1.7399	-0.4089	1.8751	0.6253	1.2400
-1.7167	-0.1612	1.8865	0.6253	1.5095
-1.7328	0.5065	1.8739	0.6355	2.1370
-1.9281	-0.3010	1.8692	0.6253	1.2745
-2.0000	-0.3188	1.8820	0.6253	1.2990
-1.9031	0.7738	1.8848	0.6375	2.4161
-1.8013	0.6031	1.8921	0.6435	2.2917
-1.8297	0.4031	1.9309	0.4232	1.8662
-1.7167	0.3522	1.9365	0.4232	1.8582
-1.8447	0.0294	1.9112	0.4232	1.4268
-1.7212	0.6893	1.9289	0.4232	2.1539
-1.7399	-0.4089	1.9138	0.4232	1.0322
-1.7167	-0.1612	1.9191	0.4232	1.2883
-1.7328	0.5065	1.9117	0.4265	1.9052
-1.9281	-0.3010	1.9085	0.4232	1.0577
-2.0000	-0.3188	1.9149	0.4232	1.0754
-1.9031	0.7738	1.9196	0.4232	2.1752
-1.8013	0.6031	1.9253	0.4232	2.0467

Regression Output:

Constant		-4.8346		
Std Err of Y Est		0.0105		
R Squared		0.9996		
No. of Observations		22.0000		
Degrees of Freedom		17.0000		
X Coefficient(s)	0.1756	0.9796	3.0788	1.5930
Std Err of Coef.	0.0261	0.0064	0.2344	0.0417
	X1	X2	X3	X4

Table A.3

Surface Area Required For Retention Basin Construction
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events
Minimum Basin Depth = 7 Ft (includes 1 Ft Freeboard)
Curve Numbers Are Adjusted For Storm Duration (per Wooduard)
Precipitation From NOAA Atlas 2, Arizona
City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

File: R7AREA.WK1

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Actual Surface Area (acres)	Computed Surface Area f(S,DA,CN,P) (acres)	Percent Difference From Actual Area (± %)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	Y				
0.0148	2.53	79.6	4.50	68.8	69.8	1.4%	2.4%	4.8%
0.0192	2.25	81.1	4.55	68.2	70.2	2.9%		
0.0143	1.07	74.5	4.22	23.0	22.6	-1.9%		
0.0190	4.89	79.5	4.63	141.5	142.5	0.7%		
0.0182	0.39	75.0	4.22	9.4	9.1	-2.8%		
0.0192	0.69	77.0	4.22	17.1	17.2	0.6%		
0.0185	3.21	74.8	4.32	70.5	70.8	0.4%		
0.0118	0.50	74.0	4.22	10.1	10.3	2.1%		
0.0100	0.48	76.2	4.22	10.7	10.6	-1.1%		
0.0125	5.94	76.7	4.34	132.9	129.6	-2.6%		
0.0158	4.01	78.0	4.40	100.2	99.6	-0.6%		
0.0148	2.53	85.3	2.65	38.2	37.7	-1.3%		
0.0192	2.25	86.4	2.65	37.5	36.6	-2.5%		
0.0143	1.07	81.5	2.65	14.2	14.3	0.8%		
0.0190	4.89	84.9	2.65	73.3	72.8	-0.6%		
0.0182	0.39	82.0	2.65	5.9	5.8	-2.5%		
0.0192	0.69	83.0	2.65	10.5	10.5	0.1%		
0.0185	3.21	81.6	2.67	41.7	43.5	4.1%		
0.0118	0.50	81.0	2.65	6.3	6.6	4.8%		
0.0100	0.48	82.2	2.65	6.5	6.4	-1.2%		
0.0125	5.94	83.1	2.65	76.8	76.5	-0.4%		
0.0158	4.01	84.2	2.65	57.4	56.9	-0.9%		

log X1	log X2	log X3	log X4	log Y
-1.8297	0.4031	1.9009	0.6532	1.8376
-1.7167	0.3522	1.9090	0.6580	1.8338
-1.8447	0.0294	1.8722	0.6253	1.3614
-1.7212	0.6893	1.9004	0.6656	2.1508
-1.7399	-0.4089	1.8751	0.6253	0.9726
-1.7167	-0.1612	1.8865	0.6253	1.2335
-1.7328	0.5065	1.8739	0.6355	1.8482
-1.9281	-0.3010	1.8692	0.6253	1.0048
-2.0000	-0.3188	1.8820	0.6253	1.0281
-1.9031	0.7738	1.8848	0.6375	2.1237
-1.8013	0.6031	1.8921	0.6435	2.0008
-1.8297	0.4031	1.9309	0.4232	1.5815
-1.7167	0.3522	1.9365	0.4232	1.5740
-1.8447	0.0294	1.9112	0.4232	1.1525
-1.7212	0.6893	1.9289	0.4232	1.8649
-1.7399	-0.4089	1.9138	0.4232	0.7735
-1.7167	-0.1612	1.9191	0.4232	1.0193
-1.7328	0.5065	1.9117	0.4265	1.6201
-1.9281	-0.3010	1.9085	0.4232	0.7965
-2.0000	-0.3188	1.9149	0.4232	0.8131
-1.9031	0.7738	1.9196	0.4232	1.8855
-1.8013	0.6031	1.9253	0.4232	1.7590

Regression Output:

Constant		-5.0051		
Std Err of Y Est		0.0102		
R Squared		0.9996		
No. of Observations		22.0000		
Degrees of Freedom		17.0000		
X Coefficient(s)	0.1705	0.9565	3.0282	1.5601
Std Err of Coef.	0.0253	0.0062	0.2271	0.0404
	X1	X2	X3	X4

Table A.4

Surface Area Required For Retention Basin Construction
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events
Minimum Basin Depth = 10 Ft (includes 1 Ft Freeboard)
Curve Numbers Are Adjusted For Storm Duration (per Woodward)
Precipitation From NOAA Atlas 2, Arizona
City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

File: R10AREA.WK1

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Actual Surface Area (acres)	Computed Surface Area f(S,DA,CN,P) (acres)	Percent Difference From Actual Area (± %)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	Y				
0.0148	2.53	79.6	4.50	47.4	48.2	1.6%	2.4%	4.9%
0.0192	2.25	81.1	4.55	47.1	48.5	3.0%		
0.0143	1.07	74.5	4.22	16.2	16.0	-1.2%		
0.0190	4.89	79.5	4.63	96.7	96.5	-0.2%		
0.0182	0.39	75.0	4.22	6.8	6.7	-2.9%		
0.0192	0.69	77.0	4.22	12.2	12.3	1.1%		
0.0185	3.21	74.8	4.32	48.6	48.8	0.3%		
0.0118	0.50	74.0	4.22	7.3	7.5	2.3%		
0.0100	0.48	76.2	4.22	7.7	7.7	-0.6%		
0.0125	5.94	76.7	4.34	90.8	87.9	-3.3%		
0.0158	4.01	78.0	4.40	68.7	68.1	-0.9%		
0.0148	2.53	85.3	2.65	26.6	26.4	-0.6%		
0.0192	2.25	86.4	2.65	26.2	25.7	-1.9%		
0.0143	1.07	81.5	2.65	10.2	10.3	1.3%		
0.0190	4.89	84.9	2.65	50.5	50.1	-0.7%		
0.0182	0.39	82.0	2.65	4.4	4.3	-3.6%		
0.0192	0.69	83.0	2.65	7.6	7.6	0.0%		
0.0185	3.21	81.6	2.67	29.0	30.3	4.3%		
0.0118	0.50	81.0	2.65	4.6	4.8	4.1%		
0.0100	0.48	82.2	2.65	4.8	4.7	-1.6%		
0.0125	5.94	83.1	2.65	52.9	52.6	-0.4%		
0.0158	4.01	84.2	2.65	39.7	39.5	-0.6%		

log X1	log X2	log X3	log X4	log Y
-1.8297	0.4031	1.9009	0.6532	1.6761
-1.7167	0.3522	1.9090	0.6580	1.6727
-1.8447	0.0294	1.8722	0.6253	1.2101
-1.7212	0.6893	1.9004	0.6656	1.9852
-1.7399	-0.4089	1.8751	0.6253	0.8355
-1.7167	-0.1612	1.8865	0.6253	1.0868
-1.7328	0.5065	1.8739	0.6355	1.6868
-1.9281	-0.3010	1.8692	0.6253	0.8651
-2.0000	-0.3188	1.8820	0.6253	0.8872
-1.9031	0.7738	1.8848	0.6375	1.9580
-1.8013	0.6031	1.8921	0.6435	1.8369
-1.8297	0.4031	1.9309	0.4232	1.4249
-1.7167	0.3522	1.9365	0.4232	1.4180
-1.8447	0.0294	1.9112	0.4232	1.0078
-1.7212	0.6893	1.9289	0.4232	1.7033
-1.7399	-0.4089	1.9138	0.4232	0.6459
-1.7167	-0.1612	1.9191	0.4232	0.8805
-1.7328	0.5065	1.9117	0.4265	1.4630
-1.9281	-0.3010	1.9085	0.4232	0.6661
-2.0000	-0.3188	1.9149	0.4232	0.6815
-1.9031	0.7738	1.9196	0.4232	1.7231
-1.8013	0.6031	1.9253	0.4232	1.5989

Regression Output:

Constant		-5.0272		
Std Err of Y Est		0.0104		
R Squared		0.9996		
No. of Observations		22.0000		
Degrees of Freedom		17.0000		
X Coefficient(s)	0.1650	0.9303	2.9684	1.5226
Std Err of Coef.	0.0257	0.0063	0.2309	0.0411
	X1	X2	X3	X4

Table A.5

Construction Cost Required For Retention Basin Installation
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events
Minimum Basin Depth = 4 Ft (includes 1 Ft Freeboard)
Curve Numbers Are Adjusted For Storm Duration (per Woodward)
Precipitation From NOAA Atlas 2, Arizona
Costs Only Reflect Excavation @ $4/cy: 10,000 < 100,000 cy
@ $3/cy: 100,000 < 500,000 cy
@ $2/cy: > 500,000 cy

File: R4COST.WK1

City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Unit Excavation Cost ($/cy)	Actual Construction Cost ($)	Computed Construction Cost f(S,DA,CN,P,$/cy) ($)	Percent Difference From Actual Cost (± %)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	X5	Y				
0.0148	2.53	79.6	4.50	2.00	8,396,349	8,809,237	4.7%	3.6%	7.4%
0.0192	2.25	81.1	4.55	2.00	9,949,595	10,310,623	3.5%		
0.0143	1.07	74.5	4.22	2.00	1,732,707	1,705,989	-1.6%		
0.0190	4.89	79.5	4.63	2.00	28,649,374	27,692,714	-3.5%		
0.0182	0.39	75.0	4.22	3.00	847,740	830,938	-2.0%		
0.0192	0.69	77.0	4.22	2.00	1,361,432	1,384,630	1.7%		
0.0185	3.21	74.8	4.32	2.00	10,172,759	9,898,929	-2.8%		
0.0118	0.50	74.0	4.22	3.00	755,367	777,309	2.8%		
0.0100	0.48	76.2	4.22	3.00	750,696	739,613	-1.5%		
0.0125	5.94	76.7	4.34	2.00	19,137,936	18,962,027	-0.9%		
0.0158	4.01	78.0	4.40	2.00	15,120,932	14,946,660	-1.2%		
0.0148	2.53	85.3	2.65	2.00	3,619,028	3,643,399	0.7%		
0.0192	2.25	86.4	2.65	2.00	4,186,730	4,047,599	-3.4%		
0.0143	1.07	81.5	2.65	3.00	1,329,273	1,372,273	3.1%		
0.0190	4.89	84.9	2.65	2.00	10,961,413	10,566,468	-3.7%		
0.0182	0.39	82.0	2.65	3.00	446,170	436,315	-2.3%		
0.0192	0.69	83.0	2.65	3.00	1,014,450	1,040,270	2.5%		
0.0185	3.21	81.6	2.67	2.00	4,759,691	4,962,247	4.1%		
0.0118	0.50	81.0	2.65	3.00	395,899	410,336	3.5%		
0.0100	0.48	82.2	2.65	3.00	389,167	364,970	-6.6%		
0.0125	5.94	83.1	2.65	2.00	8,727,286	8,958,033	2.6%		
0.0158	4.01	84.2	2.65	2.00	6,776,299	6,729,566	-0.7%		

log X1	log X2	log X3	log X4	log X5	log Y
-1.8297	0.4031	1.9009	0.6532	0.3010	6.9241
-1.7167	0.3522	1.9090	0.6580	0.3010	6.9978
-1.8447	0.0294	1.8722	0.6253	0.3010	6.2387
-1.7212	0.6893	1.9004	0.6656	0.3010	7.4571
-1.7399	-0.4089	1.8751	0.6253	0.4771	5.9283
-1.7167	-0.1612	1.8865	0.6253	0.3010	6.1340
-1.7328	0.5065	1.8739	0.6355	0.3010	7.0074
-1.9281	-0.3010	1.8692	0.6253	0.4771	5.8782
-2.0000	-0.3188	1.8820	0.6253	0.4771	5.8755
-1.9031	0.7738	1.8848	0.6375	0.3010	7.2819
-1.8013	0.6031	1.8921	0.6435	0.3010	7.1796
-1.8297	0.4031	1.9309	0.4232	0.3010	6.5586
-1.7167	0.3522	1.9365	0.4232	0.3010	6.6219
-1.8447	0.0294	1.9112	0.4232	0.4771	6.1236
-1.7212	0.6893	1.9289	0.4232	0.3010	7.0399
-1.7399	-0.4089	1.9138	0.4232	0.4771	5.6495
-1.7167	-0.1612	1.9191	0.4232	0.4771	6.0062
-1.7328	0.5065	1.9117	0.4265	0.3010	6.6776
-1.9281	-0.3010	1.9085	0.4232	0.4771	5.5976
-2.0000	-0.3188	1.9149	0.4232	0.4771	5.5901
-1.9031	0.7738	1.9196	0.4232	0.3010	6.9409
-1.8013	0.6031	1.9253	0.4232	0.3010	6.8310

Regression Output:

Constant		-2.7489			
Std Err of Y Est		0.0154			
R Squared		0.9995			
No. of Observations		22.0000			
Degrees of Freedom		16.0000			
X Coefficient(s)	0.7854	1.3510	4.6231	2.2712	1.0454
Std Err of Coef.	0.0404	0.0160	0.3449	0.0634	0.0840
	X1	X2	X3	X4	X5

Table A.6

Construction Cost Required For Retention Basin Installation
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events — File: R7COST.WK1
Minimum Basin Depth = 7 Ft (includes 1 Ft Freeboard)
Curve Numbers Are Adjusted For Storm Duration (per Woodward)
Precipitation From NOAA Atlas 2, Arizona
Costs Only Reflect Excavation @ $4/cy: 10,000 < 100,000 cy
@ $3/cy: 100,000 < 500,000 cy
@ $2/cy: > 500,000 cy

City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Unit Excavation Cost ($/cy)	Actual Construction Cost ($)	Computed Construction Cost f(S,DA,CN,P,$/cy) ($)	Percent Difference From Actual Cost (± %)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	X5	Y				
0.0148	2.53	79.6	4.50	2.00	3,834,563	4,040,968	5.1%	3.8%	7.7%
0.0192	2.25	81.1	4.55	2.00	4,369,294	4,516,184	3.3%		
0.0143	1.07	74.5	4.22	3.00	1,340,638	1,362,719	1.6%		
0.0190	4.89	79.5	4.63	2.00	11,918,706	11,414,250	-4.4%		
0.0182	0.39	75.0	4.22	3.00	464,185	449,266	-3.3%		
0.0192	0.69	77.0	4.22	3.00	1,030,013	1,076,136	4.3%		
0.0185	3.21	74.8	4.32	2.00	4,478,563	4,315,706	-3.8%		
0.0118	0.50	74.0	4.22	3.00	446,961	449,844	0.6%		
0.0100	0.48	76.2	4.22	3.00	456,402	442,128	-3.2%		
0.0125	5.94	76.7	4.34	2.00	8,465,351	8,467,262	0.0%		
0.0158	4.01	78.0	4.40	2.00	6,613,249	6,552,009	-0.9%		
0.0148	2.53	85.3	2.65	2.00	1,753,175	1,760,841	0.4%		
0.0192	2.25	86.4	2.65	2.00	1,941,944	1,871,778	-3.7%		
0.0143	1.07	81.5	2.65	3.00	725,781	748,152	3.0%		
0.0190	4.89	84.9	2.65	2.00	4,791,924	4,605,958	-4.0%		
0.0182	0.39	82.0	2.65	4.00	345,105	341,172	-1.2%		
0.0192	0.69	83.0	2.65	3.00	542,532	552,930	1.9%		
0.0185	3.21	81.6	2.67	2.00	2,198,216	2,261,062	2.8%		
0.0118	0.50	81.0	2.65	4.00	330,894	343,383	3.6%		
0.0100	0.48	82.2	2.65	4.00	334,211	316,106	-5.7%		
0.0125	5.94	83.1	2.65	2.00	4,058,554	4,190,444	3.1%		
0.0158	4.01	84.2	2.65	2.00	3,114,342	3,096,820	-0.6%		

log X1	log X2	log X3	log X4	log X5	log Y
-1.8297	0.4031	1.9009	0.6532	0.3010	6.5837
-1.7167	0.3522	1.9090	0.6580	0.3010	6.6404
-1.8447	0.0294	1.8722	0.6253	0.4771	6.1273
-1.7212	0.6893	1.9004	0.6656	0.3010	7.0762
-1.7399	-0.4089	1.8751	0.6253	0.4771	5.6667
-1.7167	-0.1612	1.8865	0.6253	0.4771	6.0128
-1.7328	0.5065	1.8739	0.6355	0.3010	6.6511
-1.9281	-0.3010	1.8692	0.6253	0.4771	5.6503
-2.0000	-0.3188	1.8820	0.6253	0.4771	5.6593
-1.9031	0.7738	1.8848	0.6375	0.3010	6.9276
-1.8013	0.6031	1.8921	0.6435	0.3010	6.8204
-1.8297	0.4031	1.9309	0.4232	0.3010	6.2438
-1.7167	0.3522	1.9365	0.4232	0.3010	6.2882
-1.8447	0.0294	1.9112	0.4232	0.4771	5.8608
-1.7212	0.6893	1.9289	0.4232	0.3010	6.6[illegible]05
-1.7399	-0.4089	1.9138	0.4232	0.6021	5.5[illegible]0
-1.7167	-0.1612	1.9191	0.4232	0.4771	5.7[illegible]1
-1.7328	0.5065	1.9117	0.4265	0.3010	6.3[illegible]1
-1.9281	-0.3010	1.9085	0.4232	0.6021	5.5[illegible]7
-2.0000	-0.3188	1.9149	0.4232	0.6021	5.5[illegible]40
-1.9031	0.7738	1.9196	0.4232	0.3010	6.6034
-1.8013	0.6031	1.9253	0.4232	0.3010	6.4934

Regression Output:

Constant		-3.1127			
Std Err of Y Est		0.0162			
R Squared		0.9992			
No. of Observations		22.0000			
Degrees of Freedom		16.0000			
X Coefficient(s)	0.5857	1.2691	4.4871	2.1548	1.1366
Std Err of Coef.	0.0419	0.0224	0.3714	0.0700	0.0914
	X1	X2	X3	X4	X5

Table A.7

Construction Cost Required For Retention Basin Installation
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events
Minimum Basin Depth = 10 Ft (includes 1 Ft Freeboard)
Curve Numbers Are Adjusted For Storm Duration (per Woodward)
Precipitation From NOAA Atlas 2, Arizona
Costs Only Reflect Excavation @ $4/cy: 10,000 < 100,000 cy
@ $3/cy: 100,000 < 500,000 cy
@ $2/cy: > 500,000 cy

File: R10COST.WK1

City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Unit Excavation Cost ($/cy)	Actual Construction Cost ($)	Computed Construction Cost f(S,DA,CN,P,$/cy) ($)	Percent Difference From Actual Cost (± %)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	X5	Y				
0.0148	2.53	79.6	4.50	2.00	2,681,393	2,820,707	4.9%	3.6%	7.3%
0.0192	2.25	81.1	4.55	2.00	2,962,630	3,062,609	3.3%		
0.0143	1.07	74.5	4.22	3.00	1,020,172	1,036,891	1.6%		
0.0190	4.89	79.5	4.63	2.00	7,713,337	7,374,597	-4.6%		
0.0182	0.39	75.0	4.22	3.00	366,712	355,132	-3.3%		
0.0192	0.69	77.0	4.22	3.00	773,339	807,846	4.3%		
0.0185	3.21	74.8	4.32	2.00	3,042,876	2,945,585	-3.3%		
0.0118	0.50	74.0	4.22	3.00	367,660	369,359	0.5%		
0.0100	0.48	76.2	4.22	3.00	380,342	368,553	-3.2%		
0.0125	5.94	76.7	4.34	2.00	5,772,065	5,767,852	-0.1%		
0.0158	4.01	78.0	4.40	2.00	4,468,369	4,432,245	-0.8%		
0.0148	2.53	85.3	2.65	2.00	1,279,841	1,283,383	0.3%		
0.0192	2.25	86.4	2.65	2.00	1,374,539	1,328,991	-3.4%		
0.0143	1.07	81.5	2.65	3.00	571,674	587,020	2.6%		
0.0190	4.89	84.9	2.65	2.00	3,237,070	3,120,048	-3.8%		
0.0182	0.39	82.0	2.65	4.00	281,742	278,116	-1.3%		
0.0192	0.69	83.0	2.65	3.00	422,747	429,607	1.6%		
0.0185	3.21	81.6	2.67	2.00	1,550,784	1,595,365	2.8%		
0.0118	0.50	81.0	2.65	4.00	280,211	290,674	3.6%		
0.0100	0.48	82.2	2.65	4.00	286,393	272,652	-5.0%		
0.0125	5.94	83.1	2.65	2.00	2,876,536	2,960,102	2.8%		
0.0158	4.01	84.2	2.65	2.00	2,188,458	2,177,804	-0.5%		

log X1	log X2	log X3	log X4	log X5	log Y
-1.8297	0.4031	1.9009	0.6532	0.3010	6.4284
-1.7167	0.3522	1.9090	0.6580	0.3010	6.4717
-1.8447	0.0294	1.8722	0.6253	0.4771	6.0087
-1.7212	0.6893	1.9004	0.6656	0.3010	6.8872
-1.7399	-0.4089	1.8751	0.6253	0.4771	5.5643
-1.7167	-0.1612	1.8865	0.6253	0.4771	5.8884
-1.7328	0.5065	1.8739	0.6355	0.3010	6.4833
-1.9281	-0.3010	1.8692	0.6253	0.4771	5.5654
-2.0000	-0.3188	1.8820	0.6253	0.4771	5.5802
-1.9031	0.7738	1.8848	0.6375	0.3010	6.7613
-1.8013	0.6031	1.8921	0.6435	0.3010	6.6501
-1.8297	0.4031	1.9309	0.4232	0.3010	6.1072
-1.7167	0.3522	1.9365	0.4232	0.3010	6.1382
-1.8447	0.0294	1.9112	0.4232	0.4771	5.7571
-1.7212	0.6893	1.9289	0.4232	0.3010	6.5102
-1.7399	-0.4089	1.9138	0.4232	0.6021	5.4499
-1.7167	-0.1612	1.9191	0.4232	0.4771	5.6261
-1.7328	0.5065	1.9117	0.4265	0.3010	6.1906
-1.9281	-0.3010	1.9085	0.4232	0.6021	5.4475
-2.0000	-0.3188	1.9149	0.4232	0.6021	5.4570
-1.9031	0.7738	1.9196	0.4232	0.3010	6.4589
-1.8013	0.6031	1.9253	0.4232	0.3010	6.3401

Regression Output:

Constant		-2.9092			
Std Err of Y Est		0.0154			
R Squared		0.9992			
No. of Observations		22.0000			
Degrees of Freedom		16.0000			
X Coefficient(s)	0.4669	1.2013	4.2372	2.0406	1.1363
Std Err of Coef.	0.0399	0.0213	0.3534	0.0666	0.0869
	X1	X2	X3	X4	X5

Table A.8

Excavation Volume Required For Retention Basin Construction
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events
Minimum Basin Depth = 4 Ft (includes 1 Ft Freeboard)
Curve Numbers Are Adjusted For Storm Duration (per Woodward)
Precipitation From NOAA Atlas 2, Arizona
City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

File: R4EXC.WK1

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Actual Excavation Volume (cy)	Computed Excavation Vol. f(S,DA,CN,P) (cy)	Percent Difference From Actual Volume (± %)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	Y				
0.0148	2.53	79.6	4.50	4,198,174	4,409,811	4.8%	3.5%	7.2%
0.0192	2.25	81.1	4.55	4,974,798	5,153,703	3.5%		
0.0143	1.07	74.5	4.22	866,353	860,889	-0.6%		
0.0190	4.89	79.5	4.63	14,324,687	13,771,204	-4.0%		
0.0182	0.39	75.0	4.22	282,580	275,871	-2.4%		
0.0192	0.69	77.0	4.22	680,716	698,962	2.6%		
0.0185	3.21	74.8	4.32	5,086,379	4,946,615	-2.8%		
0.0118	0.50	74.0	4.22	251,789	258,485	2.6%		
0.0100	0.48	76.2	4.22	250,232	246,180	-1.6%		
0.0125	5.94	76.7	4.34	9,568,968	9,456,871	-1.2%		
0.0158	4.01	78.0	4.40	7,560,466	7,458,965	-1.4%		
0.0148	2.53	85.3	2.65	1,809,514	1,830,429	1.1%		
0.0192	2.25	86.4	2.65	2,093,365	2,030,892	-3.1%		
0.0143	1.07	81.5	2.65	443,091	454,437	2.5%		
0.0190	4.89	84.9	2.65	5,480,707	5,275,231	-3.9%		
0.0182	0.39	82.0	2.65	148,723	145,240	-2.4%		
0.0192	0.69	83.0	2.65	338,150	344,703	1.9%		
0.0185	3.21	81.6	2.67	2,379,845	2,486,734	4.3%		
0.0118	0.50	81.0	2.65	131,966	136,811	3.5%		
0.0100	0.48	82.2	2.65	129,722	121,832	-6.5%		
0.0125	5.94	83.1	2.65	4,363,643	4,481,351	2.6%		
0.0158	4.01	84.2	2.65	3,388,149	3,369,292	-0.6%		

log X1	log X2	log X3	log X4	log Y
-1.8297	0.4031	1.9009	0.6532	6.6231
-1.7167	0.3522	1.9090	0.6580	6.6968
-1.8447	0.0294	1.8722	0.6253	5.9377
-1.7212	0.6893	1.9004	0.6656	7.1561
-1.7399	-0.4089	1.8751	0.6253	5.4511
-1.7167	-0.1612	1.8865	0.6253	5.8330
-1.7328	0.5065	1.8739	0.6355	6.7064
-1.9281	-0.3010	1.8692	0.6253	5.4010
-2.0000	-0.3188	1.8820	0.6253	5.3983
-1.9031	0.7738	1.8848	0.6375	6.9809
-1.8013	0.6031	1.8921	0.6435	6.8785
-1.8297	0.4031	1.9309	0.4232	6.2576
-1.7167	0.3522	1.9365	0.4232	6.3208
-1.8447	0.0294	1.9112	0.4232	5.6465
-1.7212	0.6893	1.9289	0.4232	6.7388
-1.7399	-0.4089	1.9138	0.4232	5.1724
-1.7167	-0.1612	1.9191	0.4232	5.5291
-1.7328	0.5065	1.9117	0.4265	6.3765
-1.9281	-0.3010	1.9085	0.4232	5.1205
-2.0000	-0.3188	1.9149	0.4232	5.1130
-1.9031	0.7738	1.9196	0.4232	6.6398
-1.8013	0.6031	1.9253	0.4232	6.5300

Regression Output:

Constant		-2.7045		
Std Err of Y Est		0.0151		
R Squared		0.9996		
No. of Observations		22.0000		
Degrees of Freedom		17.0000		
X Coefficient(s)	0.7782	1.3441	4.6049	2.2620
Std Err of Coef.	0.0374	0.0092	0.3361	0.0598
	X1	X2	X3	X4

Table A.9

Excavation Volume Required For Retention Basin Construction
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events
Minimum Basin Depth = 7 Ft (includes 1 Ft Freeboard)
Curve Numbers Are Adjusted For Storm Duration (per Woodward)
Precipitation From NOAA Atlas 2, Arizona
City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

File: R7EXC.WK1

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Actual Excavation Volume (cy)	Computed Excavation Vol. f(S,DA,CN,P) (cy)	Percent Difference From Actual Volume (± %)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	Y				
0.0148	2.53	79.6	4.50	1,917,281	2,032,005	5.6%	3.9%	8.0%
0.0192	2.25	81.1	4.55	2,184,647	2,260,768	3.4%		
0.0143	1.07	74.5	4.22	446,879	449,229	0.5%		
0.0190	4.89	79.5	4.63	5,959,353	5,594,365	-6.5%		
0.0182	0.39	75.0	4.22	154,728	151,836	-1.9%		
0.0192	0.69	77.0	4.22	343,338	355,818	3.5%		
0.0185	3.21	74.8	4.32	2,239,282	2,168,076	-3.3%		
0.0118	0.50	74.0	4.22	148,987	152,432	2.3%		
0.0100	0.48	76.2	4.22	152,134	149,858	-1.5%		
0.0125	5.94	76.7	4.34	4,232,675	4,191,182	-1.0%		
0.0158	4.01	78.0	4.40	3,306,625	3,257,605	-1.5%		
0.0148	2.53	85.3	2.65	876,588	896,887	2.3%		
0.0192	2.25	86.4	2.65	970,972	950,337	-2.2%		
0.0143	1.07	81.5	2.65	241,927	248,419	2.6%		
0.0190	4.89	84.9	2.65	2,395,962	2,290,539	-4.6%		
0.0182	0.39	82.0	2.65	86,276	83,755	-3.0%		
0.0192	0.69	83.0	2.65	180,844	184,529	2.0%		
0.0185	3.21	81.6	2.67	1,099,108	1,145,336	4.0%		
0.0118	0.50	81.0	2.65	82,723	84,506	2.1%		
0.0100	0.48	82.2	2.65	83,553	77,974	-7.2%		
0.0125	5.94	83.1	2.65	2,029,277	2,094,565	3.1%		
0.0158	4.01	84.2	2.65	1,557,171	1,556,527	-0.0%		

log X1	log X2	log X3	log X4	log Y
-1.8297	0.4031	1.9009	0.6532	6.2827
-1.7167	0.3522	1.9090	0.6580	6.3394
-1.8447	0.0294	1.8722	0.6253	5.6502
-1.7212	0.6893	1.9004	0.6656	6.7752
-1.7399	-0.4089	1.8751	0.6253	5.1896
-1.7167	-0.1612	1.8865	0.6253	5.5357
-1.7328	0.5065	1.8739	0.6355	6.3501
-1.9281	-0.3010	1.8692	0.6253	5.1731
-2.0000	-0.3188	1.8820	0.6253	5.1822
-1.9031	0.7738	1.8848	0.6375	6.6266
-1.8013	0.6031	1.8921	0.6435	6.5194
-1.8297	0.4031	1.9309	0.4232	5.9428
-1.7167	0.3522	1.9365	0.4232	5.9872
-1.8447	0.0294	1.9112	0.4232	5.3837
-1.7212	0.6893	1.9289	0.4232	6.3795
-1.7399	-0.4089	1.9138	0.4232	4.9359
-1.7167	-0.1612	1.9191	0.4232	5.2573
-1.7328	0.5065	1.9117	0.4265	6.0410
-1.9281	-0.3010	1.9085	0.4232	4.9176
-2.0000	-0.3188	1.9149	0.4232	4.9220
-1.9031	0.7738	1.9196	0.4232	6.3073
-1.8013	0.6031	1.9253	0.4232	6.1923

Regression Output:

Constant		-2.7975		
Std Err of Y Est		0.0167		
R Squared		0.9994		
No. of Observations		22.0000		
Degrees of Freedom		17.0000		
X Coefficient(s)	0.5667	1.2590	4.3470	2.1123
Std Err of Coef.	0.0414	0.0102	0.3722	0.0662
	X1	X2	X3	X4

Table A.10

Excavation Volume Required For Retention Basin Construction
Regression Analysis Is Based On The 100-Year, 2-Hour & 24-Hour Events
Minimum Basin Depth = 10 Ft (Includes 1 Ft Freeboard)
Curve Numbers Are Adjusted For Storm Duration (per Woodward)
Precipitation From NOAA Atlas 2, Arizona
City of Phoenix
Paradise Valley Fan Terrace, Areas C & D

File: R10EXC.WK1

Ground Slope at Basin Outlet (ft/ft)	Total Contributing Drainage Area (sq mi)	Area Weighted Curve Number	Area Weighted Precipitation (inches)	Actual Excavation Volume (cy)	Computed Excavation Vol. f(S,DA,CN,P) (cy)	Percent Difference From Actual Volume (± %)	1 Standard Error (± 68%)	2 Standard Errors (± 95%)
X1	X2	X3	X4	Y				
0.0148	2.53	79.6	4.50	1,340,697	1,418,380	5.5%	3.8%	7.7%
0.0192	2.25	81.1	4.55	1,481,315	1,533,116	3.4%		
0.0143	1.07	74.5	4.22	340,057	341,825	0.5%		
0.0190	4.89	79.5	4.63	3,856,668	3,614,580	-6.7%		
0.0182	0.39	75.0	4.22	122,237	120,019	-1.8%		
0.0192	0.69	77.0	4.22	257,780	267,114	3.5%		
0.0185	3.21	74.8	4.32	1,521,438	1,479,757	-2.8%		
0.0118	0.50	74.0	4.22	122,553	125,155	2.1%		
0.0100	0.48	76.2	4.22	126,781	124,916	-1.5%		
0.0125	5.94	76.7	4.34	2,886,032	2,855,063	-1.1%		
0.0158	4.01	78.0	4.40	2,234,185	2,203,699	-1.4%		
0.0148	2.53	85.3	2.65	639,920	653,671	2.1%		
0.0192	2.25	86.4	2.65	687,270	674,734	-1.9%		
0.0143	1.07	81.5	2.65	190,558	194,918	2.2%		
0.0190	4.89	84.9	2.65	1,618,535	1,551,613	-4.3%		
0.0182	0.39	82.0	2.65	70,436	68,277	-3.2%		
0.0192	0.69	83.0	2.65	140,916	143,372	1.7%		
0.0185	3.21	81.6	2.67	775,392	808,109	4.0%		
0.0118	0.50	81.0	2.65	70,053	71,537	2.1%		
0.0100	0.48	82.2	2.65	71,598	67,257	-6.5%		
0.0125	5.94	83.1	2.65	1,438,268	1,479,588	2.8%		
0.0158	4.01	84.2	2.65	1,094,229	1,094,600	0.0%		

log X1	log X2	log X3	log X4	log Y
-1.8297	0.4031	1.9009	0.6532	6.1273
-1.7167	0.3522	1.9090	0.6580	6.1706
-1.8447	0.0294	1.8722	0.6253	5.5316
-1.7212	0.6893	1.9004	0.6656	6.5862
-1.7399	-0.4089	1.8751	0.6253	5.0872
-1.7167	-0.1612	1.8865	0.6253	5.4112
-1.7328	0.5065	1.8739	0.6355	6.1823
-1.9281	-0.3010	1.8692	0.6253	5.0883
-2.0000	-0.3188	1.8820	0.6253	5.1031
-1.9031	0.7738	1.8848	0.6375	6.4603
-1.8013	0.6031	1.8921	0.6435	6.3491
-1.8297	0.4031	1.9309	0.4232	5.8061
-1.7167	0.3522	1.9365	0.4232	5.8371
-1.8447	0.0294	1.9112	0.4232	5.2800
-1.7212	0.6893	1.9289	0.4232	6.2091
-1.7399	-0.4089	1.9138	0.4232	4.8478
-1.7167	-0.1612	1.9191	0.4232	5.1490
-1.7328	0.5065	1.9117	0.4265	5.8895
-1.9281	-0.3010	1.9085	0.4232	4.8454
-2.0000	-0.3188	1.9149	0.4232	4.8549
-1.9031	0.7738	1.9196	0.4232	6.1578
-1.8013	0.6031	1.9253	0.4232	6.0391

Regression Output:

Constant		-2.5945		
Std Err of Y Est		0.0160		
R Squared		0.9994		
No. of Observations		22.0000		
Degrees of Freedom		17.0000		
X Coefficient(s)	0.4479	1.1712	4.0973	1.9981
Std Err of Coef.	0.0396	0.0098	0.3563	0.0634
	X1	X2	X3	X4

APPENDIX B

CHANNEL DESIGN CURVES

Figure B.1 Soil-Cement Banks, n = 0.025, Discharge vs. Channel Cost/L.F., 0 to 7,000 cfs

Figure B.2 Soil-Cement Banks, n = 0.025, Discharge vs. Channel Cost/L.F., 7,000 to 14,000 cfs

Figure B.3 Soil-Cement Banks, n = 0.035, Discharge vs. Channel Cost/L.F., 0 to 7,000 cfs

Figure B.4 Soil-Cement Banks, n = 0.035, Discharge vs. Channel Cost/L.F., 7,000 to 14,000 cfs

Figure B.5 Total Concrete Lining, n = 0.020, Discharge vs. Channel Cost/L.F., 0 to 7,000 cfs

Figure B.6 Total Concrete Lining, n = 0.020, Discharge vs. Channel Cost/L.F., 7,000 to 14,000 cfs

Figure B.7 Total Earth Lining, n = 0.050, Discharge vs. Channel Cost/L.F., 0 to 7,000 cfs

Figure B.8 Total Earth Lining, n = 0.050, Discharge vs. Channel Cost/L.F., 0 to 7,000 cfs

Figure B.9 Soil-Cement Banks, n = 0.025, Discharge vs. Channel R/W Width, 0 to 7,000 cfs

Figure B.10 Soil-Cement Banks, n = 0.025, Discharge vs. Channel R/W Width, 7,000 to 14,000 cfs

Figure B.11 Soil-Cement Banks, n = 0.035, Discharge vs. Channel R/W Width, 0 to 7,000 cfs

Figure B.12 Soil-Cement Banks, n = 0.035, Discharge vs. Channel R/W Width, 7,000 to 14,000 cfs

Figure B.13 Total Concrete Lining, n = 0.020, Discharge vs. Channel R/W Width, 0 to 7,000 cfs

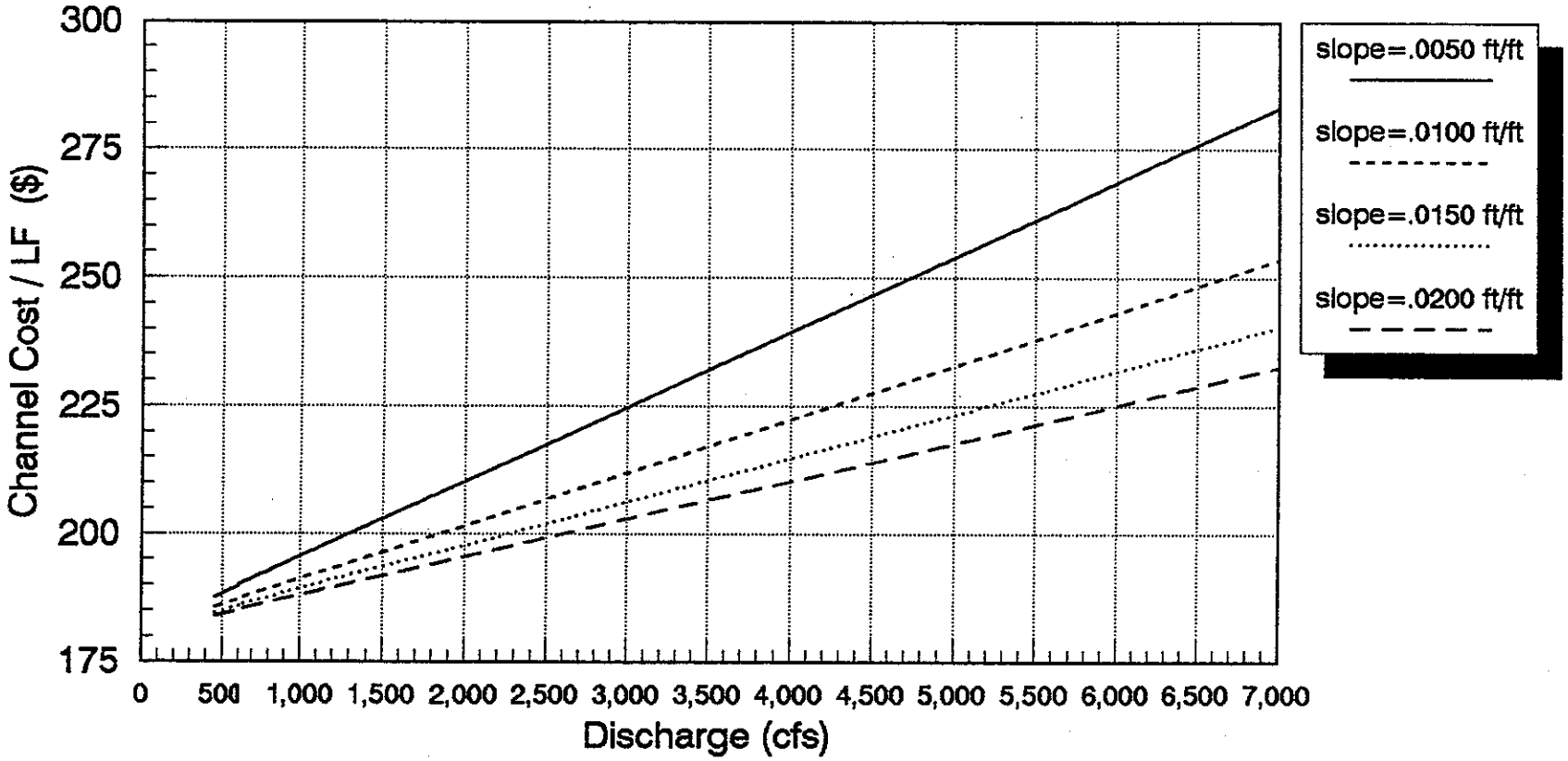
Figure B.14 Total Concrete Lining, n = 0.020, Discharge vs. Channel R/W Width, 7,000 to 14,000 cfs

APPENDIX B
(continued)

CHANNEL DESIGN CURVES

Figure B.15 Total Earth Lining, n = 0.050, Discharge vs. Channel R/W Width, 0 to 7,000 cfs

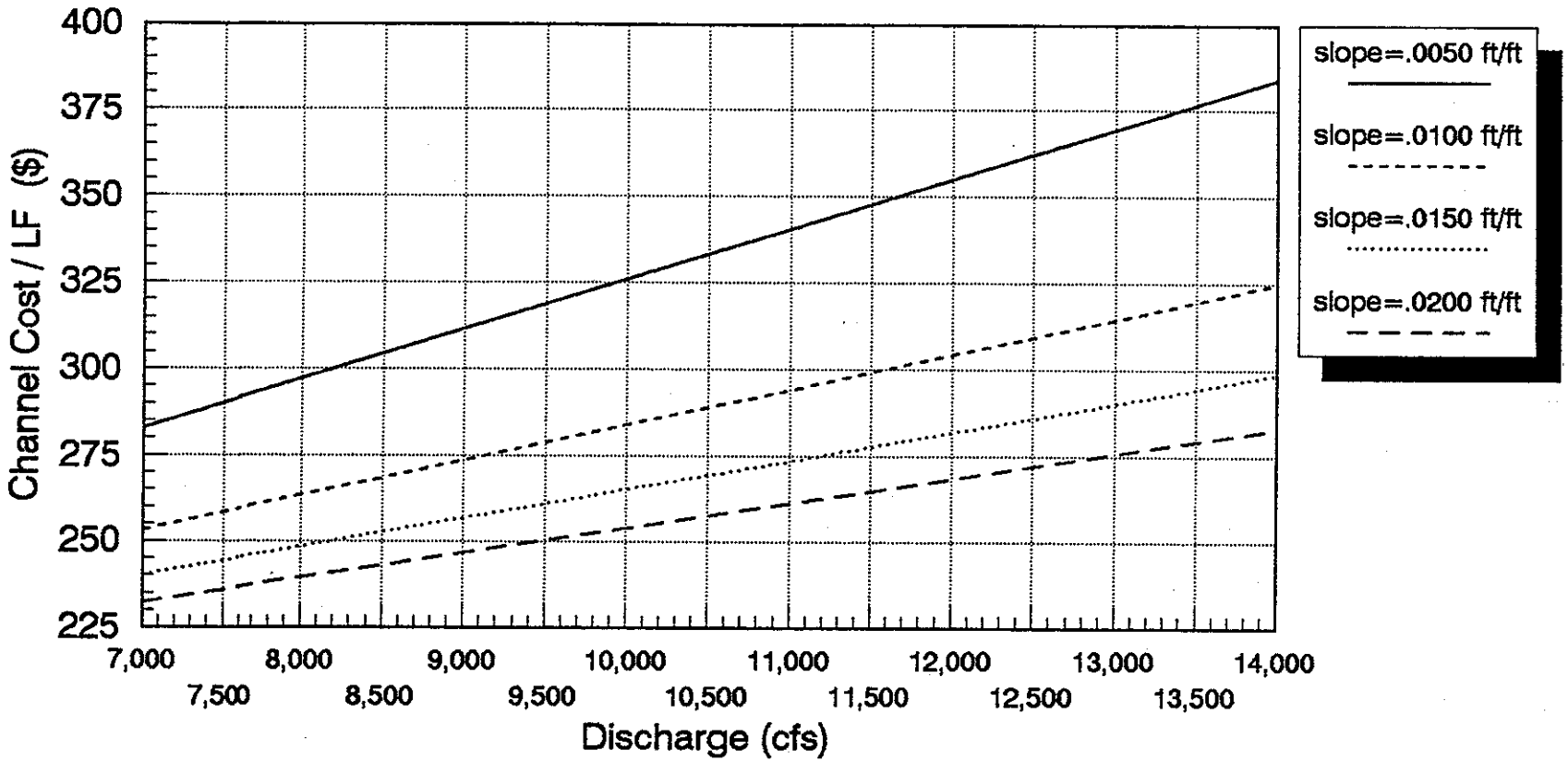
Figure B.16 Total Earth Lining, n = 0.050, Discharge vs. Channel R/W Width, 7,000 to 14,000 cfs



Flow depth = 4 ft, FB = 1.5 ft
n = .025, z = 2H:1V, Scour Toedown = 5 ft
Excavation @ $3/cy, Soil Cement @ $24/cy

Figure B.1

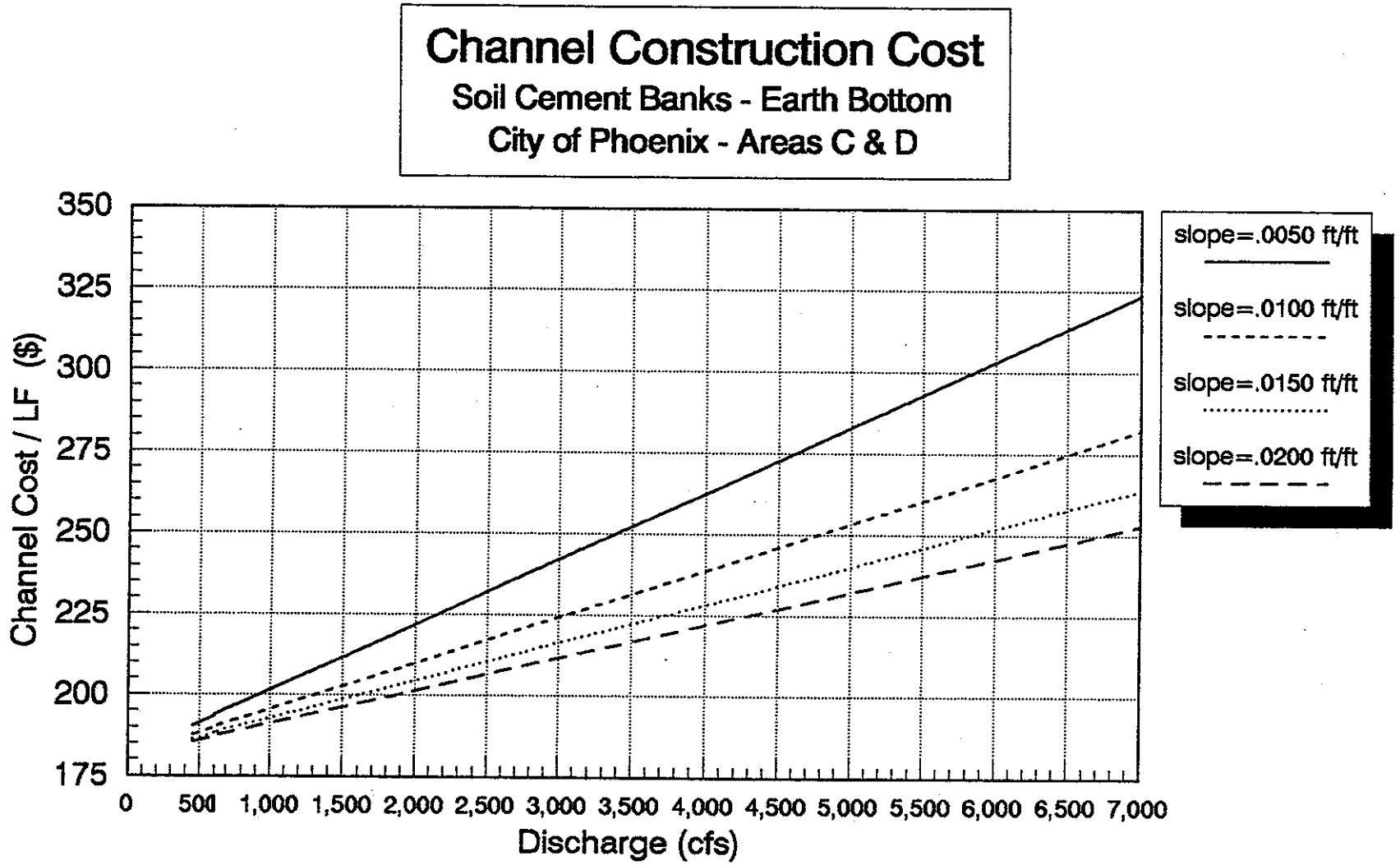
Water Resources Associates, Inc.



Flow depth = 4 ft, FB = 1.5 ft
n = .025, z = 2H:1V, Scour Toedown = 5 ft
Excavation @ $3/cy, Soil Cement @ $24/cy

Figure B.2

Water Resources Associates, Inc.



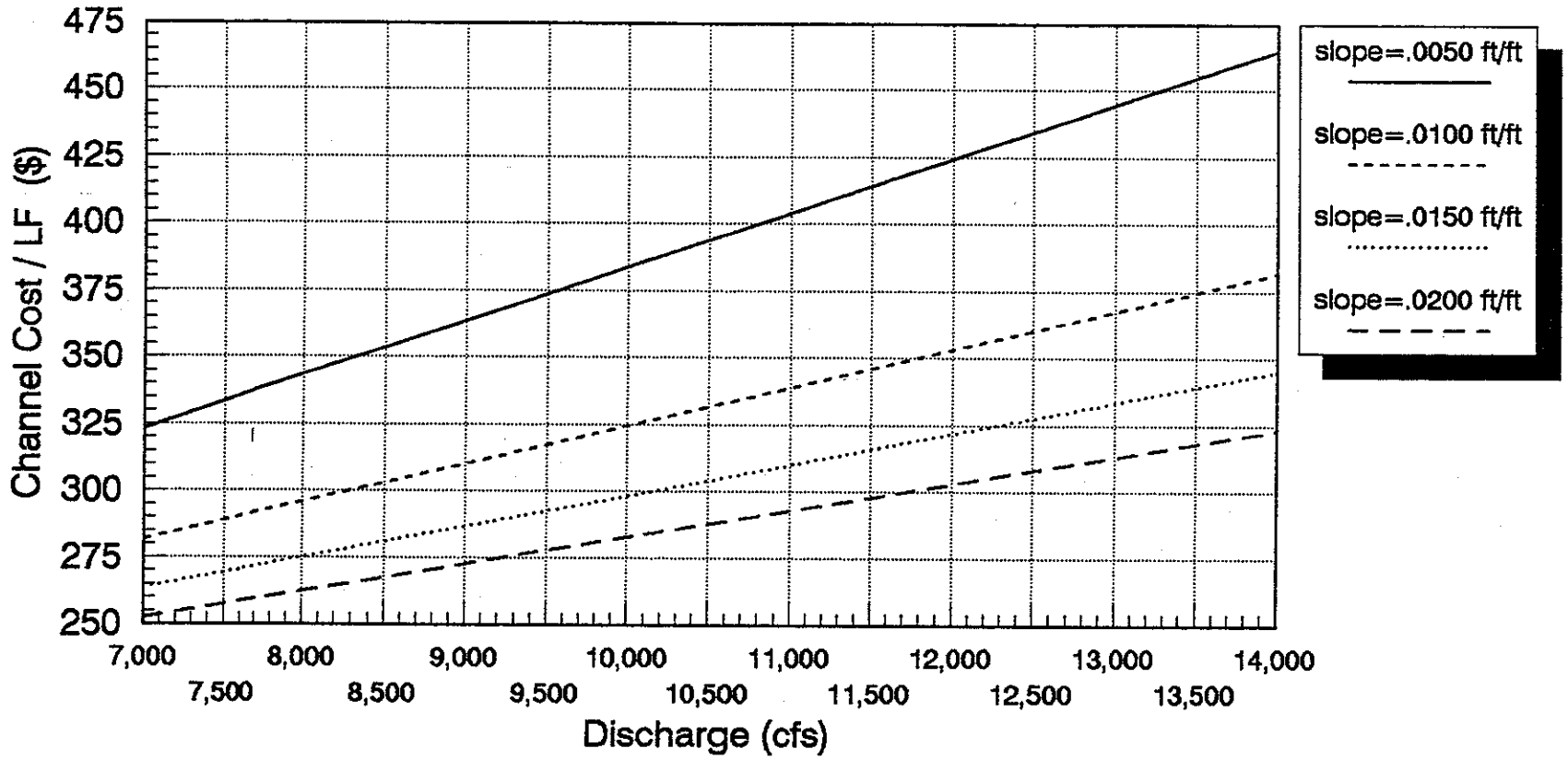
Flow depth = 4 ft, FB = 1.5 ft
n = .035, z = 2H:1V, Scour Toedown = 5 ft
Excavation @ $3/cy, Soil Cement @ $24/cy

Figure B.3

Water Resources Associates, Inc.

Channel Construction Cost

Soil Cement Banks - Earth Bottom
City of Phoenix - Areas C & D



Flow depth = 4 ft, FB = 1.5 ft
n = .035, z = 2H:1V, Scour Toedown = 5 ft
Excavation @ $3/cy, Soil Cement @ $24/cy

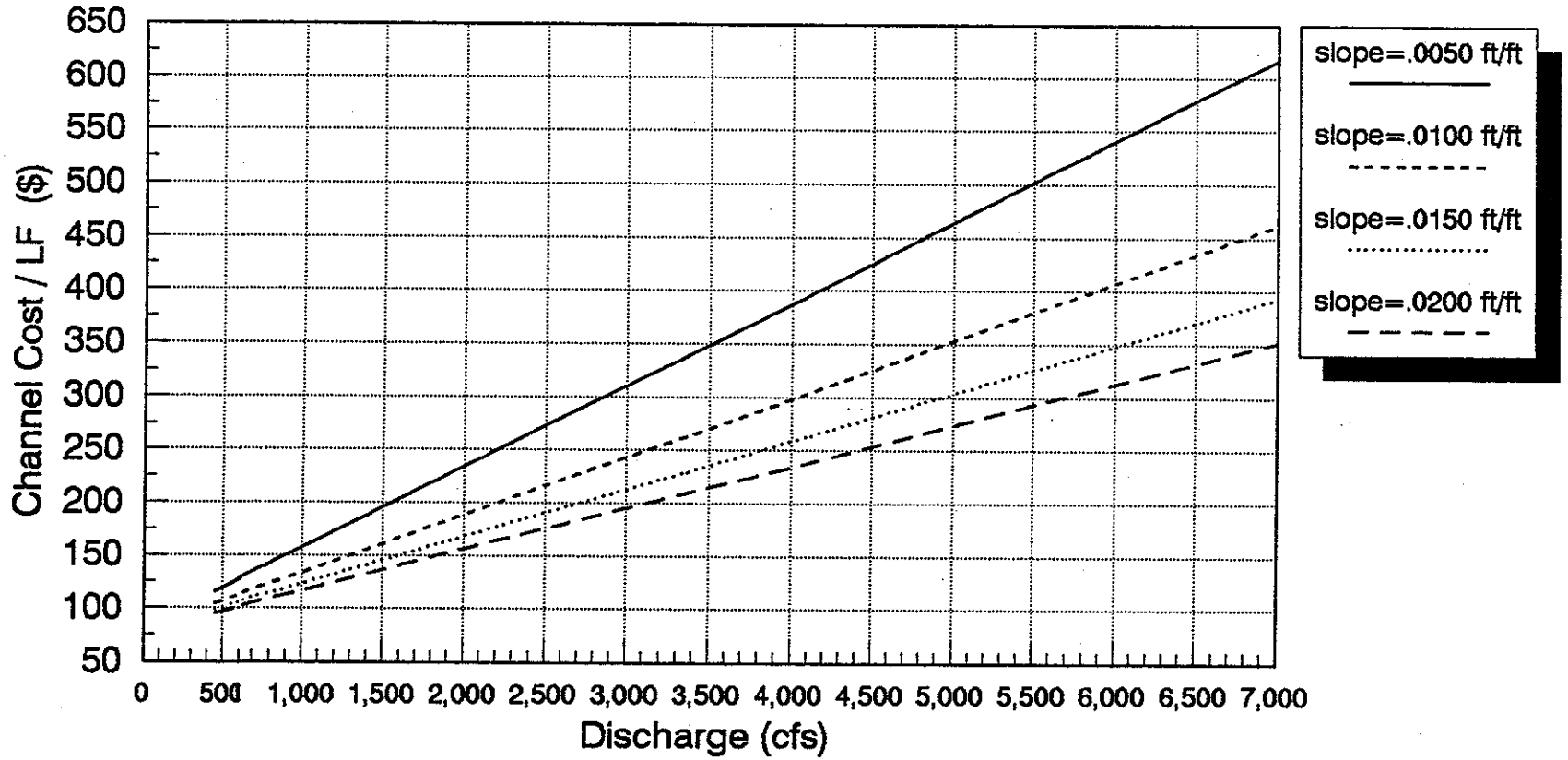
Figure B.4

Water Resources Associates, Inc.

Channel Construction Cost

Concrete Lining - Banks & Bottom (6" thick)

City of Phoenix - Areas C & D



Flow depth = 4 ft, FB = 1.5 ft
n = .020, z = 2H:1V
Excavation @ $3/cy, Concrete @ $180/cy

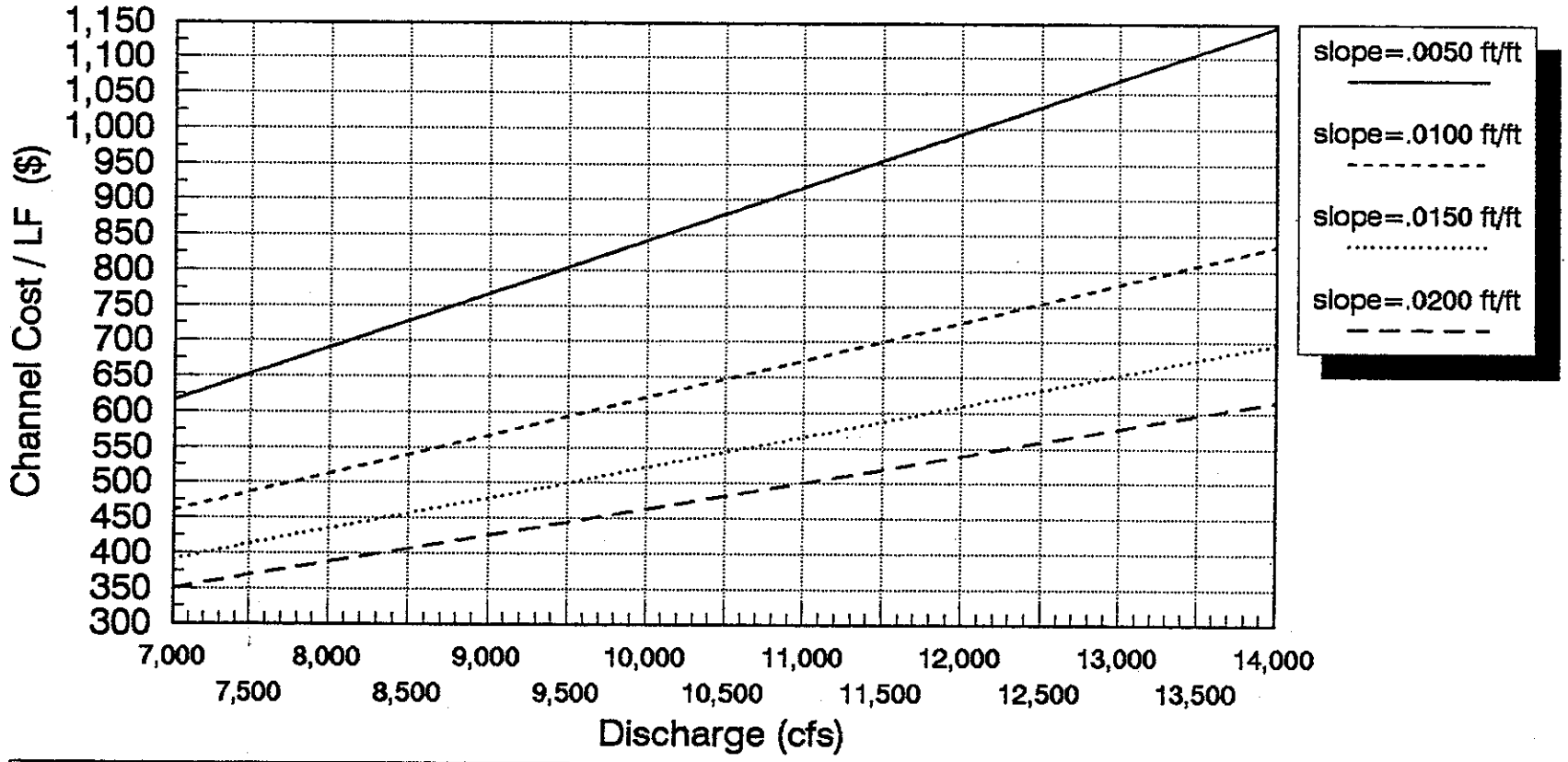
Figure B.5

Water Resources Associates, Inc.

Channel Construction Cost

Concrete Lining - Banks & Bottom (6" thick)

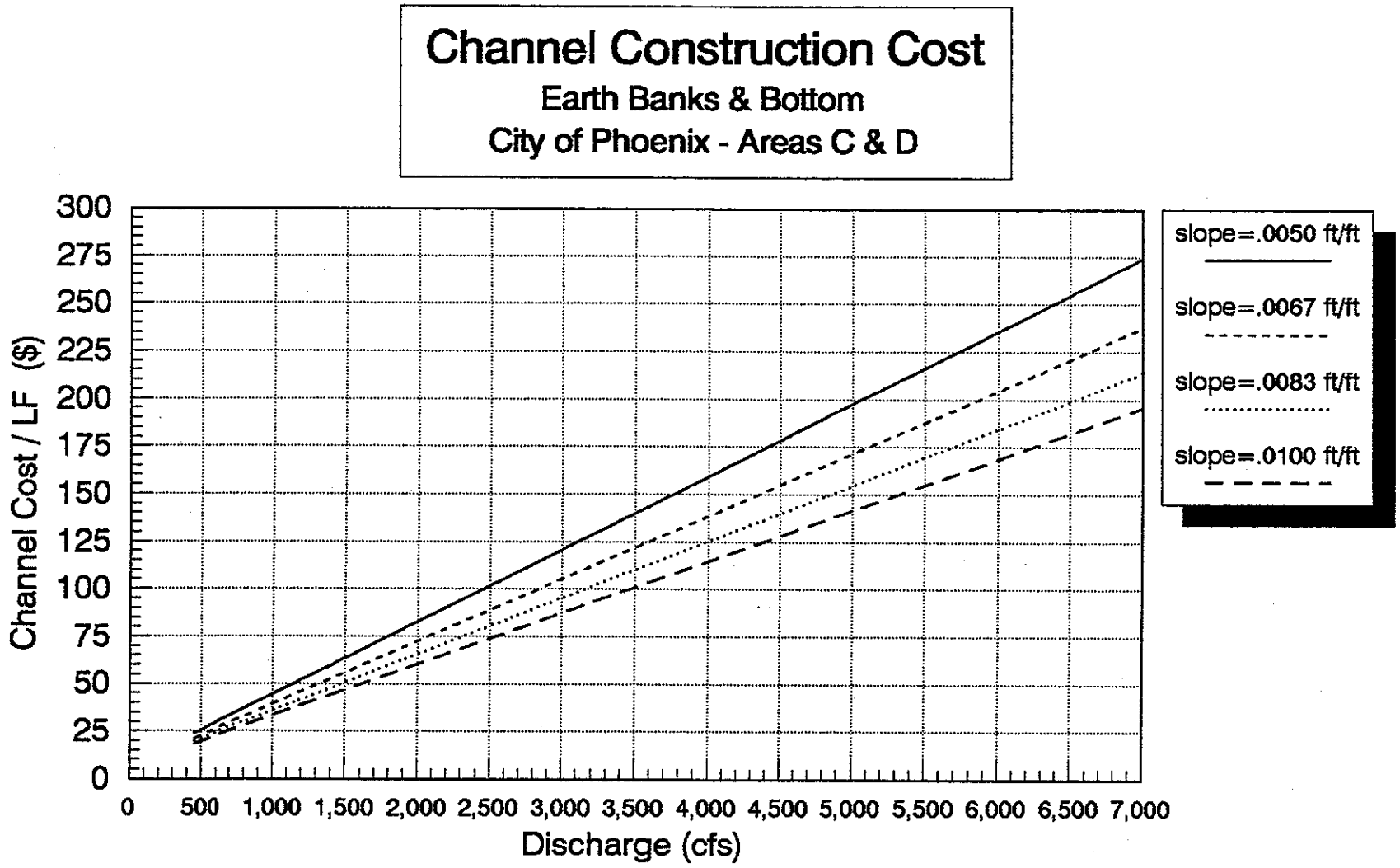
City of Phoenix - Areas C & D



Flow depth = 4 ft, FB = 1.5 ft
n = .020, z = 2H:1V
Excavation @ $3/cy, Concrete @ $180/cy

Figure B.6

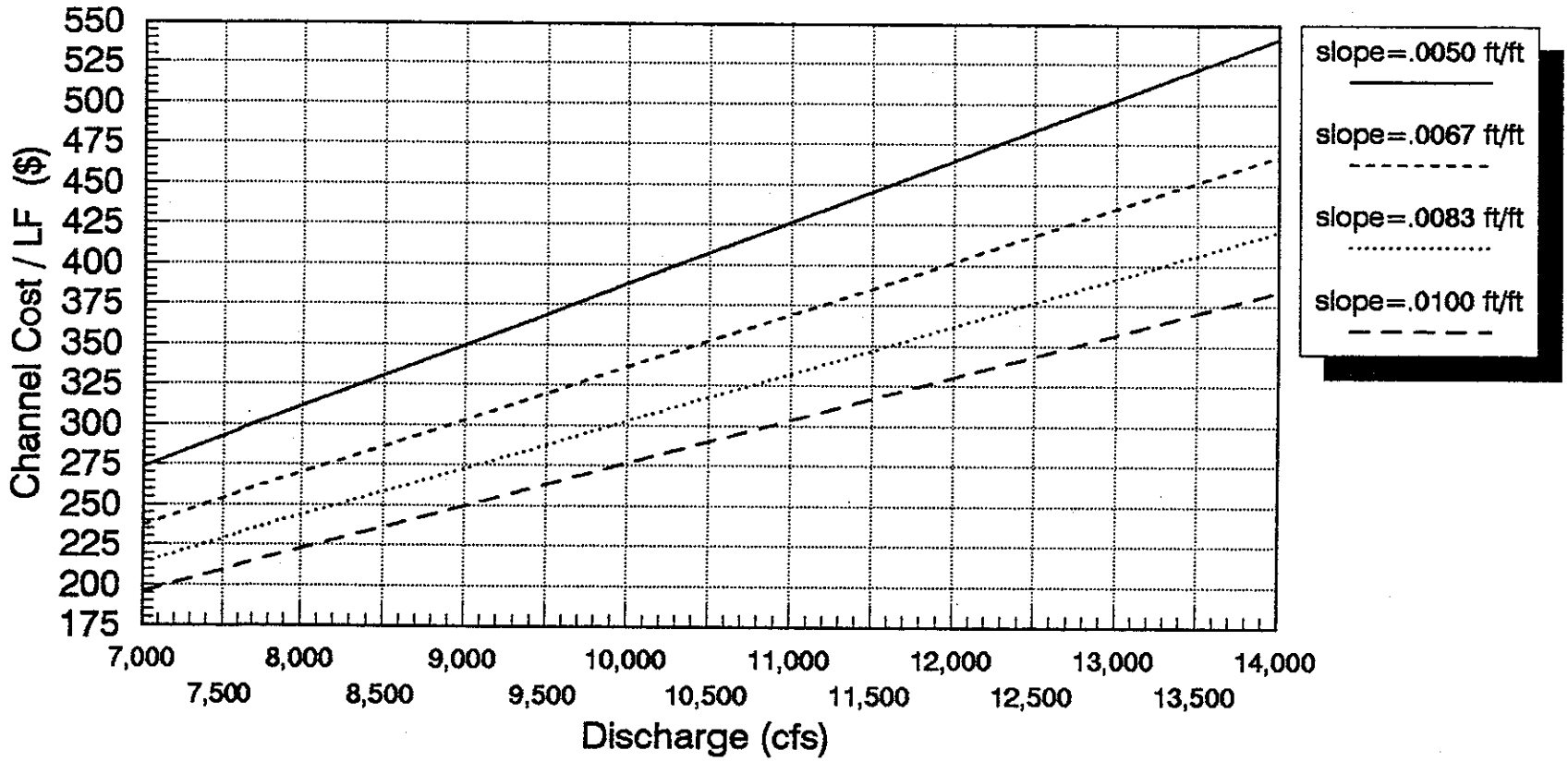
Water Resources Associates, Inc.



Flow depth = 3 ft, FB = 1.5 ft
n = .050, z = 4H:1V
Excavation @ $3/cy

Figure B.7

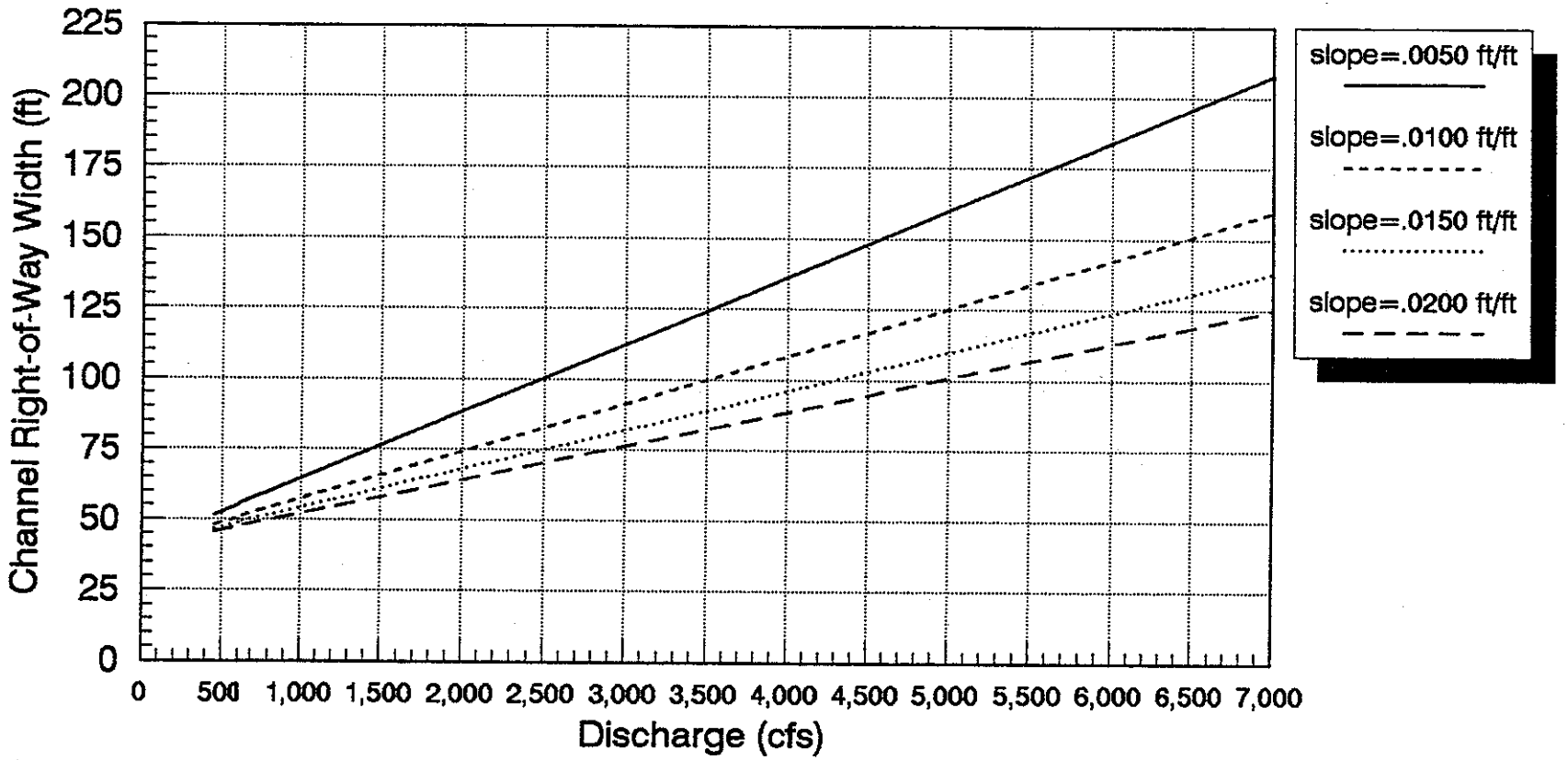
Water Resources Associates, Inc.



Flow depth = 3 ft, FB = 1.5 ft
n = .050, z = 4H:1V
Excavation @ $3/cy

Figure B.8

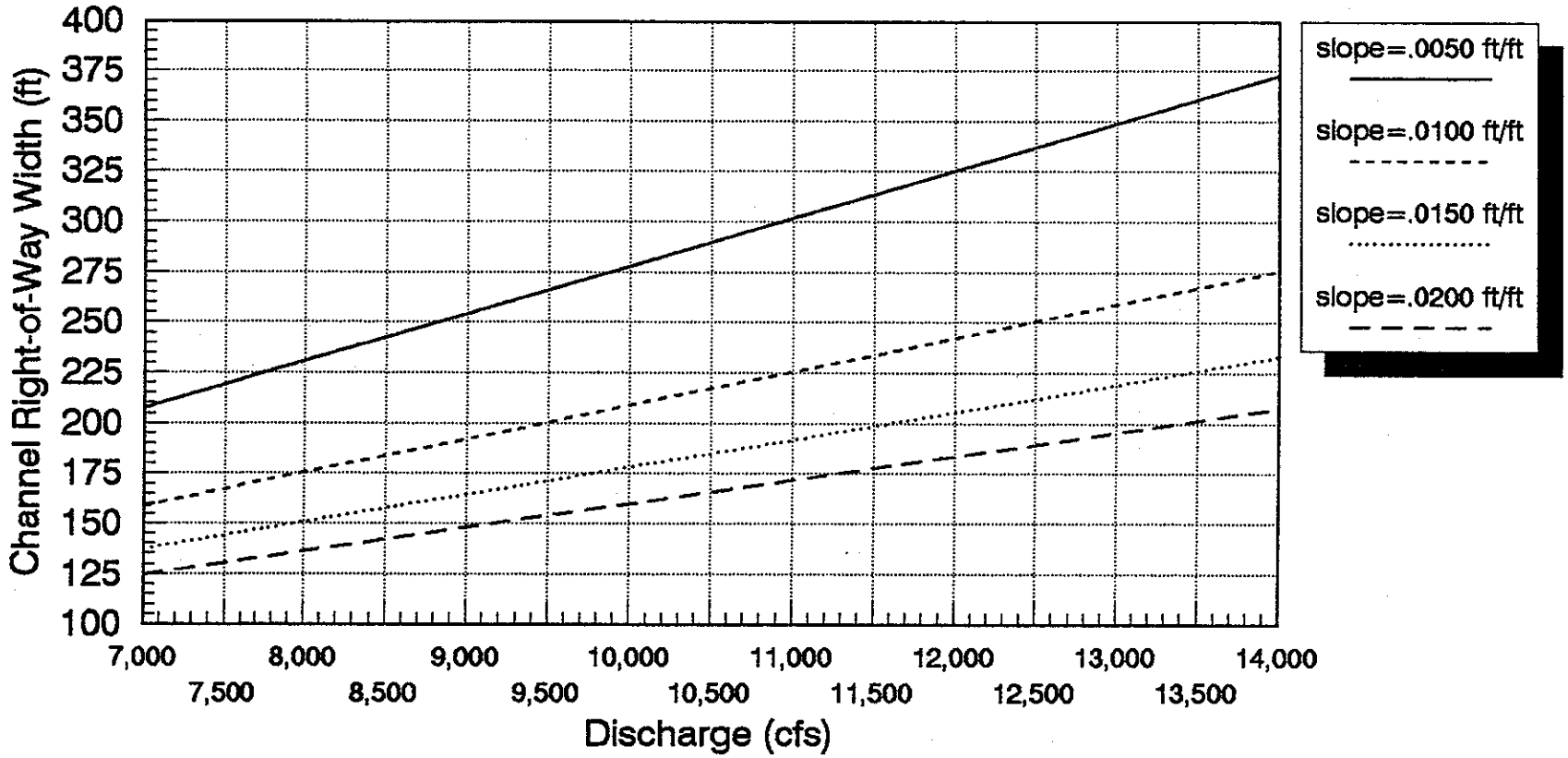
Water Resources Associates, Inc.



Flow depth = 4 ft, FB = 1.5 ft
n = .025, z = 2H:1V, Scour Toedown = 5 ft
Width includes channel FB & 12' maintenance road

Figure B.9

Water Resources Associates, Inc.



Flow depth = 4 ft, FB = 1.5 ft
n = .025, z = 2H:1V, Scour Toedown = 5 ft
Width includes channel FB & 12' maintenance road

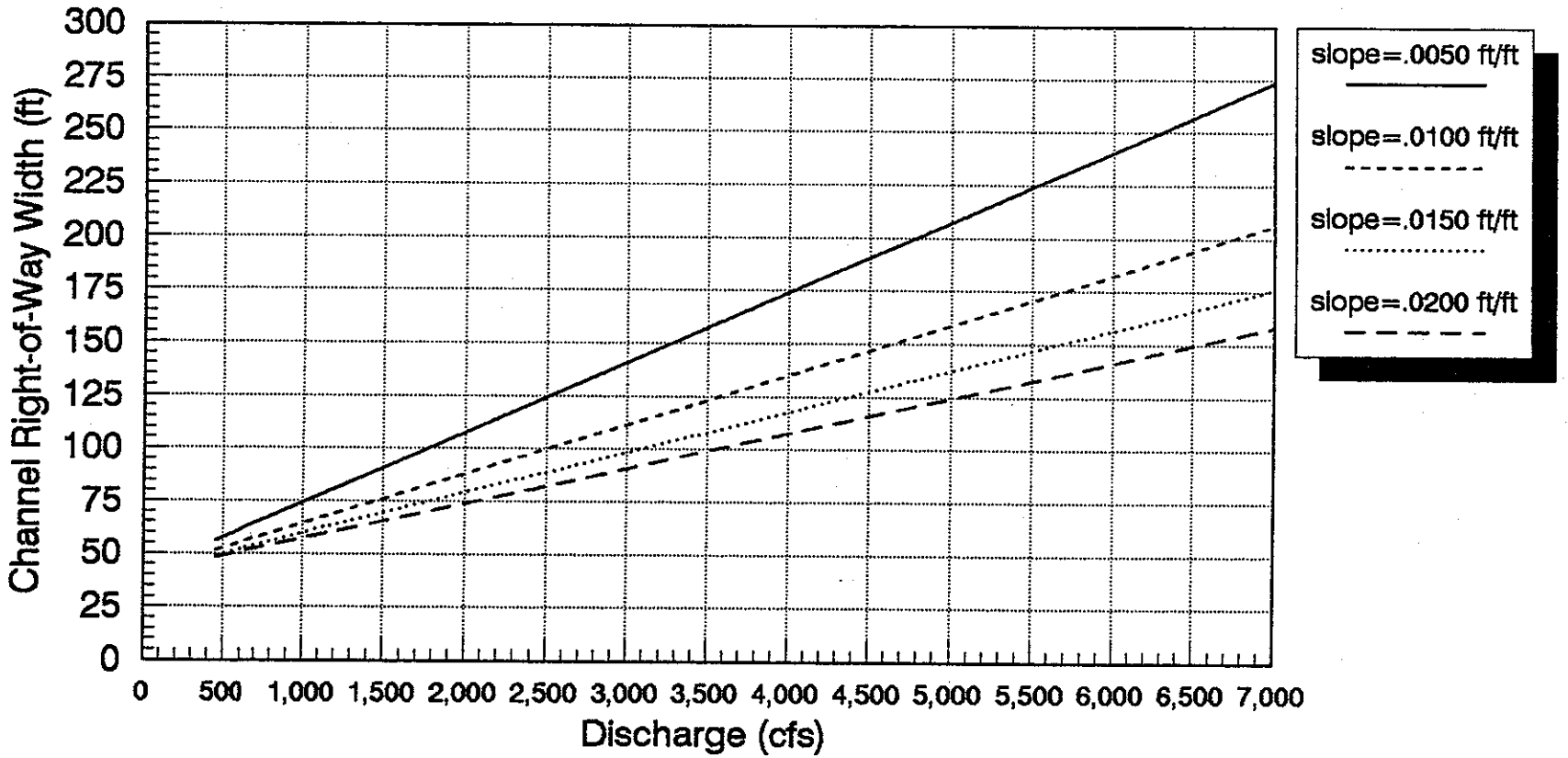
Figure B.10

Water Resources Associates, Inc.

Channel Right-of-Way Width (ft)

Soil Cement Banks - Earth Bottom

City of Phoenix - Areas C & D



Flow depth = 4 ft, FB = 1.5 ft
n = .035, z = 2H:1V, Scour Toedown = 5 ft
Width includes channel FB & 12' maintenance road

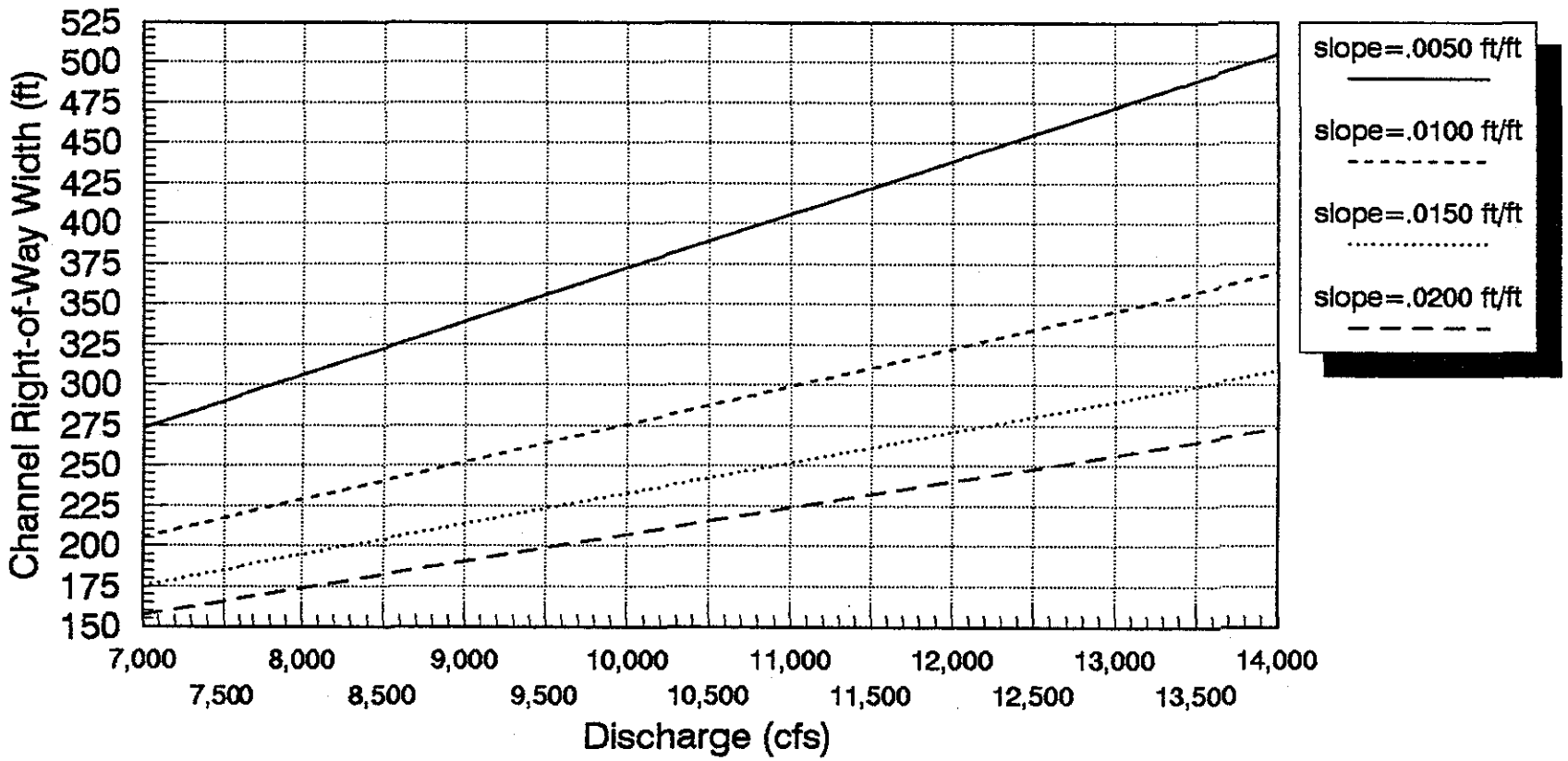
Figure B.11

Water Resources Associates, Inc.

Channel Right-of-Way Width (ft)

Soil Cement Banks - Earth Bottom

City of Phoenix - Areas C & D



Flow depth = 4 ft, FB = 1.5 ft
n = .035, z = 2H:1V, Scour Toedown = 5 ft
Width includes channel FB & 12' maintenance road

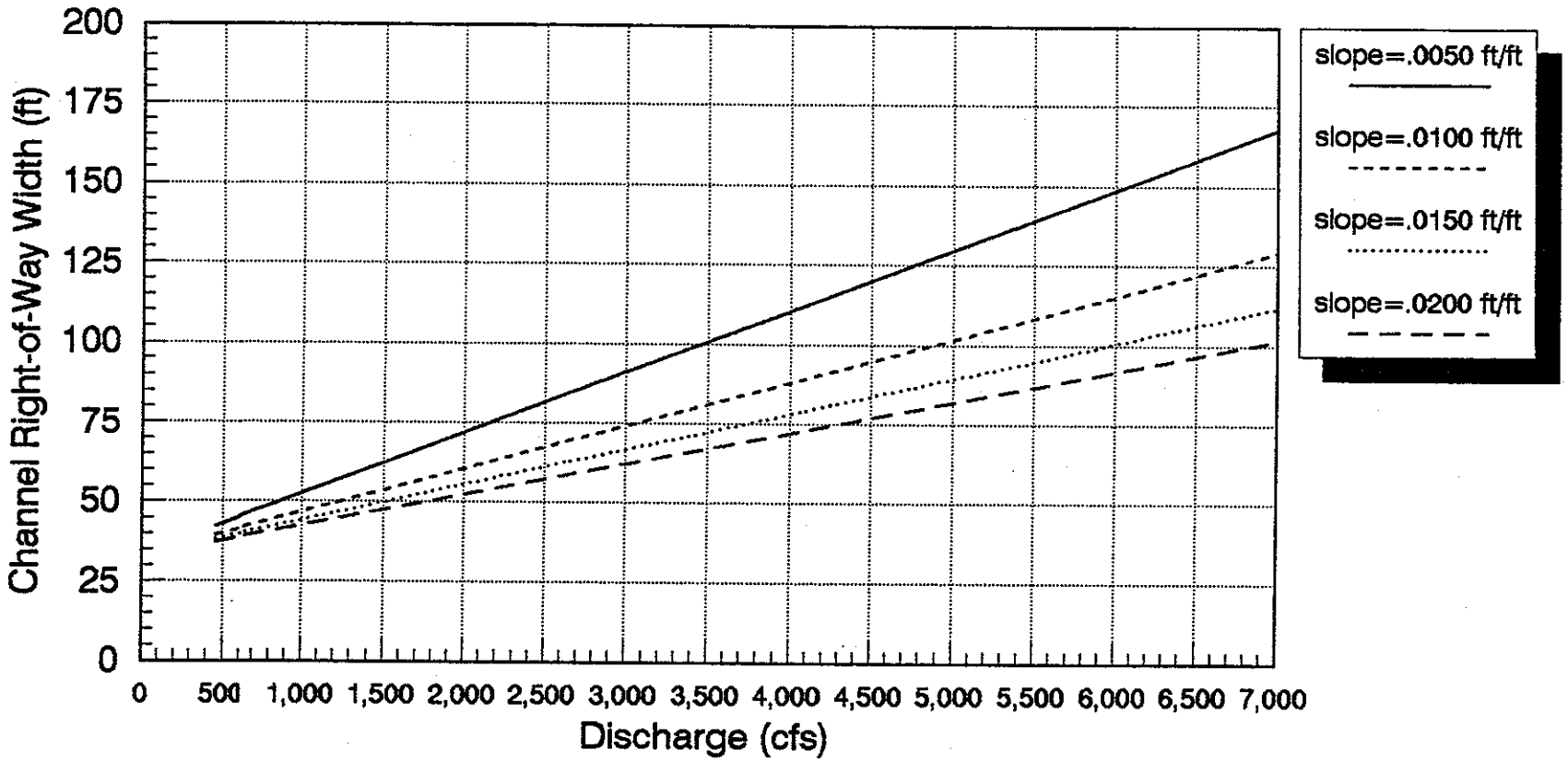
Figure B.12

Water Resources Associates, Inc.

Channel Right-of-Way Width (ft)

Concrete Lining - Banks & Bottom (6" thick)

City of Phoenix - Areas C & D



Flow depth = 4 ft, FB = 1.5 ft
n = .020, z = 2H:1V
Width includes channel FB & 12' maintenance road

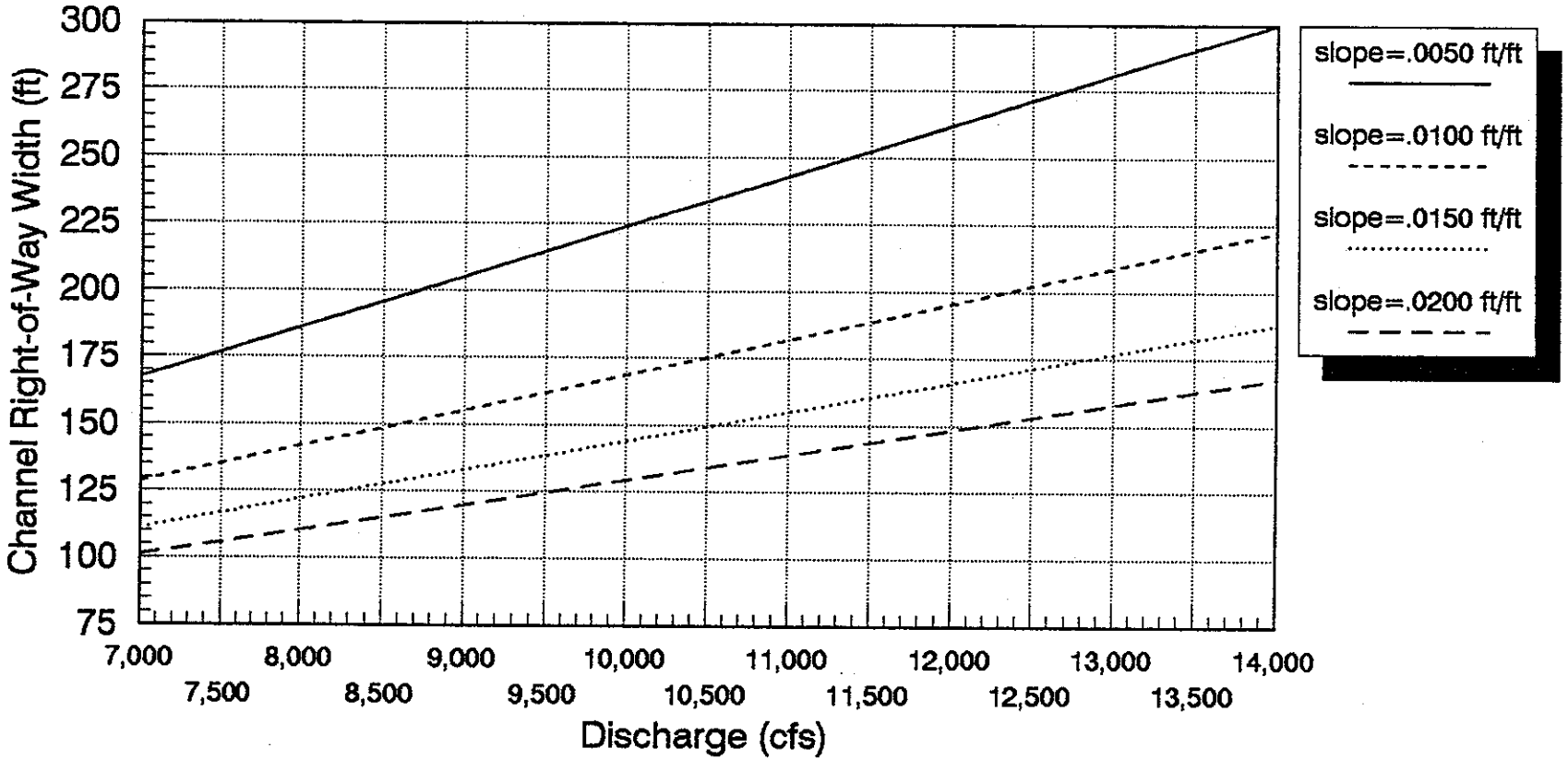
Figure B.13

Water Resources Associates, Inc.

Channel Right-of-Way Width (ft)

Concrete Lining - Banks & Bottom (6" thick)

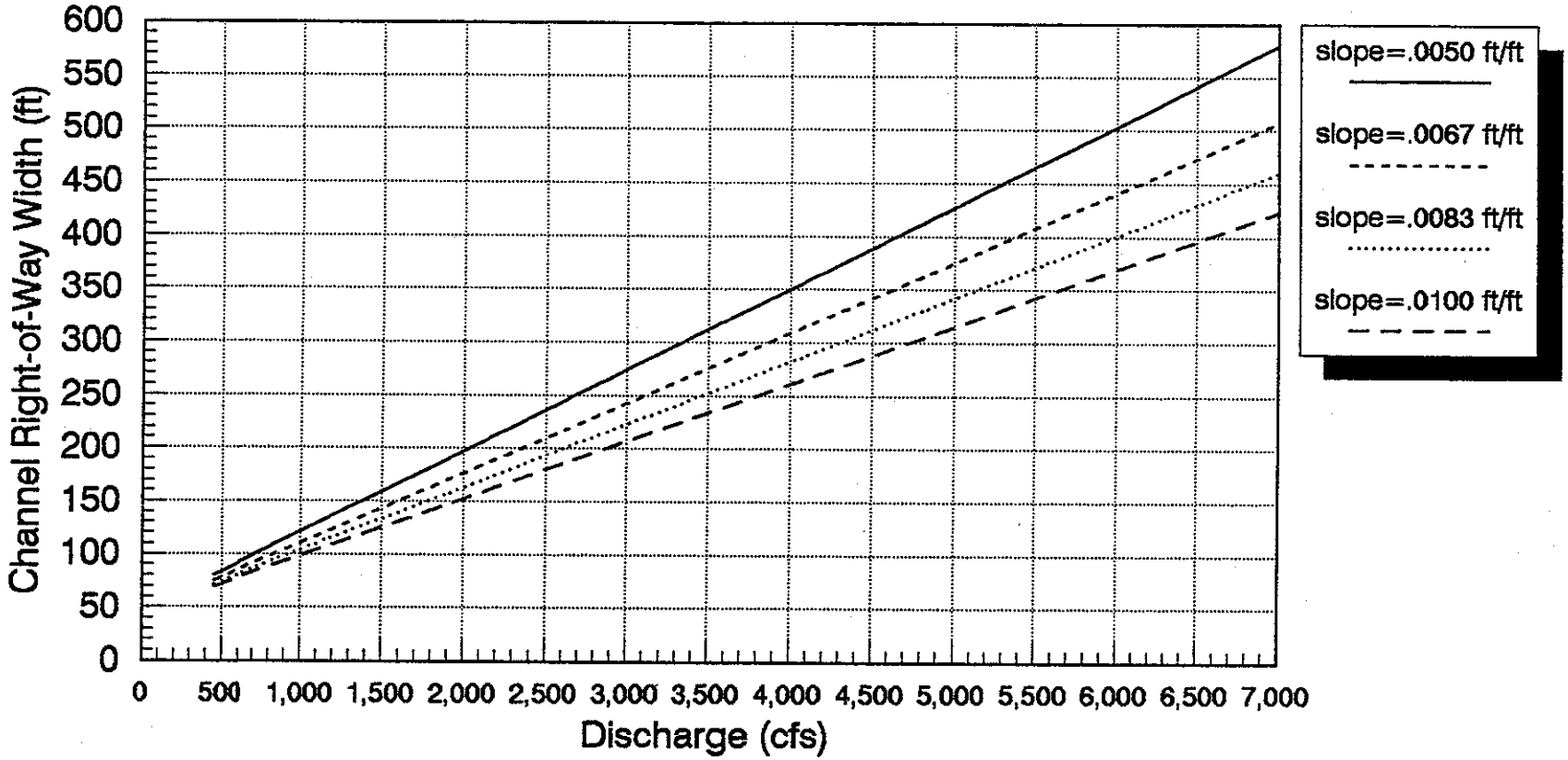
City of Phoenix - Areas C & D



Flow depth = 4 ft, FB = 1.5 ft
n = .020, z = 2H:1V
Width includes channel FB & 12' maintenance road

Figure B.14

Water Resources Associates, Inc.



Flow depth = 3 ft, FB = 1.5 ft
n = .050, z = 4H:1V
Width includes channel FB & 12' maintenance road

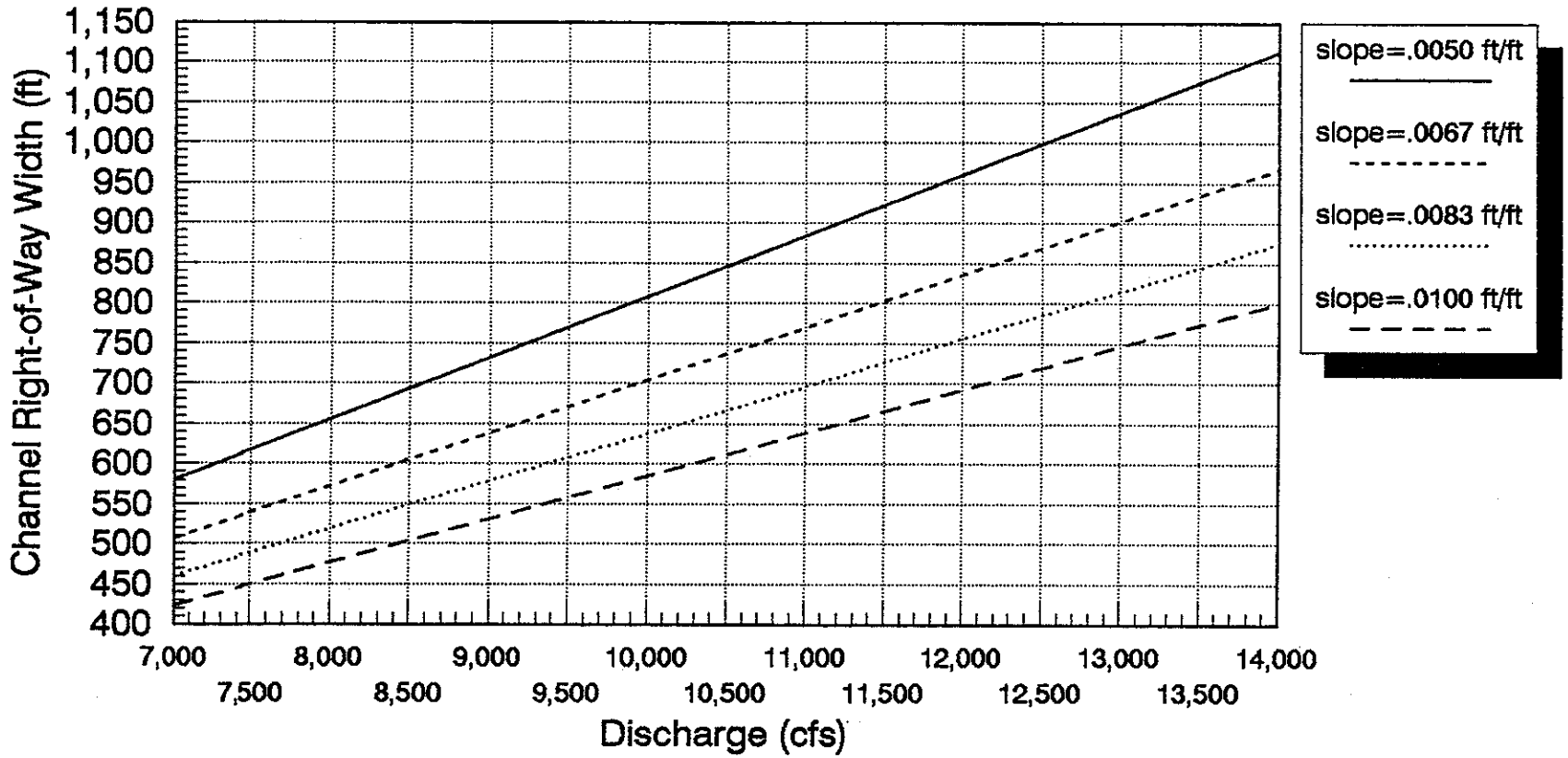
Figure B.15

Water Resources Associates, Inc.

Channel Right-of-Way Width (ft)

Earth Banks & Bottom

City of Phoenix - Areas C & D



Flow depth = 3 ft, FB = 1.5 ft
n = .050, z = 4H:1V
Width includes channel FB & 12' maintenance road

Figure B.16

Water Resources Associates, Inc.

APPENDIX C

HYDRAULIC AND COST DATA FOR DESERT RIDGE CHANNELS

Table C.1	East-West Channel from Jomax Road, Concrete Lining, No Low-Flow Culverts
Table C.2	East-West Channel from Jomax Road, Earth Lining, No Low-Flow Culverts
Table C.3	East-West Channel from Jomax Road, Soil-Cement Banks, n = 0.025, No Low-Flow Culverts
Table C.4	East-West Channel from Jomax Road, Soil-Cement Banks, n = 0.035, No Low-Flow Culverts
Table C.5	East-West Channel along Pinnacle Peak Road, Soil-Cement Banks, n = 0.035, No Low-Flow Culverts
Table C.6	North-South Channel along Scottsdale Road, Concrete Lining, No Low-Flow Culverts
Table C.7	North-South Channel along Scottsdale Road, Earth Lining, No Low-Flow Culverts
Table C.8	North-South Channel along Scottsdale Road, Soil-Cement Banks, n = 0.025, No Low-Flow Culverts
Table C.9	North-South Channel along Scottsdale Road, Soil-Cement Banks, n = 0.035, No Low-Flow Culverts
Table C.10	East-West Channel from Jomax Road, Concrete Lining, With Low-Flow Culverts
Table C.11	East-West Channel from Jomax Road, Earth Lining, With Low-Flow Culverts
Table C.12	East-West Channel from Jomax Road, Soil-Cement Banks, n = 0.035, With Low-Flow Culverts
Table C.13	East-West Channel along Pinnacle Peak Road, Soil-Cement Banks, n = 0.035, With Low-Flow Culverts
Table C.14	North-South Channel along Scottsdale Road, Concrete Lining, With Low-Flow Culverts
Table C.15	North-South Channel Along Scottsdale Road, Earth Lining, With Low-Flow Culverts
Table C.16	North-South Channel along Scottsdale Road, Soil-Cement Banks, n = 0.035, With Low-Flow Culverts

TABLE C.1
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $3.00
Channel Lining: Concrete (banks & bottom)
Horizontal Lining Width (ft): 1.12
Lining Cost/c.y.: $180.00
All Calculations Are Based On Existing Ground Slopes

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 22+30	1730	4633	0.0058	0.020	2.00	4.0	79.4	13.3	1.22	5.5	101.4
STA 22+30 to 42+30	2000	4633	0.0100	0.020	2.00	4.0	59.7	17.1	1.59	5.5	81.7
STA 42+30 to 68+90	2660	4633	0.0075	0.020	2.00	4.0	69.3	15.0	1.39	5.5	91.3
STA 68+90 to 90+80	2190	4633	0.0091	0.020	2.00	4.0	62.6	16.4	1.52	5.5	84.6
STA 90+80 to 106+80	1600	4616	0.0125	0.020	2.00	4.0	52.9	18.9	1.77	5.5	74.9
STA 106+80 to 125+10	1830	4616	0.0109	0.020	2.00	4.0	56.8	17.8	1.66	5.5	78.8
STA 125+10 to 141+20	1610	4616	0.0124	0.020	2.00	4.0	53.1	18.9	1.77	5.5	75.1
STA 141+20 to 166+20	2500	4478	0.0120	0.020	2.00	4.0	52.4	18.5	1.74	5.5	74.4
STA 166+20 to 173+10	690	4478	0.0145	0.020	2.00	4.0	47.4	20.2	1.90	5.5	69.4
STA 173+10 to 184+30	1120	4478	0.0089	0.020	2.00	4.0	61.2	16.2	1.51	5.5	83.2
STA 184+30 to 197+30	1300	4478	0.0154	0.020	2.00	4.0	45.9	20.8	1.96	5.5	67.9
STA 197+30 to 214+60	1730	3846	0.0116	0.020	2.00	4.0	45.5	18.0	1.70	5.5	67.5
STA 214+60 to 227+80	1320	3846	0.0152	0.020	2.00	4.0	39.3	20.3	1.93	5.5	61.3
STA 227+80 to 238+90	1110	3846	0.0180	0.020	2.00	4.0	35.8	21.9	2.10	5.5	57.8
STA 238+90 to 252+80	1390	3846	0.0144	0.020	2.00	4.0	40.5	19.8	1.89	5.5	62.5
STA 252+80 to 262+60	980	3846	0.0204	0.020	2.00	4.0	33.5	23.2	2.23	5.5	55.5
STA 262+60 to 274+50	1190	3846	0.0168	0.020	2.00	4.0	37.2	21.3	2.03	5.5	59.2
STA 274+50 to 288+80	1430	3846	0.0140	0.020	2.00	4.0	41.1	19.6	1.86	5.5	63.1
STA 288+80 to 303+90	1510	2527	0.0132	0.020	2.00	4.0	26.7	18.2	1.78	5.5	48.7
STA 303+90 to 314+90	1100	2527	0.0164	0.020	2.00	4.0	23.6	20.0	1.97	5.5	45.6
STA 314+90 to 323+80	890	2527	0.0135	0.020	2.00	4.0	26.4	18.4	1.80	5.5	48.4
STA 323+80 to 335+80	1200	2527	0.0167	0.020	2.00	4.0	23.4	20.1	1.99	5.5	45.4
STA 335+80 to 349+50	1370	2527	0.0146	0.020	2.00	4.0	25.2	19.0	1.87	5.5	47.2
STA 349+50 to 364+20	1470	752	0.0136	0.020	2.00	4.0	4.7	14.9	1.67	5.5	26.7
STA 364+20 to 375+00	1080	752	0.0167	0.020	2.00	4.0	3.7	16.1	1.84	5.5	25.7
STA 375+00 to 389+00	1400	752	0.0086	0.020	2.00	4.0	7.2	12.4	1.35	5.5	29.2
TOTALS:	38400										

TABLE C.1
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $3.00
Channel Lining: Concrete (banks & bottom)
Horizontal Lining Width (ft): 1.12
Lining Cost/c.y.: $180.00
All Calculations Are Based On Existing Ground Slopes

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 22+30	0.0	35229	3373	$105,688.27	$607,098.69	$712,786.96	4.6
STA 22+30 to 42+30	0.0	31994	3171	$95,982.09	$570,843.79	$666,825.88	4.4
STA 42+30 to 68+90	0.0	48203	4689	$144,609.24	$843,987.52	$988,596.75	6.5
STA 68+90 to 90+80	0.0	36444	3590	$109,331.79	$646,230.96	$755,562.75	5.0
STA 90+80 to 106+80	0.0	23176	2335	$69,528.54	$420,389.39	$489,917.94	3.3
STA 106+80 to 125+10	0.0	28082	2802	$84,245.08	$504,429.39	$588,674.46	3.9
STA 125+10 to 141+20	0.0	23384	2355	$70,150.87	$423,955.66	$494,106.53	3.3
STA 141+20 to 166+20	0.0	35907	3624	$107,720.99	$652,271.63	$759,992.62	5.1
STA 166+20 to 173+10	0.0	9147	937	$27,441.23	$168,578.17	$196,019.41	1.3
STA 173+10 to 184+30	0.0	18273	1806	$54,817.74	$325,011.35	$379,829.09	2.5
STA 184+30 to 197+30	0.0	16806	1729	$50,417.17	$311,192.53	$361,609.71	2.5
STA 197+30 to 214+60	0.0	22191	2286	$66,573.66	$411,525.64	$478,099.30	3.3
STA 214+60 to 227+80	0.0	15132	1594	$45,395.00	$286,991.02	$332,386.03	2.3
STA 227+80 to 238+90	0.0	11854	1268	$35,562.76	$228,281.78	$263,844.53	1.9
STA 238+90 to 252+80	0.0	16277	1708	$48,831.66	$307,356.98	$356,188.65	2.5
STA 252+80 to 262+60	0.0	9952	1077	$29,856.66	$193,840.62	$223,697.27	1.6
STA 262+60 to 274+50	0.0	13074	1390	$39,221.55	$250,213.11	$289,434.66	2.0
STA 274+50 to 288+80	0.0	16943	1773	$50,829.73	$319,165.98	$369,995.71	2.6
STA 288+80 to 303+90	0.0	13053	1469	$39,159.49	$264,452.10	$303,611.58	2.2
STA 303+90 to 314+90	0.0	8767	1008	$26,300.97	$181,518.17	$207,819.14	1.5
STA 314+90 to 323+80	0.0	7641	862	$22,921.86	$155,074.65	$177,996.51	1.3
STA 323+80 to 335+80	0.0	9498	1095	$28,492.59	$197,022.96	$225,515.55	1.7
STA 335+80 to 349+50	0.0	11406	1297	$34,218.25	$233,380.59	$267,598.84	2.0
STA 349+50 to 364+20	0.0	5518	831	$16,554.23	$149,607.13	$166,161.36	1.4
STA 364+20 to 375+00	0.0	3826	592	$11,477.95	$106,493.75	$117,971.70	1.0
STA 375+00 to 389+00	0.0	6035	857	$18,105.73	$154,181.97	$172,287.70	1.4
TOTALS:		477812	49517	$1,433,435.11	$8,913,095.53	$10,346,530.64	71.3

TABLE C.2
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

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	Average Channel Depth (ft)	Channel Topwidth (ft)	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)
Start @ 5+00													
STA 5+00 to 22+30	1730	4633	0.0067	0.050	4.00	3.0	301.4	4.9	0.51	6.0	349.4	0.0	125101
STA 22+30 to 42+30	2000	4633	0.0067	0.050	4.00	3.0	301.4	4.9	0.51	6.0	349.4	0.0	144625
STA 42+30 to 68+90	2660	4633	0.0067	0.050	4.00	3.0	301.4	4.9	0.51	6.0	349.4	0.0	192351
STA 68+90 to 90+80	2190	4633	0.0067	0.050	4.00	3.0	301.4	4.9	0.51	6.0	349.4	0.0	158364
STA 90+80 to 106+80	1600	4616	0.0067	0.050	4.00	3.0	300.3	4.9	0.51	6.0	348.3	0.0	115301
STA 106+80 to 125+10	1830	4616	0.0067	0.050	4.00	3.0	300.3	4.9	0.51	6.0	348.3	0.0	131877
STA 125+10 to 141+20	1610	4616	0.0067	0.050	4.00	3.0	300.3	4.9	0.51	6.0	348.3	0.0	116022
STA 141+20 to 166+20	2500	4478	0.0067	0.050	4.00	3.0	291.2	4.9	0.51	6.0	339.2	0.0	175104
STA 166+20 to 173+10	690	4478	0.0067	0.050	4.00	3.0	291.2	4.9	0.51	6.0	339.2	0.0	48328
STA 173+10 to 184+30	1120	4478	0.0067	0.050	4.00	3.0	291.2	4.9	0.51	6.0	339.2	0.0	78447
STA 184+30 to 197+30	1300	4478	0.0067	0.050	4.00	3.0	291.2	4.9	0.51	6.0	339.2	0.0	91054
STA 197+30 to 214+60	1730	3846	0.0067	0.050	4.00	3.0	249.5	4.9	0.51	6.0	297.5	0.0	105149
STA 214+60 to 227+80	1320	3846	0.0067	0.050	4.00	3.0	249.5	4.9	0.51	6.0	297.5	0.0	80229
STA 227+80 to 238+90	1110	3846	0.0067	0.050	4.00	3.0	249.5	4.9	0.51	6.0	297.5	0.0	67465
STA 238+90 to 252+80	1390	3846	0.0067	0.050	4.00	3.0	249.5	4.9	0.51	6.0	297.5	0.0	84484
STA 252+80 to 262+60	980	3846	0.0067	0.050	4.00	3.0	249.5	4.9	0.51	6.0	297.5	0.0	59564
STA 262+60 to 274+50	1190	3846	0.0067	0.050	4.00	3.0	249.5	4.9	0.51	6.0	297.5	0.0	72328
STA 274+50 to 288+80	1430	3846	0.0067	0.050	4.00	3.0	249.5	4.9	0.51	6.0	297.5	0.0	86915
STA 288+80 to 303+90	1510	2527	0.0067	0.050	4.00	3.0	162.5	4.8	0.51	6.0	210.5	0.0	62569
STA 303+90 to 314+90	1100	2527	0.0067	0.050	4.00	3.0	162.5	4.8	0.51	6.0	210.5	0.0	45582
STA 314+90 to 323+80	890	2527	0.0067	0.050	4.00	3.0	162.5	4.8	0.51	6.0	210.5	0.0	36880
STA 323+80 to 335+80	1200	2527	0.0067	0.050	4.00	3.0	162.5	4.8	0.51	6.0	210.5	0.0	49725
STA 335+80 to 349+50	1370	2527	0.0067	0.050	4.00	3.0	162.5	4.8	0.51	6.0	210.5	0.0	56770
STA 349+50 to 364+20	1470	752	0.0067	0.050	4.00	3.0	44.7	4.4	0.50	6.0	92.7	0.0	22444
STA 364+20 to 375+00	1080	752	0.0067	0.050	4.00	3.0	44.7	4.4	0.50	6.0	92.7	0.0	16488
STA 375+00 to 389+00	1400	752	0.0067	0.050	4.00	3.0	44.7	4.4	0.50	6.0	92.7	0.0	21373
TOTALS:	38400												2244539

TABLE C.2
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

Channel Reach	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 12 ft Maintenance Road (acres)	Number Of Drop Structures In This Reach	Unit Discharge For This Reach (cfs/ft)	Drop Structure Cost Per LF Of Channel Bottomwidth
Start @ 5+00								
STA 5+00 to 22+30	0	$250,201.90	$0.00	$250,201.90	14.4	0	15.4	$99
STA 22+30 to 42+30	0	$289,250.09	$0.00	$289,250.09	16.7	2	15.4	$99
STA 42+30 to 68+90	0	$384,702.62	$0.00	$384,702.62	22.2	0	15.4	$99
STA 68+90 to 90+80	0	$316,728.85	$0.00	$316,728.85	18.3	2	15.4	$99
STA 90+80 to 106+80	0	$230,602.63	$0.00	$230,602.63	13.3	3	15.4	$99
STA 106+80 to 125+10	0	$263,753.09	$0.00	$263,753.09	15.2	3	15.4	$99
STA 125+10 to 141+20	0	$232,043.91	$0.00	$232,043.91	13.4	3	15.4	$99
STA 141+20 to 166+20	0	$350,207.04	$0.00	$350,207.04	20.3	5	15.4	$99
STA 166+20 to 173+10	0	$96,656.93	$0.00	$96,656.93	5.6	1	15.4	$99
STA 173+10 to 184+30	0	$156,893.12	$0.00	$156,893.12	9.1	1	15.4	$99
STA 184+30 to 197+30	0	$182,108.57	$0.00	$182,108.57	10.5	4	15.4	$99
STA 197+30 to 214+60	0	$210,298.72	$0.00	$210,298.72	12.4	3	15.4	$99
STA 214+60 to 227+80	0	$160,458.34	$0.00	$160,458.34	9.4	4	15.4	$99
STA 227+80 to 238+90	0	$134,930.28	$0.00	$134,930.28	7.9	4	15.4	$99
STA 238+90 to 252+80	0	$168,967.68	$0.00	$168,967.68	9.9	3	15.4	$99
STA 252+80 to 262+60	0	$119,127.17	$0.00	$119,127.17	7.0	5	15.4	$99
STA 262+60 to 274+50	0	$144,655.26	$0.00	$144,655.26	8.5	4	15.4	$99
STA 274+50 to 288+80	0	$173,830.16	$0.00	$173,830.16	10.2	3	15.4	$99
STA 288+80 to 303+90	0	$125,137.66	$0.00	$125,137.66	7.8	4	15.6	$100
STA 303+90 to 314+90	0	$91,163.03	$0.00	$91,163.03	5.7	4	15.6	$100
STA 314+90 to 323+80	0	$73,759.19	$0.00	$73,759.19	4.6	2	15.6	$100
STA 323+80 to 335+80	0	$99,450.58	$0.00	$99,450.58	6.2	4	15.6	$100
STA 335+80 to 349+50	0	$113,539.42	$0.00	$113,539.42	7.1	3	15.6	$100
STA 349+50 to 364+20	0	$44,888.74	$0.00	$44,888.74	3.6	4	16.8	$102
STA 364+20 to 375+00	0	$32,975.79	$0.00	$32,975.79	2.6	3	16.8	$102
STA 375+00 to 389+00	0	$42,746.40	$0.00	$42,746.40	3.4	1	16.8	$102
TOTALS:	0	$4,489,077.14	$0.00	$4,489,077.14	265.4	75		

TABLE C.2
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

Channel Reach	Total Drop Structure Cost For This Reach	Bank Lining Cost At Each Drop Structure ($/Drop)	Cost Of Bank Lining Cutoff Wall At Each Drop Structure ($/Drop)	Total Bank Lining Cost For This Reach	Total Drop Structure Cost With Bank Lining
Start @ 5+00					
STA 5+00 to 22+30	$0	$15,700	$740	$0	$0
STA 22+30 to 42+30	$59,678	$15,700	$740	$32,881	$92,559
STA 42+30 to 68+90	$0	$15,700	$740	$0	$0
STA 68+90 to 90+80	$59,678	$15,700	$740	$32,881	$92,559
STA 90+80 to 106+80	$89,185	$15,700	$740	$49,321	$138,506
STA 106+80 to 125+10	$89,185	$15,700	$740	$49,321	$138,506
STA 125+10 to 141+20	$89,185	$15,700	$740	$49,321	$138,506
STA 141+20 to 166+20	$144,137	$15,700	$740	$82,202	$226,339
STA 166+20 to 173+10	$28,827	$15,700	$740	$16,440	$45,268
STA 173+10 to 184+30	$28,828	$15,700	$740	$16,440	$45,268
STA 184+30 to 197+30	$115,310	$15,700	$740	$65,761	$181,072
STA 197+30 to 214+60	$74,104	$15,700	$740	$49,321	$123,426
STA 214+60 to 227+80	$98,805	$15,700	$740	$65,761	$164,567
STA 227+80 to 238+90	$98,805	$15,700	$740	$65,761	$164,566
STA 238+90 to 252+80	$74,104	$15,700	$740	$49,321	$123,425
STA 252+80 to 262+60	$123,506	$15,700	$740	$82,202	$205,707
STA 262+60 to 274+50	$98,805	$15,700	$740	$65,761	$164,567
STA 274+50 to 288+80	$74,104	$15,700	$740	$49,321	$123,425
STA 288+80 to 303+90	$64,985	$15,800	$740	$66,161	$131,147
STA 303+90 to 314+90	$64,988	$15,800	$740	$66,161	$131,149
STA 314+90 to 323+80	$32,494	$15,800	$740	$33,081	$65,575
STA 323+80 to 335+80	$64,988	$15,800	$740	$66,161	$131,149
STA 335+80 to 349+50	$48,741	$15,800	$740	$49,621	$98,362
STA 349+50 to 364+20	$18,241	$16,200	$740	$67,761	$86,002
STA 364+20 to 375+00	$13,678	$16,200	$740	$50,821	$64,499
STA 375+00 to 389+00	$4,559	$16,200	$740	$16,940	$21,500
TOTALS:	$1,658,922			$1,238,728	$2,897,649

TABLE C.3
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
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

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 22+30	1730	4633	0.0058	0.025	2.00	4.0	99.8	10.7	0.98	5.5	121.8
STA 22+30 to 42+30	2000	4633	0.0100	0.025	2.00	4.0	75.3	13.9	1.28	5.5	97.3
STA 42+30 to 68+90	2660	4633	0.0075	0.025	2.00	4.0	87.3	12.2	1.12	5.5	109.3
STA 68+90 to 90+80	2190	4633	0.0091	0.025	2.00	4.0	78.9	13.3	1.23	5.5	100.9
STA 90+80 to 106+80	1600	4616	0.0125	0.025	2.00	4.0	66.9	15.4	1.43	5.5	88.9
STA 106+80 to 125+10	1830	4616	0.0109	0.025	2.00	4.0	71.7	14.5	1.34	5.5	93.7
STA 125+10 to 141+20	1610	4616	0.0124	0.025	2.00	4.0	67.1	15.4	1.42	5.5	89.1
STA 141+20 to 166+20	2500	4478	0.0120	0.025	2.00	4.0	66.2	15.1	1.40	5.5	88.2
STA 166+20 to 173+10	690	4478	0.0145	0.025	2.00	4.0	60.0	16.5	1.53	5.5	82.0
STA 173+10 to 184+30	1120	4478	0.0089	0.025	2.00	4.0	77.1	13.2	1.21	5.5	99.1
STA 184+30 to 197+30	1300	4478	0.0154	0.025	2.00	4.0	58.1	16.9	1.58	5.5	80.1
STA 197+30 to 214+60	1730	3846	0.0116	0.025	2.00	4.0	57.6	14.7	1.37	5.5	79.6
STA 214+60 to 227+80	1320	3846	0.0152	0.025	2.00	4.0	49.9	16.6	1.56	5.5	71.9
STA 227+80 to 238+90	1110	3846	0.0180	0.025	2.00	4.0	45.5	18.0	1.70	5.5	67.5
STA 238+90 to 252+80	1390	3846	0.0144	0.025	2.00	4.0	51.3	16.2	1.52	5.5	73.3
STA 252+80 to 262+60	980	3846	0.0204	0.025	2.00	4.0	42.6	19.0	1.80	5.5	64.6
STA 262+60 to 274+50	1190	3846	0.0168	0.025	2.00	4.0	47.3	17.4	1.64	5.5	69.3
STA 274+50 to 288+80	1430	3846	0.0140	0.025	2.00	4.0	52.1	16.0	1.50	5.5	74.1
STA 288+80 to 303+90	1510	2527	0.0132	0.025	2.00	4.0	34.2	15.0	1.44	5.5	56.2
STA 303+90 to 314+90	1100	2527	0.0164	0.025	2.00	4.0	30.4	16.4	1.59	5.5	52.4
STA 314+90 to 323+80	890	2527	0.0135	0.025	2.00	4.0	33.8	15.1	1.45	5.5	55.8
STA 323+80 to 335+80	1200	2527	0.0167	0.025	2.00	4.0	30.1	16.6	1.61	5.5	52.1
STA 335+80 to 349+50	1370	2527	0.0146	0.025	2.00	4.0	32.4	15.6	1.51	5.5	54.4
STA 349+50 to 364+20	1470	752	0.0136	0.025	2.00	4.0	7.1	12.5	1.36	5.5	29.1
STA 364+20 to 375+00	1080	752	0.0167	0.025	2.00	4.0	5.9	13.5	1.49	5.5	27.9
STA 375+00 to 389+00	1400	752	0.0086	0.025	2.00	4.0	10.1	10.4	1.10	5.5	32.1
TOTALS:	38400										

TABLE C.3
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
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

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 22+30	5.0	50786	10764	$101,571.89	$258,346.67	$359,918.55	5.7
STA 22+30 to 42+30	5.0	48728	12444	$97,456.41	$298,666.67	$396,123.08	5.5
STA 42+30 to 68+90	5.0	71267	16551	$142,533.67	$397,226.67	$539,760.33	8.0
STA 68+90 to 90+80	5.0	54969	13627	$109,937.60	$327,040.00	$436,977.60	6.2
STA 90+80 to 106+80	5.0	36219	9956	$72,438.98	$238,933.33	$311,372.32	4.1
STA 106+80 to 125+10	5.0	43224	11387	$86,447.37	$273,280.00	$359,727.37	4.9
STA 125+10 to 141+20	5.0	36517	10018	$73,033.86	$240,426.67	$313,460.53	4.1
STA 141+20 to 166+20	5.0	56243	15556	$112,486.14	$373,333.33	$485,819.47	6.3
STA 166+20 to 173+10	5.0	14652	4293	$29,304.00	$103,040.00	$132,344.00	1.6
STA 173+10 to 184+30	5.0	27694	6969	$55,388.54	$167,253.33	$222,641.88	3.1
STA 184+30 to 197+30	5.0	27117	8089	$54,233.84	$194,133.33	$248,367.17	3.0
STA 197+30 to 214+60	5.0	35889	10764	$71,777.20	$258,346.67	$330,123.87	4.0
STA 214+60 to 227+80	5.0	25330	8213	$50,659.83	$197,120.00	$247,779.83	2.8
STA 227+80 to 238+90	5.0	20309	6907	$40,617.13	$165,760.00	$206,377.13	2.3
STA 238+90 to 252+80	5.0	27064	8649	$54,128.67	$207,573.33	$261,702.00	3.0
STA 252+80 to 262+60	5.0	17345	6098	$34,690.03	$146,346.67	$181,036.70	1.9
STA 262+60 to 274+50	5.0	22188	7404	$44,376.82	$177,706.67	$222,083.48	2.5
STA 274+50 to 288+80	5.0	28068	8898	$56,136.99	$213,546.67	$269,683.65	3.2
STA 288+80 to 303+90	5.0	24130	9396	$48,259.71	$225,493.33	$273,753.04	2.7
STA 303+90 to 314+90	5.0	16735	6844	$33,470.14	$164,266.67	$197,736.81	1.9
STA 314+90 to 323+80	5.0	14162	5538	$28,324.06	$132,906.67	$161,230.72	1.6
STA 323+80 to 335+80	5.0	18181	7467	$36,361.94	$179,200.00	$215,561.94	2.0
STA 335+80 to 349+50	5.0	21396	8524	$42,792.37	$204,586.67	$247,379.03	2.4
STA 349+50 to 364+20	5.0	15373	9147	$30,746.06	$219,520.00	$250,266.06	1.7
STA 364+20 to 375+00	5.0	11040	6720	$22,079.69	$161,280.00	$183,359.69	1.2
STA 375+00 to 389+00	5.0	15513	8711	$31,026.10	$209,066.67	$240,092.76	1.7
TOTALS:		780140	238933	$1,560,279.02	$5,734,400.00	$7,294,679.02	87.7

TABLE C.4
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, No Low-Flow Culverts
City of Phoenix, Areas C & D
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

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 22+30	1730	4633	0.0058	0.035	2.00	4.0	140.6	7.8	0.70	5.5	162.6
STA 22+30 to 42+30	2000	4633	0.0100	0.035	2.00	4.0	106.4	10.1	0.92	5.5	128.4
STA 42+30 to 68+90	2660	4633	0.0075	0.035	2.00	4.0	123.1	8.8	0.80	5.5	145.1
STA 68+90 to 90+80	2190	4633	0.0091	0.035	2.00	4.0	111.5	9.7	0.88	5.5	133.5
STA 90+80 to 106+80	1600	4616	0.0125	0.035	2.00	4.0	94.6	11.2	1.03	5.5	116.6
STA 106+80 to 125+10	1830	4616	0.0109	0.035	2.00	4.0	101.3	10.6	0.96	5.5	123.3
STA 125+10 to 141+20	1610	4616	0.0124	0.035	2.00	4.0	94.9	11.2	1.03	5.5	116.9
STA 141+20 to 166+20	2500	4478	0.0120	0.035	2.00	4.0	93.6	11.0	1.01	5.5	115.6
STA 166+20 to 173+10	690	4478	0.0145	0.035	2.00	4.0	85.0	12.0	1.11	5.5	107.0
STA 173+10 to 184+30	1120	4478	0.0089	0.035	2.00	4.0	108.9	9.6	0.87	5.5	130.9
STA 184+30 to 197+30	1300	4478	0.0154	0.035	2.00	4.0	82.4	12.4	1.14	5.5	104.4
STA 197+30 to 214+60	1730	3846	0.0116	0.035	2.00	4.0	81.6	10.7	0.99	5.5	103.6
STA 214+60 to 227+80	1320	3846	0.0152	0.035	2.00	4.0	71.0	12.2	1.13	5.5	93.0
STA 227+80 to 238+90	1110	3846	0.0180	0.035	2.00	4.0	64.9	13.2	1.22	5.5	86.9
STA 238+90 to 252+80	1390	3846	0.0144	0.035	2.00	4.0	72.9	11.9	1.10	5.5	94.9
STA 252+80 to 262+60	980	3846	0.0204	0.035	2.00	4.0	60.8	14.0	1.30	5.5	82.8
STA 262+60 to 274+50	1190	3846	0.0168	0.035	2.00	4.0	67.3	12.8	1.18	5.5	89.3
STA 274+50 to 288+80	1430	3846	0.0140	0.035	2.00	4.0	74.0	11.7	1.08	5.5	96.0
STA 288+80 to 303+90	1510	2527	0.0132	0.035	2.00	4.0	49.1	11.1	1.04	5.5	71.1
STA 303+90 to 314+90	1100	2527	0.0164	0.035	2.00	4.0	43.9	12.2	1.15	5.5	65.9
STA 314+90 to 323+80	890	2527	0.0135	0.035	2.00	4.0	48.6	11.2	1.05	5.5	70.6
STA 323+80 to 335+80	1200	2527	0.0167	0.035	2.00	4.0	43.4	12.3	1.16	5.5	65.4
STA 335+80 to 349+50	1370	2527	0.0146	0.035	2.00	4.0	46.6	11.6	1.09	5.5	68.6
STA 349+50 to 364+20	1470	752	0.0136	0.035	2.00	4.0	11.7	9.5	0.99	5.5	33.7
STA 364+20 to 375+00	1080	752	0.0167	0.035	2.00	4.0	10.2	10.3	1.09	5.5	32.2
STA 375+00 to 389+00	1400	752	0.0086	0.035	2.00	4.0	15.9	7.9	0.80	5.5	37.9
TOTALS:	38400										

TABLE C.4
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, No Low-Flow Culverts
City of Phoenix, Areas C & D
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

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 22+30	5.0	65166	10764	$130,332.34	$258,346.67	$388,679.00	7.3
STA 22+30 to 42+30	5.0	61391	12444	$122,781.70	$298,666.67	$421,448.36	6.9
STA 42+30 to 68+90	5.0	90669	16551	$181,337.27	$397,226.67	$578,563.94	10.2
STA 68+90 to 90+80	5.0	69473	13627	$138,945.58	$327,040.00	$465,985.58	7.8
STA 90+80 to 106+80	5.0	45256	9956	$90,512.80	$238,933.33	$329,446.14	5.1
STA 106+80 to 125+10	5.0	54271	11387	$108,541.15	$273,280.00	$381,821.15	6.1
STA 125+10 to 141+20	5.0	45638	10018	$91,276.82	$240,426.67	$331,703.49	5.1
STA 141+20 to 166+20	5.0	70225	15556	$140,449.93	$373,333.33	$513,783.26	7.9
STA 166+20 to 173+10	5.0	18167	4293	$36,334.21	$103,040.00	$139,374.21	2.0
STA 173+10 to 184+30	5.0	34946	6969	$69,892.71	$167,253.33	$237,146.05	3.9
STA 184+30 to 197+30	5.0	33547	8089	$67,094.13	$194,133.33	$261,227.46	3.8
STA 197+30 to 214+60	5.0	44368	10764	$88,735.41	$258,346.67	$347,082.08	5.0
STA 214+60 to 227+80	5.0	30992	8213	$61,983.45	$197,120.00	$259,103.45	3.5
STA 227+80 to 238+90	5.0	24681	6907	$49,361.38	$165,760.00	$215,121.38	2.8
STA 238+90 to 252+80	5.0	33180	8649	$66,360.13	$207,573.33	$273,933.46	3.7
STA 252+80 to 262+60	5.0	20976	6098	$41,952.24	$146,346.67	$188,298.91	2.4
STA 262+60 to 274+50	5.0	27039	7404	$54,077.51	$177,706.67	$231,784.18	3.0
STA 274+50 to 288+80	5.0	34449	8898	$68,897.57	$213,546.67	$282,444.23	3.9
STA 288+80 to 303+90	5.0	28713	9396	$57,426.90	$225,493.33	$282,920.23	3.2
STA 303+90 to 314+90	5.0	19748	6844	$39,495.46	$164,266.67	$203,762.13	2.2
STA 314+90 to 323+80	5.0	16840	5538	$33,680.52	$132,906.67	$166,587.19	1.9
STA 323+80 to 335+80	5.0	21438	7467	$42,876.74	$179,200.00	$222,076.74	2.4
STA 335+80 to 349+50	5.0	25362	8524	$50,724.71	$204,586.67	$255,311.38	2.8
STA 349+50 to 364+20	5.0	16775	9147	$33,549.95	$219,520.00	$253,069.95	1.9
STA 364+20 to 375+00	5.0	11980	6720	$23,960.15	$161,280.00	$185,240.15	1.3
STA 375+00 to 389+00	5.0	17162	8711	$34,324.86	$209,066.67	$243,391.52	1.9
TOTALS:		962453	238933	$1,924,905.62	$5,734,400.00	$7,659,305.62	108.2

TABLE C.5
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, Pinnacle Peak Road Alignment
No Low-Flow Culverts
City of Phoenix, Areas C & D
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

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 22+30	1730	5403	0.0058	0.035	2.00	4.0	164.4	7.8	0.71	5.5	186.4
STA 22+30 to 42+30	2000	5403	0.0100	0.035	2.00	4.0	124.5	10.2	0.93	5.5	146.5
STA 42+30 to 69+90	2760	5403	0.0072	0.035	2.00	4.0	146.6	8.7	0.79	5.5	168.6
STA 69+90 to 95+90	2600	5403	0.0077	0.035	2.00	4.0	142.2	9.0	0.81	5.5	164.2
STA 95+90 to 118+10	2220	5403	0.0090	0.035	2.00	4.0	131.2	9.7	0.88	5.5	153.2
STA 118+10 to 136+10	1800	4465	0.0056	0.035	2.00	4.0	138.2	7.6	0.69	5.5	160.2
STA 136+10 to 147+60	1150	4465	0.0087	0.035	2.00	4.0	110.1	9.5	0.86	5.5	132.1
STA 147+60 to 168+80	2120	4465	0.0094	0.035	2.00	4.0	105.6	9.8	0.90	5.5	127.6
STA 168+80 to 192+60	2380	4465	0.0084	0.035	2.00	4.0	112.0	9.3	0.85	5.5	134.0
STA 192+60 to 208+70	1610	2972	0.0062	0.035	2.00	4.0	86.2	7.9	0.72	5.5	108.2
STA 208+70 to 220+50	1180	2972	0.0085	0.035	2.00	4.0	73.4	9.1	0.84	5.5	95.4
STA 220+50 to 230+00	950	2972	0.0105	0.035	2.00	4.0	65.6	10.1	0.94	5.5	87.6
STA 230+00 to 240+30	1030	2972	0.0097	0.035	2.00	4.0	68.4	9.7	0.90	5.5	90.4
STA 240+30 to 253+30	1300	2972	0.0077	0.035	2.00	4.0	77.2	8.7	0.80	5.5	99.2
STA 253+30 to 264+80	1150	1103	0.0087	0.035	2.00	4.0	24.9	8.4	0.82	5.5	46.9
STA 264+80 to 274+00	920	1103	0.0109	0.035	2.00	4.0	21.9	9.2	0.91	5.5	43.9
STA 274+00 to 286+00	1200	1103	0.0083	0.035	2.00	4.0	25.5	8.2	0.81	5.5	47.5
STA 286+00 to 294+80	880	1103	0.0114	0.035	2.00	4.0	21.3	9.4	0.93	5.5	43.3
STA 294+80 to 304+20	940	1103	0.0106	0.035	2.00	4.0	22.2	9.1	0.91	5.5	44.2
TOTALS:	29920										

TABLE C.5
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, Pinnacle Peak Road Alignment
No Low-Flow Culverts
City of Phoenix, Areas C & D
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

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 22+30	5.0	73522	10764	$147,043.94	$258,346.67	$405,390.61	8.3
STA 22+30 to 42+30	5.0	68744	12444	$137,488.69	$298,666.67	$436,155.36	7.7
STA 42+30 to 69+90	5.0	107300	17173	$214,599.50	$412,160.00	$626,759.50	12.1
STA 69+90 to 95+90	5.0	98764	16178	$197,527.48	$388,266.67	$585,794.14	11.1
STA 95+90 to 118+10	5.0	79373	13813	$158,745.08	$331,520.00	$490,265.08	8.9
STA 118+10 to 136+10	5.0	66916	11200	$133,831.18	$268,800.00	$402,631.18	7.5
STA 136+10 to 147+60	5.0	36152	7156	$72,304.33	$171,733.33	$244,037.67	4.1
STA 147+60 to 168+80	5.0	64710	13191	$129,419.36	$316,586.67	$446,006.03	7.3
STA 168+80 to 192+60	5.0	75758	14809	$151,515.45	$355,413.33	$506,928.78	8.5
STA 192+60 to 208+70	5.0	42785	10018	$85,570.83	$240,426.67	$325,997.49	4.8
STA 208+70 to 220+50	5.0	28292	7342	$56,584.86	$176,213.33	$232,798.19	3.2
STA 220+50 to 230+00	5.0	21267	5911	$42,533.82	$141,866.67	$184,400.49	2.4
STA 230+00 to 240+30	5.0	23648	6409	$47,296.58	$153,813.33	$201,109.91	2.7
STA 240+30 to 253+30	5.0	32167	8089	$64,334.51	$194,133.33	$258,467.84	3.6
STA 253+30 to 264+80	5.0	16211	7156	$32,422.00	$171,733.33	$204,155.34	1.8
STA 264+80 to 274+00	5.0	12403	5724	$24,805.59	$137,386.67	$162,192.26	1.4
STA 274+00 to 286+00	5.0	17066	7467	$34,131.56	$179,200.00	$213,331.56	1.9
STA 286+00 to 294+80	5.0	11762	5476	$23,524.78	$131,413.33	$154,938.11	1.3
STA 294+80 to 304+20	5.0	12726	5849	$25,451.04	$140,373.33	$165,824.38	1.4
TOTALS:		889565	186169	$1,779,130.58	$4,468,053.33	$6,247,183.91	100.1

TABLE C.6
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Concrete (banks & bottom)
Horizontal Lining Width (ft): 1.12
Lining Cost/c.y.: $180.00
All Calculations Are Based On Existing Ground Slopes

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 16+20	1120	13069	0.0089	0.020	2.00	4.0	183.0	17.1	1.54	5.5	205.0
STA 16+20 to 25+80	960	13069	0.0104	0.020	2.00	4.0	169.3	18.4	1.66	5.5	191.3
STA 25+80 to 34+80	900	13069	0.0111	0.020	2.00	4.0	163.8	19.0	1.71	5.5	185.8
STA 34+80 to 50+40	1560	13069	0.0128	0.020	2.00	4.0	152.4	20.4	1.84	5.5	174.4
STA 50+40 to 66+10	1570	13069	0.0127	0.020	2.00	4.0	152.9	20.3	1.83	5.5	174.9
STA 66+10 to 80+10	1400	13069	0.0143	0.020	2.00	4.0	144.3	21.5	1.94	5.5	166.3
STA 80+10 to 95+10	1500	13069	0.0133	0.020	2.00	4.0	149.4	20.8	1.87	5.5	171.4
STA 95+10 to 107+50	1240	13069	0.0161	0.020	2.00	4.0	135.6	22.7	2.06	5.5	157.6
STA 107+50 to 123+00	1550	13069	0.0129	0.020	2.00	4.0	151.9	20.4	1.84	5.5	173.9
STA 123+00 to 128+00	500	13069	0.0200	0.020	2.00	4.0	121.6	25.2	2.29	5.5	143.6
STA 128+00 to 140+80	1280	13069	0.0156	0.020	2.00	4.0	137.8	22.4	2.03	5.5	159.8
STA 140+80 to 153+20	1240	13069	0.0161	0.020	2.00	4.0	135.6	22.7	2.06	5.5	157.6
STA 153+20 to 171+50	1830	13069	0.0164	0.020	2.00	4.0	134.5	22.9	2.08	5.5	156.5
STA 171+50 to 177+30	580	10986	0.0172	0.020	2.00	4.0	109.9	23.3	2.12	5.5	131.9
STA 177+30 to 189+70	1240	10986	0.0161	0.020	2.00	4.0	113.7	22.6	2.05	5.5	135.7
STA 189+70 to 201+00	1130	10986	0.0177	0.020	2.00	4.0	108.4	23.6	2.15	5.5	130.4
STA 201+00 to 210+70	970	10986	0.0206	0.020	2.00	4.0	100.3	25.4	2.32	5.5	122.3
STA 210+70 to 221+30	1060	10986	0.0189	0.020	2.00	4.0	104.9	24.3	2.22	5.5	126.9
STA 221+30 to 231+30	1000	10986	0.0200	0.020	2.00	4.0	101.9	25.0	2.28	5.5	123.9
STA 231+30 to 244+40	1310	10986	0.0153	0.020	2.00	4.0	116.9	22.0	2.00	5.5	138.9
STA 244+40 to 250+60	620	10986	0.0161	0.020	2.00	4.0	113.7	22.6	2.05	5.5	135.7
STA 250+60 to 264+50	1390	1106	0.0144	0.020	2.00	4.0	8.8	16.5	1.77	5.5	30.8
STA 264+50 to 274+00	950	1106	0.0211	0.020	2.00	4.0	6.4	19.2	2.11	5.5	28.4
STA 274+00 to 286+90	1290	1106	0.0155	0.020	2.00	4.0	8.3	17.0	1.83	5.5	30.3
STA 286+90 to 296+60	970	1106	0.0206	0.020	2.00	4.0	6.5	19.0	2.09	5.5	28.5
STA 296+60 to 310+50	1390	1106	0.0144	0.020	2.00	4.0	8.8	16.5	1.77	5.5	30.8
STA 310+50 to 321+70	1120	1106	0.0179	0.020	2.00	4.0	7.4	18.0	1.95	5.5	29.4
TOTALS:	31670										

TABLE C.6
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Concrete (banks & bottom)
Horizontal Lining Width (ft): 1.12
Lining Cost/c.y.: $180.00
All Calculations Are Based On Existing Ground Slopes

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 16+20	0.0	48594	4332	$97,188.29	$779,834.88	$877,023.17	5.7
STA 16+20 to 25+80	0.0	17692	3470	$1,593.71	$624,525.34	$626,119.05	4.5
STA 25+80 to 34+80	0.0	35217	3162	$70,434.46	$569,178.46	$639,612.92	4.2
STA 34+80 to 50+40	0.0	57073	5150	$114,146.29	$927,025.18	$1,041,171.47	6.8
STA 50+40 to 66+10	0.0	57611	5198	$115,222.57	$935,551.93	$1,050,774.50	6.8
STA 66+10 to 80+10	0.0	48689	4411	$97,377.93	$793,987.81	$891,365.74	5.8
STA 80+10 to 95+10	0.0	53878	4869	$107,756.69	$876,375.19	$984,131.88	6.4
STA 95+10 to 107+50	0.0	40750	3709	$81,500.99	$667,636.12	$749,137.12	4.9
STA 107+50 to 123+00	0.0	56536	5103	$113,072.63	$918,518.06	$1,031,590.69	6.7
STA 123+00 to 128+00	0.0	14871	1366	$29,741.53	$245,794.79	$275,536.31	1.8
STA 128+00 to 140+80	0.0	42692	3881	$85,383.53	$698,573.79	$783,957.32	5.1
STA 140+80 to 153+20	0.0	40750	3709	$81,500.77	$667,634.44	$749,135.21	4.9
STA 153+20 to 171+50	0.0	59686	5436	$119,372.90	$978,500.76	$1,097,873.66	7.2
STA 171+50 to 177+30	0.0	15740	1458	$31,480.75	$262,476.26	$293,957.00	2.0
STA 177+30 to 189+70	0.0	34699	3205	$69,397.82	$576,862.31	$646,260.13	4.3
STA 189+70 to 201+00	0.0	30300	2810	$60,600.37	$505,880.14	$566,480.51	3.8
STA 201+00 to 210+70	0.0	24256	2266	$48,512.59	$407,947.12	$456,459.72	3.1
STA 210+70 to 221+30	0.0	27603	2568	$55,205.54	$462,236.20	$517,441.74	3.5
STA 221+30 to 231+30	0.0	25356	2366	$50,712.88	$425,813.29	$476,526.18	3.2
STA 231+30 to 244+40	0.0	37597	3464	$75,193.93	$623,515.81	$698,709.74	4.6
STA 244+40 to 250+60	0.0	17350	1602	$34,699.13	$288,432.83	$323,131.96	2.1
STA 250+60 to 264+50	0.0	6490	892	$12,979.42	$160,544.34	$173,523.77	1.5
STA 264+50 to 274+00	0.0	3939	568	$7,877.73	$102,276.28	$110,154.01	0.9
STA 274+00 to 286+90	0.0	5881	816	$11,762.59	$146,871.40	$158,633.99	1.3
STA 286+90 to 296+60	0.0	4047	582	$8,094.66	$104,812.59	$112,907.25	1.0
STA 296+60 to 310+50	0.0	6490	892	$12,979.36	$160,543.87	$173,523.23	1.5
STA 310+50 to 321+70	0.0	4885	690	$9,769.46	$124,193.59	$133,963.05	1.1
TOTALS:		818675	77975	$1,603,558.53	$14,035,542.79	$15,639,101.32	104.8

TABLE C.7
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

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	Average Channel Depth (ft/ft)	Channel Topwidth (ft)	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)
Start @ 5+00													
STA 5+00 to 16+20	1120	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	219344
STA 16+20 to 25+80	960	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	188009
STA 25+80 to 34+80	900	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	176259
STA 34+80 to 50+40	1560	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	305515
STA 50+40 to 66+10	1570	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	307473
STA 66+10 to 80+10	1400	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	274180
STA 80+10 to 95+10	1500	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	293764
STA 95+10 to 107+50	1240	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	242845
STA 107+50 to 123+00	1550	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	303556
STA 123+00 to 128+00	500	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	97921
STA 128+00 to 140+80	1280	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	250679
STA 140+80 to 153+20	1240	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	242845
STA 153+20 to 171+50	1830	13069	0.0067	0.050	4.00	3.0	857.3	5.0	0.51	6.0	905.3	0.0	358392
STA 171+50 to 177+30	580	10986	0.0067	0.050	4.00	3.0	720.1	5.0	0.51	6.0	768.1	0.0	95901
STA 177+30 to 189+70	1240	10986	0.0067	0.050	4.00	3.0	720.1	5.0	0.51	6.0	768.1	0.0	205030
STA 189+70 to 201+00	1130	10986	0.0067	0.050	4.00	3.0	720.1	5.0	0.51	6.0	768.1	0.0	186841
STA 201+00 to 210+70	970	10986	0.0067	0.050	4.00	3.0	720.1	5.0	0.51	6.0	768.1	0.0	160386
STA 210+70 to 221+30	1060	10986	0.0067	0.050	4.00	3.0	720.1	5.0	0.51	6.0	768.1	0.0	175267
STA 221+30 to 231+30	1000	10986	0.0067	0.050	4.00	3.0	720.1	5.0	0.51	6.0	768.1	0.0	165346
STA 231+30 to 244+40	1310	10986	0.0067	0.050	4.00	3.0	720.1	5.0	0.51	6.0	768.1	0.0	216604
STA 244+40 to 250+60	620	10986	0.0067	0.050	4.00	3.0	720.1	5.0	0.51	6.0	768.1	0.0	102515
STA 250+60 to 264+50	1390	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0	28531
STA 264+50 to 274+00	950	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0	19499
STA 274+00 to 286+90	1290	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0	26478
STA 286+90 to 296+60	970	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0	19910
STA 296+60 to 310+50	1390	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0	28531
STA 310+50 to 321+70	1120	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0	22989
TOTALS:	31670												4714609

TABLE C.7
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

Channel Reach	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 12 ft Maintenance Road (acres)	Number Of Drop Structures In This Reach	Unit Discharge For This Reach (cfs/ft)	Drop Structure Cost Per LF Of Channel Bottomwidth
Start @ 5+00								
STA 5+00 to 16+20	0	$438,687.91	$0.00	$438,687.91	23.6	0	15.2	$98
STA 16+20 to 25+80	0	$376,018.21	$0.00	$376,018.21	20.3	1	15.2	$98
STA 25+80 to 34+80	0	$352,517.07	$0.00	$352,517.07	19.0	1	15.2	$98
STA 34+80 to 50+40	0	$611,029.59	$0.00	$611,029.59	32.9	3	15.2	$98
STA 50+40 to 66+10	0	$614,946.44	$0.00	$614,946.44	33.1	4	15.2	$98
STA 66+10 to 80+10	0	$548,359.88	$0.00	$548,359.88	29.5	3	15.2	$98
STA 80+10 to 95+10	0	$587,528.45	$0.00	$587,528.45	31.7	4	15.2	$98
STA 95+10 to 107+50	0	$485,690.18	$0.00	$485,690.18	26.2	4	15.2	$98
STA 107+50 to 123+00	0	$607,112.73	$0.00	$607,112.73	32.7	3	15.2	$98
STA 123+00 to 128+00	0	$195,842.82	$0.00	$195,842.82	10.6	2	15.2	$98
STA 128+00 to 140+80	0	$501,357.61	$0.00	$501,357.61	27.0	4	15.2	$98
STA 140+80 to 153+20	0	$485,690.18	$0.00	$485,690.18	26.2	5	15.2	$98
STA 153+20 to 171+50	0	$716,784.71	$0.00	$716,784.71	38.6	6	15.2	$98
STA 171+50 to 177+30	0	$191,801.79	$0.00	$191,801.79	10.4	2	15.3	$99
STA 177+30 to 189+70	0	$410,059.00	$0.00	$410,059.00	22.3	4	15.3	$99
STA 189+70 to 201+00	0	$373,682.80	$0.00	$373,682.80	20.3	5	15.3	$99
STA 201+00 to 210+70	0	$320,771.96	$0.00	$320,771.96	17.4	4	15.3	$99
STA 210+70 to 221+30	0	$350,534.31	$0.00	$350,534.31	19.0	5	15.3	$99
STA 221+30 to 231+30	0	$330,692.74	$0.00	$330,692.74	18.0	4	15.3	$99
STA 231+30 to 244+40	0	$433,207.49	$0.00	$433,207.49	23.5	4	15.3	$99
STA 244+40 to 250+60	0	$205,029.50	$0.00	$205,029.50	11.1	2	15.3	$99
STA 250+60 to 264+50	0	$57,061.01	$0.00	$57,061.01	4.2	4	16.2	$101
STA 264+50 to 274+00	0	$38,998.53	$0.00	$38,998.53	2.8	4	16.2	$101
STA 274+00 to 286+90	0	$52,955.90	$0.00	$52,955.90	3.9	4	16.2	$101
STA 286+90 to 296+60	0	$39,819.55	$0.00	$39,819.55	2.9	5	16.2	$101
STA 296+60 to 310+50	0	$57,061.01	$0.00	$57,061.01	4.2	3	16.2	$101
STA 310+50 to 321+70	0	$45,977.21	$0.00	$45,977.21	3.4	4	16.2	$101
TOTALS:	0	$9,429,218.56	$0.00	$9,429,218.56	514.7	94		

TABLE C.7
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

Channel Reach	Total Drop Structure Cost For This Reach	Bank Lining Cost At Each Drop Structure ($/Drop)	Cost Of Bank Lining Cutoff Wall At Each Drop Structure ($/Drop)	Total Bank Lining Cost For This Reach	Total Drop Structure Cost With Bank Lining
Start @ 5+00					
STA 5+00 to 16+20	$0	$15,500	$740	$0	$0
STA 16+20 to 25+80	$84,015	$15,500	$740	$16,240	$100,255
STA 25+80 to 34+80	$84,015	$15,500	$740	$16,240	$100,255
STA 34+80 to 50+40	$252,044	$15,500	$740	$48,721	$300,765
STA 50+40 to 66+10	$336,059	$15,500	$740	$64,961	$401,020
STA 66+10 to 80+10	$252,044	$15,500	$740	$48,721	$300,765
STA 80+10 to 95+10	$336,059	$15,500	$740	$64,961	$401,020
STA 95+10 to 107+50	$336,059	$15,500	$740	$64,961	$401,020
STA 107+50 to 123+00	$252,044	$15,500	$740	$48,721	$300,765
STA 123+00 to 128+00	$168,029	$15,500	$740	$32,481	$200,510
STA 128+00 to 140+80	$336,059	$15,500	$740	$64,961	$401,020
STA 140+80 to 153+20	$420,073	$15,500	$740	$81,202	$501,275
STA 153+20 to 171+50	$504,088	$15,500	$740	$97,442	$601,530
STA 171+50 to 177+30	$142,572	$15,600	$740	$32,681	$175,252
STA 177+30 to 189+70	$285,143	$15,600	$740	$65,361	$350,505
STA 189+70 to 201+00	$356,429	$15,600	$740	$81,702	$438,131
STA 201+00 to 210+70	$285,143	$15,600	$740	$65,361	$350,505
STA 210+70 to 221+30	$356,429	$15,600	$740	$81,702	$438,131
STA 221+30 to 231+30	$285,143	$15,600	$740	$65,361	$350,505
STA 231+30 to 244+40	$285,143	$15,600	$740	$65,361	$350,505
STA 244+40 to 250+60	$142,572	$15,600	$740	$32,681	$175,252
STA 250+60 to 264+50	$27,619	$16,000	$740	$66,961	$94,581
STA 264+50 to 274+00	$27,619	$16,000	$740	$66,961	$94,581
STA 274+00 to 286+90	$27,619	$16,000	$740	$66,961	$94,581
STA 286+90 to 296+60	$34,524	$16,000	$740	$83,702	$118,226
STA 296+60 to 310+50	$20,715	$16,000	$740	$50,221	$70,936
STA 310+50 to 321+70	$27,619	$16,000	$740	$66,961	$94,581
TOTALS:	$5,664,878			$1,541,595	$7,206,473

TABLE C.8
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
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

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 16+20	1120	13069	0.0089	0.025	2.00	4.0	229.2	13.8	1.23	5.5	251.2
STA 16+20 to 25+80	960	13069	0.0104	0.025	2.00	4.0	212.1	14.8	1.33	5.5	234.1
STA 25+80 to 34+80	900	13069	0.0111	0.025	2.00	4.0	205.3	15.3	1.37	5.5	227.3
STA 34+80 to 50+40	1560	13069	0.0128	0.025	2.00	4.0	191.0	16.4	1.48	5.5	213.0
STA 50+40 to 66+10	1570	13069	0.0127	0.025	2.00	4.0	191.6	16.4	1.47	5.5	213.6
STA 66+10 to 80+10	1400	13069	0.0143	0.025	2.00	4.0	180.8	17.3	1.56	5.5	202.8
STA 80+10 to 95+10	1500	13069	0.0133	0.025	2.00	4.0	187.2	16.7	1.50	5.5	209.2
STA 95+10 to 107+50	1240	13069	0.0161	0.025	2.00	4.0	170.1	18.3	1.65	5.5	192.1
STA 107+50 to 123+00	1550	13069	0.0129	0.025	2.00	4.0	190.4	16.5	1.48	5.5	212.4
STA 123+00 to 128+00	500	13069	0.0200	0.025	2.00	4.0	152.5	20.4	1.84	5.5	174.5
STA 128+00 to 140+80	1280	13069	0.0156	0.025	2.00	4.0	172.8	18.1	1.63	5.5	194.8
STA 140+80 to 153+20	1240	13069	0.0161	0.025	2.00	4.0	170.1	18.3	1.65	5.5	192.1
STA 153+20 to 171+50	1830	13069	0.0164	0.025	2.00	4.0	168.7	18.5	1.67	5.5	190.7
STA 171+50 to 177+30	580	10986	0.0172	0.025	2.00	4.0	137.9	18.8	1.70	5.5	159.9
STA 177+30 to 189+70	1240	10986	0.0161	0.025	2.00	4.0	142.6	18.2	1.65	5.5	164.6
STA 189+70 to 201+00	1130	10986	0.0177	0.025	2.00	4.0	136.1	19.1	1.73	5.5	158.1
STA 201+00 to 210+70	970	10986	0.0206	0.025	2.00	4.0	125.9	20.5	1.86	5.5	147.9
STA 210+70 to 221+30	1060	10986	0.0189	0.025	2.00	4.0	131.7	19.7	1.78	5.5	153.7
STA 221+30 to 231+30	1000	10986	0.0200	0.025	2.00	4.0	127.9	20.2	1.83	5.5	149.9
STA 231+30 to 244+40	1310	10986	0.0153	0.025	2.00	4.0	146.7	17.8	1.60	5.5	168.7
STA 244+40 to 250+60	620	10986	0.0161	0.025	2.00	4.0	142.6	18.2	1.65	5.5	164.6
STA 250+60 to 264+50	1390	1106	0.0144	0.025	2.00	4.0	12.1	13.8	1.43	5.5	34.1
STA 264+50 to 274+00	950	1106	0.0211	0.025	2.00	4.0	9.2	16.1	1.71	5.5	31.2
STA 274+00 to 286+90	1290	1106	0.0155	0.025	2.00	4.0	11.5	14.2	1.48	5.5	33.5
STA 286+90 to 296+60	970	1106	0.0206	0.025	2.00	4.0	9.4	15.9	1.70	5.5	31.4
STA 296+60 to 310+50	1390	1106	0.0144	0.025	2.00	4.0	12.1	13.8	1.43	5.5	34.1
STA 310+50 to 321+70	1120	1106	0.0179	0.025	2.00	4.0	10.4	15.0	1.59	5.5	32.4
TOTALS:	31670										

TABLE C.8
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D

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

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 16+20	5.0	62398	6969	$124,796.67	$167,253.33	$292,050.00	7.0
STA 16+20 to 25+80	5.0	50132	5973	$100,264.34	$143,360.00	$243,624.34	5.6
STA 25+80 to 34+80	5.0	45753	5600	$91,506.71	$134,400.00	$225,906.71	5.2
STA 34+80 to 50+40	5.0	74760	9707	$149,519.08	$232,960.00	$382,479.08	8.4
STA 50+40 to 66+10	5.0	75436	9769	$150,872.24	$234,453.33	$385,325.57	8.5
STA 66+10 to 80+10	5.0	64194	8711	$128,388.60	$209,066.67	$337,455.27	7.2
STA 80+10 to 95+10	5.0	70740	9333	$141,479.02	$224,000.00	$365,479.02	8.0
STA 95+10 to 107+50	5.0	54140	7716	$108,279.23	$185,173.33	$293,452.57	6.1
STA 107+50 to 123+00	5.0	74085	9644	$148,169.33	$231,466.67	$379,635.99	8.3
STA 123+00 to 128+00	5.0	20043	3111	$40,086.98	$74,666.67	$114,753.65	2.3
STA 128+00 to 140+80	5.0	56604	7964	$113,207.41	$191,146.67	$304,354.07	6.4
STA 140+80 to 153+20	5.0	54140	7716	$108,279.11	$185,173.33	$293,452.44	6.1
STA 153+20 to 171+50	5.0	79380	11387	$158,760.15	$273,280.00	$432,040.15	8.9
STA 171+50 to 177+30	5.0	21522	3609	$43,044.49	$86,613.33	$129,657.83	2.4
STA 177+30 to 189+70	5.0	47212	7716	$94,423.78	$185,173.33	$279,597.11	5.3
STA 189+70 to 201+00	5.0	41512	7031	$83,023.71	$168,746.67	$251,770.38	4.7
STA 201+00 to 210+70	5.0	33628	6036	$67,255.01	$144,853.33	$212,108.34	3.8
STA 210+70 to 221+30	5.0	38002	6596	$76,003.36	$158,293.33	$234,296.69	4.3
STA 221+30 to 231+30	5.0	35068	6222	$70,135.78	$149,333.33	$219,469.12	3.9
STA 231+30 to 244+40	5.0	50952	8151	$101,904.09	$195,626.67	$297,530.76	5.7
STA 244+40 to 250+60	5.0	23606	3858	$47,212.01	$92,586.67	$139,798.68	2.7
STA 250+60 to 264+50	5.0	15961	8649	$31,921.07	$207,573.33	$239,494.41	1.8
STA 264+50 to 274+00	5.0	10351	5911	$20,702.93	$141,866.67	$162,569.60	1.2
STA 274+00 to 286+90	5.0	14653	8027	$29,306.84	$192,640.00	$221,946.84	1.6
STA 286+90 to 296+60	5.0	10598	6036	$21,196.04	$144,853.33	$166,049.37	1.2
STA 296+60 to 310+50	5.0	15961	8649	$31,921.01	$207,573.33	$239,494.34	1.8
STA 310+50 to 321+70	5.0	12474	6969	$24,947.69	$167,253.33	$192,201.03	1.4
TOTALS:		1153303	197058	$2,306,606.69	$4,729,386.67	$7,035,993.35	129.8

TABLE C.9
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
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

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 16+20	1120	13069	0.0089	0.035	2.00	4.0	321.6	9.9	0.88	5.5	343.6
STA 16+20 to 25+80	960	13069	0.0104	0.035	2.00	4.0	297.6	10.7	0.95	5.5	319.6
STA 25+80 to 34+80	900	13069	0.0111	0.035	2.00	4.0	288.1	11.0	0.99	5.5	310.1
STA 34+80 to 50+40	1560	13069	0.0128	0.035	2.00	4.0	268.1	11.8	1.06	5.5	290.1
STA 50+40 to 66+10	1570	13069	0.0127	0.035	2.00	4.0	269.0	11.8	1.05	5.5	291.0
STA 66+10 to 80+10	1400	13069	0.0143	0.035	2.00	4.0	253.9	12.5	1.12	5.5	275.9
STA 80+10 to 95+10	1500	13069	0.0133	0.035	2.00	4.0	262.9	12.1	1.08	5.5	284.9
STA 95+10 to 107+50	1240	13069	0.0161	0.035	2.00	4.0	238.8	13.2	1.19	5.5	260.8
STA 107+50 to 123+00	1550	13069	0.0129	0.035	2.00	4.0	267.3	11.9	1.06	5.5	289.3
STA 123+00 to 128+00	500	13069	0.0200	0.035	2.00	4.0	214.3	14.7	1.32	5.5	236.3
STA 128+00 to 140+80	1280	13069	0.0156	0.035	2.00	4.0	242.7	13.0	1.17	5.5	264.7
STA 140+80 to 153+20	1240	13069	0.0161	0.035	2.00	4.0	238.8	13.2	1.19	5.5	260.8
STA 153+20 to 171+50	1830	13069	0.0164	0.035	2.00	4.0	236.9	13.3	1.19	5.5	258.9
STA 171+50 to 177+30	580	10986	0.0172	0.035	2.00	4.0	193.8	13.6	1.22	5.5	215.8
STA 177+30 to 189+70	1240	10986	0.0161	0.035	2.00	4.0	200.5	13.2	1.18	5.5	222.5
STA 189+70 to 201+00	1130	10986	0.0177	0.035	2.00	4.0	191.3	13.8	1.24	5.5	213.3
STA 201+00 to 210+70	970	10986	0.0206	0.035	2.00	4.0	177.1	14.8	1.34	5.5	199.1
STA 210+70 to 221+30	1060	10986	0.0189	0.035	2.00	4.0	185.2	14.2	1.28	5.5	207.2
STA 221+30 to 231+30	1000	10986	0.0200	0.035	2.00	4.0	179.8	14.6	1.32	5.5	201.8
STA 231+30 to 244+40	1310	10986	0.0153	0.035	2.00	4.0	206.1	12.8	1.15	5.5	228.1
STA 244+40 to 250+60	620	10986	0.0161	0.035	2.00	4.0	200.5	13.2	1.18	5.5	222.5
STA 250+60 to 264+50	1390	1106	0.0144	0.035	2.00	4.0	18.6	10.4	1.04	5.5	40.6
STA 264+50 to 274+00	950	1106	0.0211	0.035	2.00	4.0	14.7	12.2	1.25	5.5	36.7
STA 274+00 to 286+90	1290	1106	0.0155	0.035	2.00	4.0	17.8	10.7	1.08	5.5	39.8
STA 286+90 to 296+60	970	1106	0.0206	0.035	2.00	4.0	14.9	12.1	1.24	5.5	36.9
STA 296+60 to 310+50	1390	1106	0.0144	0.035	2.00	4.0	18.6	10.4	1.04	5.5	40.6
STA 310+50 to 321+70	1120	1106	0.0179	0.035	2.00	4.0	16.3	11.4	1.16	5.5	38.3
TOTALS:	31670										

TABLE C.9
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With No Low-Flow Culverts
City of Phoenix, Areas C & D
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

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 16+20	5.0	83480	6969	$166,959.52	$167,253.33	$334,212.85	9.4
STA 16+20 to 25+80	5.0	66863	5973	$133,726.59	$143,360.00	$277,086.59	7.5
STA 25+80 to 34+80	5.0	60941	5600	$121,882.80	$134,400.00	$256,282.80	6.9
STA 34+80 to 50+40	5.0	99271	9707	$198,541.01	$232,960.00	$431,501.01	11.2
STA 50+40 to 66+10	5.0	100183	9769	$200,366.14	$234,453.33	$434,819.47	11.3
STA 66+10 to 80+10	5.0	85035	8711	$170,069.64	$209,066.67	$379,136.30	9.6
STA 80+10 to 95+10	5.0	93851	9333	$187,701.83	$224,000.00	$411,701.83	10.6
STA 95+10 to 107+50	5.0	71514	7716	$143,027.43	$185,173.33	$328,200.77	8.1
STA 107+50 to 123+00	5.0	98361	9644	$196,721.17	$231,466.67	$428,187.84	11.1
STA 123+00 to 128+00	5.0	26336	3111	$52,672.70	$74,666.67	$127,339.37	3.0
STA 128+00 to 140+80	5.0	74825	7964	$149,649.37	$191,146.67	$340,796.04	8.4
STA 140+80 to 153+20	5.0	71514	7716	$143,027.43	$185,173.33	$328,200.77	8.1
STA 153+20 to 171+50	5.0	104814	11387	$209,627.90	$273,280.00	$482,907.90	11.8
STA 171+50 to 177+30	5.0	28133	3609	$56,266.26	$86,613.33	$142,879.59	3.2
STA 177+30 to 189+70	5.0	61823	7716	$123,646.41	$185,173.33	$308,819.74	7.0
STA 189+70 to 201+00	5.0	54225	7031	$108,449.24	$168,746.67	$277,195.90	6.1
STA 201+00 to 210+70	5.0	43741	6036	$87,481.33	$144,853.33	$232,334.67	4.9
STA 210+70 to 221+30	5.0	49553	6596	$99,105.39	$158,293.33	$257,398.72	5.6
STA 221+30 to 231+30	5.0	45653	6222	$91,306.31	$149,333.33	$240,639.65	5.1
STA 231+30 to 244+40	5.0	66817	8151	$133,633.10	$195,626.67	$329,259.76	7.5
STA 244+40 to 250+60	5.0	30912	3858	$61,823.21	$92,586.67	$154,409.87	3.5
STA 250+60 to 264+50	5.0	17802	8649	$35,604.54	$207,573.33	$243,177.88	2.0
STA 264+50 to 274+00	5.0	11407	5911	$22,814.28	$141,866.67	$164,680.94	1.3
STA 274+00 to 286+90	5.0	16304	8027	$32,608.72	$192,640.00	$225,248.72	1.8
STA 286+90 to 296+60	5.0	11686	6036	$23,372.58	$144,853.33	$168,225.91	1.3
STA 296+60 to 310+50	5.0	17802	8649	$35,604.54	$207,573.33	$243,177.88	2.0
STA 310+50 to 321+70	5.0	13816	6969	$27,632.87	$167,253.33	$194,886.20	1.6
TOTALS:		1506661	197058	$3,013,322.31	$4,729,386.67	$7,742,708.97	169.6

TABLE C.10
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $3.00
Channel Lining: Concrete (banks & bottom)
Horizontal Lining Width (ft): 1.12
Lining Cost/c.y.: $180.00
All Calculations Are Based On Existing Ground Slopes

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 22+30	1730	2899	0.0058	0.020	2.00	4.0	48.7	12.8	1.20	5.5	70.7
STA 22+30 to 42+30	2000	2899	0.0100	0.020	2.00	4.0	36.3	16.4	1.57	5.5	58.3
STA 42+30 to 68+90	2660	2910	0.0075	0.020	2.00	4.0	42.5	14.4	1.37	5.5	64.5
STA 68+90 to 90+80	2190	2910	0.0091	0.020	2.00	4.0	38.3	15.7	1.50	5.5	60.3
STA 90+80 to 106+80	1600	2910	0.0125	0.020	2.00	4.0	32.2	18.1	1.74	5.5	54.2
STA 106+80 to 125+10	1830	2910	0.0109	0.020	2.00	4.0	34.7	17.0	1.64	5.5	56.7
STA 125+10 to 141+20	1610	2910	0.0124	0.020	2.00	4.0	32.3	18.0	1.74	5.5	54.3
STA 141+20 to 166+20	2500	2782	0.0120	0.020	2.00	4.0	31.4	17.7	1.71	5.5	53.4
STA 166+20 to 173+10	690	2782	0.0145	0.020	2.00	4.0	28.2	19.2	1.87	5.5	50.2
STA 173+10 to 184+30	1120	2782	0.0089	0.020	2.00	4.0	36.9	15.5	1.48	5.5	58.9
STA 184+30 to 197+30	1300	2782	0.0154	0.020	2.00	4.0	27.3	19.7	1.92	5.5	49.3
STA 197+30 to 214+60	1730	2546	0.0116	0.020	2.00	4.0	29.0	17.2	1.67	5.5	51.0
STA 214+60 to 227+80	1320	2546	0.0152	0.020	2.00	4.0	24.9	19.3	1.90	5.5	46.9
STA 227+80 to 238+90	1110	2546	0.0180	0.020	2.00	4.0	22.5	20.8	2.06	5.5	44.5
STA 238+90 to 252+80	1390	2546	0.0144	0.020	2.00	4.0	25.7	18.9	1.85	5.5	47.7
STA 252+80 to 262+60	980	2546	0.0204	0.020	2.00	4.0	20.9	22.0	2.19	5.5	42.9
STA 262+60 to 274+50	1190	2546	0.0168	0.020	2.00	4.0	23.5	20.2	2.00	5.5	45.5
STA 274+50 to 288+80	1430	2546	0.0140	0.020	2.00	4.0	26.1	18.7	1.83	5.5	48.1
STA 288+80 to 303+90	1510	1856	0.0132	0.020	2.00	4.0	18.6	17.5	1.75	5.5	40.6
STA 303+90 to 314+90	1100	1856	0.0164	0.020	2.00	4.0	16.3	19.1	1.94	5.5	38.3
STA 314+90 to 323+80	890	1856	0.0135	0.020	2.00	4.0	18.4	17.6	1.77	5.5	40.4
STA 323+80 to 335+80	1200	1856	0.0167	0.020	2.00	4.0	16.1	19.2	1.95	5.5	38.1
STA 335+80 to 349+50	1370	1856	0.0146	0.020	2.00	4.0	17.5	18.2	1.84	5.5	39.5
STA 349+50 to 364+20	1470	502	0.0136	0.020	2.00	4.0	1.3	13.5	1.62	5.5	23.3
STA 364+20 to 375+00	1080	502	0.0167	0.020	2.00	4.0	0.6	14.5	1.78	5.5	22.6
STA 375+00 to 389+00	1400	502	0.0086	0.020	2.00	4.0	3.1	11.3	1.31	5.5	25.1
TOTALS:	38400										

TABLE C.10
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $3.00
Channel Lining: Concrete (banks & bottom)
Horizontal Lining Width (ft): 1.12
Lining Cost/c.y.: $180.00
All Calculations Are Based On Existing Ground Slopes

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 22+30	0.0	23420	2389	$70,259.21	$429,953.40	$500,212.61	3.4
STA 22+30 to 42+30	0.0	21564	2302	$64,690.69	$414,386.77	$479,077.46	3.4
STA 42+30 to 68+90	0.0	32351	3368	$97,052.16	$606,202.14	$703,254.30	4.9
STA 68+90 to 90+80	0.0	24580	2602	$73,740.51	$468,274.57	$542,015.08	3.8
STA 90+80 to 106+80	0.0	15814	1722	$47,443.23	$309,962.83	$357,406.07	2.5
STA 106+80 to 125+10	0.0	19091	2053	$57,273.45	$369,571.25	$426,844.69	3.0
STA 125+10 to 141+20	0.0	15953	1736	$47,858.84	$312,495.53	$360,354.37	2.6
STA 141+20 to 166+20	0.0	24230	2651	$72,690.85	$477,120.91	$549,811.76	3.9
STA 166+20 to 173+10	0.0	6207	692	$18,621.66	$124,480.32	$143,101.98	1.0
STA 173+10 to 184+30	0.0	12229	1302	$36,686.14	$234,353.38	$271,039.53	1.9
STA 184+30 to 197+30	0.0	11425	1280	$34,275.34	$230,483.37	$264,758.71	1.9
STA 197+30 to 214+60	0.0	15864	1759	$47,590.98	$316,612.24	$364,203.22	2.6
STA 214+60 to 227+80	0.0	10896	1241	$32,688.98	$223,460.91	$256,149.89	1.9
STA 227+80 to 238+90	0.0	8578	995	$25,733.91	$179,137.55	$204,871.45	1.5
STA 238+90 to 252+80	0.0	11705	1326	$35,113.65	$238,766.94	$273,880.60	2.0
STA 252+80 to 262+60	0.0	7228	850	$21,682.72	$152,970.93	$174,653.64	1.3
STA 262+60 to 274+50	0.0	9442	1087	$28,325.49	$195,732.77	$224,058.26	1.7
STA 274+50 to 288+80	0.0	12174	1376	$36,522.07	$247,627.66	$284,149.73	2.1
STA 288+80 to 303+90	0.0	10344	1243	$31,033.29	$223,821.13	$254,854.43	1.9
STA 303+90 to 314+90	0.0	6982	860	$20,945.14	$154,739.02	$175,684.16	1.3
STA 314+90 to 323+80	0.0	6058	730	$18,172.61	$131,328.41	$149,501.02	1.1
STA 323+80 to 335+80	0.0	7567	934	$22,700.21	$168,061.05	$190,761.27	1.5
STA 335+80 to 349+50	0.0	9059	1101	$27,178.03	$198,179.47	$225,357.50	1.7
STA 349+50 to 364+20	0.0	4426	740	$13,279.34	$133,232.71	$146,512.06	1.3
STA 364+20 to 375+00	0.0	3090	530	$9,270.74	$95,457.70	$104,728.44	0.9
STA 375+00 to 389+00	0.0	4766	751	$14,296.88	$135,137.73	$149,434.61	1.3
TOTALS:		335042	37620	$1,005,126.14	$6,771,550.70	$7,776,676.83	56.5

TABLE C.11
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

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	Average Channel Depth (ft)	Average Channel Topwidth (ft)	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)
Start @ 5+00													
STA 5+00 to 22+30	1730	2899	0.0067	0.050	4.00	3.0	187.0	4.9	0.51	6.0	235.0	0.0	81130
STA 22+30 to 42+30	2000	2899	0.0067	0.050	4.00	3.0	187.0	4.9	0.51	6.0	235.0	0.0	93792
STA 42+30 to 68+90	2660	2910	0.0067	0.050	4.00	3.0	187.8	4.9	0.51	6.0	235.8	0.0	125172
STA 68+90 to 90+80	2190	2910	0.0067	0.050	4.00	3.0	187.8	4.9	0.51	6.0	235.8	0.0	103055
STA 90+80 to 106+80	1600	2910	0.0067	0.050	4.00	3.0	187.8	4.9	0.51	6.0	235.8	0.0	75292
STA 106+80 to 125+10	1830	2910	0.0067	0.050	4.00	3.0	187.8	4.9	0.51	6.0	235.8	0.0	86115
STA 125+10 to 141+20	1610	2910	0.0067	0.050	4.00	3.0	187.8	4.9	0.51	6.0	235.8	0.0	75762
STA 141+20 to 166+20	2500	2782	0.0067	0.050	4.00	3.0	179.3	4.8	0.51	6.0	227.3	0.0	112949
STA 166+20 to 173+10	690	2782	0.0067	0.050	4.00	3.0	179.3	4.8	0.51	6.0	227.3	0.0	31174
STA 173+10 to 184+30	1120	2782	0.0067	0.050	4.00	3.0	179.3	4.8	0.51	6.0	227.3	0.0	50601
STA 184+30 to 197+30	1300	2782	0.0067	0.050	4.00	3.0	179.3	4.8	0.51	6.0	227.3	0.0	58733
STA 197+30 to 214+60	1730	2546	0.0067	0.050	4.00	3.0	163.7	4.8	0.51	6.0	211.7	0.0	72170
STA 214+60 to 227+80	1320	2546	0.0067	0.050	4.00	3.0	163.7	4.8	0.51	6.0	211.7	0.0	55066
STA 227+80 to 238+90	1110	2546	0.0067	0.050	4.00	3.0	163.7	4.8	0.51	6.0	211.7	0.0	46305
STA 238+90 to 252+80	1390	2546	0.0067	0.050	4.00	3.0	163.7	4.8	0.51	6.0	211.7	0.0	57986
STA 252+80 to 262+60	980	2546	0.0067	0.050	4.00	3.0	163.7	4.8	0.51	6.0	211.7	0.0	40882
STA 262+60 to 274+50	1190	2546	0.0067	0.050	4.00	3.0	163.7	4.8	0.51	6.0	211.7	0.0	49643
STA 274+50 to 288+80	1430	2546	0.0067	0.050	4.00	3.0	163.7	4.8	0.51	6.0	211.7	0.0	59655
STA 288+80 to 303+90	1510	1856	0.0067	0.050	4.00	3.0	118.1	4.8	0.51	6.0	166.1	0.0	47688
STA 303+90 to 314+90	1100	1856	0.0067	0.050	4.00	3.0	118.1	4.8	0.51	6.0	166.1	0.0	34740
STA 314+90 to 323+80	890	1856	0.0067	0.050	4.00	3.0	118.1	4.8	0.51	6.0	166.1	0.0	28107
STA 323+80 to 335+80	1200	1856	0.0067	0.050	4.00	3.0	118.1	4.8	0.51	6.0	166.1	0.0	37898
STA 335+80 to 349+50	1370	1856	0.0067	0.050	4.00	3.0	118.1	4.8	0.51	6.0	166.1	0.0	43267
STA 349+50 to 364+20	1470	502	0.0067	0.050	4.00	3.0	27.8	4.2	0.49	6.0	75.8	0.0	16920
STA 364+20 to 375+00	1080	502	0.0067	0.050	4.00	3.0	27.8	4.2	0.49	6.0	75.8	0.0	12431
STA 375+00 to 389+00	1400	502	0.0067	0.050	4.00	3.0	27.8	4.2	0.49	6.0	75.8	0.0	16115
TOTALS:	38400												1512649

TABLE C.11
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

Channel Reach	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 12 ft Maintenance Road (acres)	Number Of Drop Structures In This Reach	Unit Discharge For This Reach (cfs/ft)	Drop Structure Cost Per LF Of Channel Bottomwidth
Start @ 5+00								
STA 5+00 to 22+30	0	$162,259.86	$0.00	$162,259.86	9.9	0	15.5	$99
STA 22+30 to 42+30	0	$187,583.65	$0.00	$187,583.65	11.4	2	15.5	$99
STA 42+30 to 68+90	0	$250,344.65	$0.00	$250,344.65	15.3	0	15.5	$99
STA 68+90 to 90+80	0	$206,110.82	$0.00	$206,110.82	12.6	2	15.5	$99
STA 90+80 to 106+80	0	$150,583.25	$0.00	$150,583.25	9.2	3	15.5	$99
STA 106+80 to 125+10	0	$172,229.59	$0.00	$172,229.59	10.5	3	15.5	$99
STA 125+10 to 141+20	0	$151,524.39	$0.00	$151,524.39	9.2	3	15.5	$99
STA 141+20 to 166+20	0	$225,897.85	$0.00	$225,897.85	13.8	5	15.5	$99
STA 166+20 to 173+10	0	$62,347.81	$0.00	$62,347.81	3.8	1	15.5	$99
STA 173+10 to 184+30	0	$101,202.24	$0.00	$101,202.24	6.2	1	15.5	$99
STA 184+30 to 197+30	0	$117,466.88	$0.00	$117,466.88	7.2	4	15.5	$99
STA 197+30 to 214+60	0	$144,339.52	$0.00	$144,339.52	9.0	3	15.6	$100
STA 214+60 to 227+80	0	$110,131.89	$0.00	$110,131.89	6.8	4	15.6	$100
STA 227+80 to 238+90	0	$92,610.91	$0.00	$92,610.91	5.8	4	15.6	$100
STA 238+90 to 252+80	0	$115,972.22	$0.00	$115,972.22	7.2	3	15.6	$100
STA 252+80 to 262+60	0	$81,764.58	$0.00	$81,764.58	5.1	5	15.6	$100
STA 262+60 to 274+50	0	$99,285.57	$0.00	$99,285.57	6.2	4	15.6	$100
STA 274+50 to 288+80	0	$119,309.54	$0.00	$119,309.54	7.4	3	15.6	$100
STA 288+80 to 303+90	0	$95,375.99	$0.00	$95,375.99	6.2	4	15.7	$101
STA 303+90 to 314+90	0	$69,479.20	$0.00	$69,479.20	4.5	4	15.7	$101
STA 314+90 to 323+80	0	$56,214.99	$0.00	$56,214.99	3.7	2	15.7	$101
STA 323+80 to 335+80	0	$75,795.49	$0.00	$75,795.49	5.0	4	15.7	$101
STA 335+80 to 349+50	0	$86,533.19	$0.00	$86,533.19	5.7	3	15.7	$101
STA 349+50 to 364+20	0	$33,840.98	$0.00	$33,840.98	3.0	4	18.1	$105
STA 364+20 to 375+00	0	$24,862.76	$0.00	$24,862.76	2.2	3	18.1	$105
STA 375+00 to 389+00	0	$32,229.51	$0.00	$32,229.51	2.9	1	18.1	$105
TOTALS:	0	$3,025,297.32	$0.00	$3,025,297.32	189.8	75		

TABLE C.11
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

Channel Reach	Total Drop Structure Cost For This Reach	Bank Lining Cost At Each Drop Structure ($/Drop)	Cost Of Bank Lining Cutoff Wall At Each Drop Structure ($/Drop)	Total Bank Lining Cost For This Reach	Total Drop Structure Cost With Bank Lining
Start @ 5+00					
STA 5+00 to 22+30	$0	$15,750	$740	$0	$0
STA 22+30 to 42+30	$37,032	$15,750	$740	$32,981	$70,013
STA 42+30 to 68+90	$0	$15,750	$740	$0	$0
STA 68+90 to 90+80	$37,176	$15,750	$740	$32,981	$70,157
STA 90+80 to 106+80	$55,764	$15,750	$740	$49,471	$105,235
STA 106+80 to 125+10	$55,764	$15,750	$740	$49,471	$105,235
STA 125+10 to 141+20	$55,764	$15,750	$740	$49,471	$105,235
STA 141+20 to 166+20	$88,757	$15,750	$740	$82,452	$171,209
STA 166+20 to 173+10	$17,751	$15,750	$740	$16,490	$34,242
STA 173+10 to 184+30	$17,751	$15,750	$740	$16,490	$34,242
STA 184+30 to 197+30	$71,006	$15,750	$740	$65,961	$136,967
STA 197+30 to 214+60	$49,117	$15,800	$740	$49,621	$98,739
STA 214+60 to 227+80	$65,490	$15,800	$740	$66,161	$131,651
STA 227+80 to 238+90	$65,490	$15,800	$740	$66,161	$131,651
STA 238+90 to 252+80	$49,117	$15,800	$740	$49,621	$98,739
STA 252+80 to 262+60	$81,862	$15,800	$740	$82,702	$164,564
STA 262+60 to 274+50	$65,490	$15,800	$740	$66,161	$131,651
STA 274+50 to 288+80	$49,117	$15,800	$740	$49,621	$98,739
STA 288+80 to 303+90	$47,719	$15,850	$740	$66,361	$114,081
STA 303+90 to 314+90	$47,719	$15,850	$740	$66,361	$114,081
STA 314+90 to 323+80	$23,860	$15,850	$740	$33,181	$57,040
STA 323+80 to 335+80	$47,719	$15,850	$740	$66,361	$114,081
STA 335+80 to 349+50	$35,789	$15,850	$740	$49,771	$85,560
STA 349+50 to 364+20	$11,675	$16,500	$740	$68,961	$80,636
STA 364+20 to 375+00	$8,756	$16,500	$740	$51,721	$60,477
STA 375+00 to 389+00	$2,919	$16,500	$740	$17,240	$20,159
TOTALS:	$1,088,607			$1,245,778	$2,334,385

TABLE C.12
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With Low-Flow Diverts
City of Phoenix, Areas C & D
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

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 22+30	1730	2899	0.0058	0.035	2.00	4.0	87.2	7.6	0.70	5.5	109.2
STA 22+30 to 42+30	2000	2899	0.0100	0.035	2.00	4.0	65.7	9.8	0.91	5.5	87.7
STA 42+30 to 68+90	2660	2910	0.0075	0.035	2.00	4.0	76.4	8.6	0.79	5.5	98.4
STA 68+90 to 90+80	2190	2910	0.0091	0.035	2.00	4.0	69.1	9.4	0.87	5.5	91.1
STA 90+80 to 106+80	1600	2910	0.0125	0.035	2.00	4.0	58.7	10.9	1.02	5.5	80.7
STA 106+80 to 125+10	1830	2910	0.0109	0.035	2.00	4.0	63.0	10.3	0.95	5.5	85.0
STA 125+10 to 141+20	1610	2910	0.0124	0.035	2.00	4.0	58.9	10.9	1.01	5.5	80.9
STA 141+20 to 166+20	2500	2782	0.0120	0.035	2.00	4.0	57.2	10.7	1.00	5.5	79.2
STA 166+20 to 173+10	690	2782	0.0145	0.035	2.00	4.0	51.8	11.6	1.09	5.5	73.8
STA 173+10 to 184+30	1120	2782	0.0089	0.035	2.00	4.0	66.7	9.3	0.86	5.5	88.7
STA 184+30 to 197+30	1300	2782	0.0154	0.035	2.00	4.0	50.2	12.0	1.12	5.5	72.2
STA 197+30 to 214+60	1730	2546	0.0116	0.035	2.00	4.0	53.1	10.4	0.98	5.5	75.1
STA 214+60 to 227+80	1320	2546	0.0152	0.035	2.00	4.0	46.1	11.8	1.11	5.5	68.1
STA 227+80 to 238+90	1110	2546	0.0180	0.035	2.00	4.0	42.0	12.7	1.21	5.5	64.0
STA 238+90 to 252+80	1390	2546	0.0144	0.035	2.00	4.0	47.3	11.5	1.08	5.5	69.3
STA 252+80 to 262+60	980	2546	0.0204	0.035	2.00	4.0	39.3	13.5	1.28	5.5	61.3
STA 262+60 to 274+50	1190	2546	0.0168	0.035	2.00	4.0	43.6	12.3	1.17	5.5	65.6
STA 274+50 to 288+80	1430	2546	0.0140	0.035	2.00	4.0	48.1	11.4	1.07	5.5	70.1
STA 288+80 to 303+90	1510	1856	0.0132	0.035	2.00	4.0	35.2	10.7	1.03	5.5	57.2
STA 303+90 to 314+90	1100	1856	0.0164	0.035	2.00	4.0	31.4	11.8	1.14	5.5	53.4
STA 314+90 to 323+80	890	1856	0.0135	0.035	2.00	4.0	34.9	10.8	1.04	5.5	56.9
STA 323+80 to 335+80	1200	1856	0.0167	0.035	2.00	4.0	31.1	11.9	1.15	5.5	53.1
STA 335+80 to 349+50	1370	1856	0.0146	0.035	2.00	4.0	33.4	11.2	1.08	5.5	55.4
STA 349+50 to 364+20	1470	502	0.0136	0.035	2.00	4.0	6.3	8.8	0.97	5.5	28.3
STA 364+20 to 375+00	1080	502	0.0167	0.035	2.00	4.0	5.2	9.5	1.06	5.5	27.2
STA 375+00 to 389+00	1400	502	0.0086	0.035	2.00	4.0	9.2	7.3	0.78	5.5	31.2
TOTALS:	38400										

TABLE C.12
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, With Low-Flow Diverts
City of Phoenix, Areas C & D
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

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 22+30	5.0	46322	10764	$92,644.13	$258,346.67	$350,990.80	5.2
STA 22+30 to 42+30	5.0	44794	12444	$89,588.53	$298,666.67	$388,255.19	5.0
STA 42+30 to 68+90	5.0	65404	16551	$130,807.32	$397,226.67	$528,033.99	7.4
STA 68+90 to 90+80	5.0	50584	13627	$101,168.79	$327,040.00	$428,208.79	5.7
STA 90+80 to 106+80	5.0	33560	9956	$67,119.07	$238,933.33	$306,052.40	3.8
STA 106+80 to 125+10	5.0	39973	11387	$79,945.90	$273,280.00	$353,225.90	4.5
STA 125+10 to 141+20	5.0	33832	10018	$67,664.23	$240,426.67	$308,090.90	3.8
STA 141+20 to 166+20	5.0	51678	15556	$103,355.20	$373,333.33	$476,688.53	5.8
STA 166+20 to 173+10	5.0	13504	4293	$27,007.46	$103,040.00	$130,047.46	1.5
STA 173+10 to 184+30	5.0	25328	6969	$50,655.27	$167,253.33	$217,908.60	2.8
STA 184+30 to 197+30	5.0	25016	8089	$50,032.19	$194,133.33	$244,165.52	2.8
STA 197+30 to 214+60	5.0	34332	10764	$68,664.00	$258,346.67	$327,010.66	3.9
STA 214+60 to 227+80	5.0	24290	8213	$48,579.78	$197,120.00	$245,699.78	2.7
STA 227+80 to 238+90	5.0	19505	6907	$39,010.19	$165,760.00	$204,770.19	2.2
STA 238+90 to 252+80	5.0	25941	8649	$51,882.13	$207,573.33	$259,455.47	2.9
STA 252+80 to 262+60	5.0	16678	6098	$33,355.01	$146,346.67	$179,701.68	1.9
STA 262+60 to 274+50	5.0	21297	7404	$42,594.44	$177,706.67	$220,301.10	2.4
STA 274+50 to 288+80	5.0	26897	8898	$53,793.42	$213,546.67	$267,340.09	3.0
STA 288+80 to 303+90	5.0	24455	9396	$48,910.64	$225,493.33	$274,403.97	2.7
STA 303+90 to 314+90	5.0	16949	6844	$33,898.37	$164,266.67	$198,165.03	1.9
STA 314+90 to 323+80	5.0	14352	5538	$28,704.43	$132,906.67	$161,611.10	1.6
STA 323+80 to 335+80	5.0	18412	7467	$36,825.00	$179,200.00	$216,025.00	2.1
STA 335+80 to 349+50	5.0	21678	8524	$43,355.84	$204,586.67	$247,942.51	2.4
STA 349+50 to 364+20	5.0	15138	9147	$30,276.76	$219,520.00	$249,796.76	1.7
STA 364+20 to 375+00	5.0	10882	6720	$21,764.91	$161,280.00	$183,044.91	1.2
STA 375+00 to 389+00	5.0	15238	8711	$30,476.93	$209,066.67	$239,543.60	1.7
TOTALS:		736040	238933	$1,472,079.94	$5,734,400.00	$7,206,479.94	82.7

TABLE C.13
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, Pinnacle Peak Road Alignment
With Low-Flow Culverts
City of Phoenix, Areas C & D
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

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 22+30	1730	3955	0.0058	0.035	2.00	4.0	119.8	7.7	0.70	5.5	141.8
STA 22+30 to 42+30	2000	3955	0.0100	0.035	2.00	4.0	90.5	10.0	0.92	5.5	112.5
STA 42+30 to 69+90	2760	3955	0.0072	0.035	2.00	4.0	106.7	8.6	0.79	5.5	128.7
STA 69+90 to 95+90	2600	3955	0.0077	0.035	2.00	4.0	103.5	8.9	0.81	5.5	125.5
STA 95+90 to 118+10	2220	3955	0.0090	0.035	2.00	4.0	95.5	9.6	0.87	5.5	117.5
STA 118+10 to 136+10	1800	3314	0.0056	0.035	2.00	4.0	102.0	7.5	0.69	5.5	124.0
STA 136+10 to 147+60	1150	3314	0.0087	0.035	2.00	4.0	81.1	9.3	0.86	5.5	103.1
STA 147+60 to 168+80	2120	3314	0.0094	0.035	2.00	4.0	77.7	9.7	0.89	5.5	99.7
STA 168+80 to 192+60	2380	3314	0.0084	0.035	2.00	4.0	82.5	9.2	0.84	5.5	104.5
STA 192+60 to 208+70	1610	2273	0.0062	0.035	2.00	4.0	65.3	7.7	0.72	5.5	87.3
STA 208+70 to 220+50	1180	2273	0.0085	0.035	2.00	4.0	55.5	8.9	0.84	5.5	77.5
STA 220+50 to 230+00	950	2273	0.0105	0.035	2.00	4.0	49.5	9.9	0.93	5.5	71.5
STA 230+00 to 240+30	1030	2273	0.0097	0.035	2.00	4.0	51.7	9.5	0.89	5.5	73.7
STA 240+30 to 253+30	1300	2273	0.0077	0.035	2.00	4.0	58.4	8.6	0.80	5.5	80.4
STA 253+30 to 264+80	1150	780	0.0087	0.035	2.00	4.0	16.5	8.0	0.81	5.5	38.5
STA 264+80 to 274+00	920	780	0.0109	0.035	2.00	4.0	14.3	8.7	0.90	5.5	36.3
STA 274+00 to 286+00	1200	780	0.0083	0.035	2.00	4.0	16.9	7.8	0.79	5.5	38.9
STA 286+00 to 294+80	880	780	0.0114	0.035	2.00	4.0	13.9	8.9	0.92	5.5	35.9
STA 294+80 to 304+20	940	780	0.0106	0.035	2.00	4.0	14.5	8.7	0.89	5.5	36.5
TOTALS:	29920										

TABLE C.13
Hydraulic Data & Cost Estimates for East-West Desert Ridge Channel, Pinnacle Peak Road Alignment
With Low-Flow Culverts
City of Phoenix, Areas C & D

Excavation Cost/c.y.:	$2.00
Channel Lining: Soil Cement Banks, Ear	8.00
Horizontal Lining Width (ft):	$24.00
Lining Cost/c.y.:	

All Calculations Are Based On Existing Ground Slopes

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 22+30	5.0	57804	10764	$115,607.80	$258,346.67	$373,954.47	6.5
STA 22+30 to 42+30	5.0	54910	12444	$109,819.15	$298,666.67	$408,485.82	6.2
STA 42+30 to 69+90	5.0	84892	17173	$169,783.58	$412,160.00	$581,943.58	9.5
STA 69+90 to 95+90	5.0	78273	16178	$156,545.49	$388,266.67	$544,812.16	8.8
STA 95+90 to 118+10	5.0	63198	13813	$126,396.96	$331,520.00	$457,916.96	7.1
STA 118+10 to 136+10	5.0	53645	11200	$107,290.56	$268,800.00	$376,090.56	6.0
STA 136+10 to 147+60	5.0	29366	7156	$58,731.51	$171,733.33	$230,464.84	3.3
STA 147+60 to 168+80	5.0	52695	13191	$105,389.46	$316,586.67	$421,976.13	5.9
STA 168+80 to 192+60	5.0	61472	14809	$122,944.93	$355,413.33	$478,358.26	6.9
STA 192+60 to 208+70	5.0	35944	10018	$71,888.40	$240,426.67	$312,315.07	4.0
STA 208+70 to 220+50	5.0	23992	7342	$47,983.23	$176,213.33	$224,196.56	2.7
STA 220+50 to 230+00	5.0	18155	5911	$36,310.07	$141,866.67	$178,176.74	2.0
STA 230+00 to 240+30	5.0	20137	6409	$40,274.84	$153,813.33	$194,088.18	2.3
STA 240+30 to 253+30	5.0	27197	8089	$54,394.39	$194,133.33	$248,527.72	3.1
STA 253+30 to 264+80	5.0	14238	7156	$28,475.66	$171,733.33	$200,208.99	1.6
STA 264+80 to 274+00	5.0	10981	5724	$21,962.94	$137,386.67	$159,349.61	1.2
STA 274+00 to 286+00	5.0	14965	7467	$29,929.75	$179,200.00	$209,129.75	1.7
STA 286+00 to 294+80	5.0	10431	5476	$20,861.87	$131,413.33	$152,275.21	1.2
STA 294+80 to 304+20	5.0	11259	5849	$22,517.09	$140,373.33	$162,890.42	1.3
TOTALS:		723554	186169	$1,447,107.70	$4,468,053.33	$5,915,161.03	81.4

TABLE C.14
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Concrete (banks & bottom)
Horizontal Lining Width (ft): 1.12
Lining Cost/c.y.: $180.00
All Calculations Are Based On Existing Ground Slopes

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 16+20	1120	11775	0.0089	0.020	2.00	4.0	164.7	17.0	1.54	5.5	186.7
STA 16+20 to 25+80	960	11775	0.0104	0.020	2.00	4.0	152.3	18.4	1.66	5.5	174.3
STA 25+80 to 34+80	900	11775	0.0111	0.020	2.00	4.0	147.4	18.9	1.71	5.5	169.4
STA 34+80 to 50+40	1560	11775	0.0128	0.020	2.00	4.0	137.1	20.3	1.84	5.5	159.1
STA 50+40 to 66+10	1570	11775	0.0127	0.020	2.00	4.0	137.5	20.2	1.83	5.5	159.5
STA 66+10 to 80+10	1400	11775	0.0143	0.020	2.00	4.0	129.8	21.4	1.94	5.5	151.8
STA 80+10 to 95+10	1500	11775	0.0133	0.020	2.00	4.0	134.4	20.7	1.87	5.5	156.4
STA 95+10 to 107+50	1240	11775	0.0161	0.020	2.00	4.0	122.0	22.6	2.06	5.5	144.0
STA 107+50 to 123+00	1550	11775	0.0129	0.020	2.00	4.0	136.7	20.4	1.84	5.5	158.7
STA 123+00 to 128+00	500	11775	0.0200	0.020	2.00	4.0	109.3	25.1	2.28	5.5	131.3
STA 128+00 to 140+80	1280	11775	0.0156	0.020	2.00	4.0	124.0	22.3	2.02	5.5	146.0
STA 140+80 to 153+20	1240	11775	0.0161	0.020	2.00	4.0	122.0	22.6	2.06	5.5	144.0
STA 153+20 to 171+50	1830	11775	0.0164	0.020	2.00	4.0	121.0	22.8	2.07	5.5	143.0
STA 171+50 to 177+30	580	10636	0.0172	0.020	2.00	4.0	106.3	23.3	2.12	5.5	128.3
STA 177+30 to 189+70	1240	10636	0.0161	0.020	2.00	4.0	110.0	22.5	2.05	5.5	132.0
STA 189+70 to 201+00	1130	10636	0.0177	0.020	2.00	4.0	104.9	23.6	2.15	5.5	126.9
STA 201+00 to 210+70	970	10636	0.0206	0.020	2.00	4.0	97.0	25.3	2.31	5.5	119.0
STA 210+70 to 221+30	1060	10636	0.0189	0.020	2.00	4.0	101.5	24.3	2.22	5.5	123.5
STA 221+30 to 231+30	1000	10636	0.0200	0.020	2.00	4.0	98.5	25.0	2.28	5.5	120.5
STA 231+30 to 244+40	1310	10636	0.0153	0.020	2.00	4.0	113.1	22.0	2.00	5.5	135.1
STA 244+40 to 250+60	620	10636	0.0161	0.020	2.00	4.0	110.0	22.5	2.05	5.5	132.0
STA 250+60 to 264+50	1390	1106	0.0144	0.020	2.00	4.0	8.8	16.5	1.77	5.5	30.8
STA 264+50 to 274+00	950	1106	0.0211	0.020	2.00	4.0	6.4	19.2	2.11	5.5	28.4
STA 274+00 to 286+90	1290	1106	0.0155	0.020	2.00	4.0	8.3	17.0	1.83	5.5	30.3
STA 286+90 to 296+60	970	1106	0.0206	0.020	2.00	4.0	6.5	19.0	2.09	5.5	28.5
STA 296+60 to 310+50	1390	1106	0.0144	0.020	2.00	4.0	8.8	16.5	1.77	5.5	30.8
STA 310+50 to 321+70	1120	1106	0.0179	0.020	2.00	4.0	7.4	18.0	1.95	5.5	29.4
TOTALS:	31670										

TABLE C.14
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Concrete (banks & bottom)
Horizontal Lining Width (ft): 1.12
Lining Cost/c.y.: $180.00
All Calculations Are Based On Existing Ground Slopes

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 16+20	0.0	44036	3953	$88,072.27	$711,464.66	$799,536.93	5.2
STA 16+20 to 25+80	0.0	17692	3168	$1,593.71	$570,257.99	$571,851.70	4.2
STA 25+80 to 34+80	0.0	31933	2888	$63,865.66	$519,912.48	$583,778.14	3.8
STA 34+80 to 50+40	0.0	51772	4708	$103,543.92	$847,507.38	$951,051.30	6.2
STA 50+40 to 66+10	0.0	52259	4752	$104,518.44	$855,270.95	$959,789.38	6.3
STA 66+10 to 80+10	0.0	44181	4035	$88,362.24	$726,370.16	$814,732.40	5.4
STA 80+10 to 95+10	0.0	48880	4452	$97,759.50	$801,396.26	$899,155.77	5.9
STA 95+10 to 107+50	0.0	36992	3396	$73,983.86	$611,257.64	$685,241.51	4.5
STA 107+50 to 123+00	0.0	51286	4665	$102,572.37	$839,766.08	$942,338.45	6.2
STA 123+00 to 128+00	0.0	13509	1252	$27,017.97	$225,368.13	$252,386.10	1.7
STA 128+00 to 140+80	0.0	38750	3553	$77,500.55	$639,451.42	$716,951.97	4.7
STA 140+80 to 153+20	0.0	36992	3396	$73,983.86	$611,257.64	$685,241.51	4.5
STA 153+20 to 171+50	0.0	54184	4978	$108,368.19	$895,965.43	$1,004,333.62	6.6
STA 171+50 to 177+30	0.0	15280	1420	$30,560.04	$255,570.96	$286,131.00	1.9
STA 177+30 to 189+70	0.0	33682	3120	$67,363.52	$561,605.09	$628,968.61	4.2
STA 189+70 to 201+00	0.0	29415	2737	$58,829.79	$492,600.73	$551,430.52	3.7
STA 201+00 to 210+70	0.0	23552	2208	$47,104.05	$397,383.04	$444,487.09	3.0
STA 210+70 to 221+30	0.0	26798	2501	$53,596.83	$450,170.90	$503,767.73	3.4
STA 221+30 to 231+30	0.0	24619	2304	$49,238.28	$414,753.75	$463,992.03	3.1
STA 231+30 to 244+40	0.0	36492	3372	$72,984.83	$606,947.54	$679,932.37	4.5
STA 244+40 to 250+60	0.0	16841	1560	$33,681.76	$280,802.54	$314,484.31	2.1
STA 250+60 to 264+50	0.0	6490	892	$12,979.42	$160,544.34	$173,523.77	1.5
STA 264+50 to 274+00	0.0	3939	568	$7,877.73	$102,276.28	$110,154.01	0.9
STA 274+00 to 286+90	0.0	5881	816	$11,762.59	$146,871.40	$158,633.99	1.3
STA 286+90 to 296+60	0.0	4047	582	$8,094.66	$104,812.59	$112,907.25	1.0
STA 296+60 to 310+50	0.0	6490	892	$12,979.36	$160,543.87	$173,523.23	1.5
STA 310+50 to 321+70	0.0	4885	690	$9,769.46	$124,193.59	$133,963.05	1.1
TOTALS:		760878	72857	$1,487,964.85	$13,114,322.86	$14,602,287.72	98.5

TABLE C.15
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

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	Average Channel Depth (ft/ft)	Average Channel Topwidth (ft)	Approximate Toedown Depth (ft)
Start @ 5+00												
STA 5+00 to 16+20	1120	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 16+20 to 25+80	960	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 25+80 to 34+80	900	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 34+80 to 50+40	1560	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 50+40 to 66+10	1570	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 66+10 to 80+10	1400	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 80+10 to 95+10	1500	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 95+10 to 107+50	1240	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 107+50 to 123+00	1550	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 123+00 to 128+00	500	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 128+00 to 140+80	1280	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 140+80 to 153+20	1240	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 153+20 to 171+50	1830	11775	0.0067	0.050	4.00	3.0	772.0	5.0	0.51	6.0	820.0	0.0
STA 171+50 to 177+30	580	10636	0.0067	0.050	4.00	3.0	697.0	5.0	0.51	6.0	745.0	0.0
STA 177+30 to 189+70	1240	10636	0.0067	0.050	4.00	3.0	697.0	5.0	0.51	6.0	745.0	0.0
STA 189+70 to 201+00	1130	10636	0.0067	0.050	4.00	3.0	697.0	5.0	0.51	6.0	745.0	0.0
STA 201+00 to 210+70	970	10636	0.0067	0.050	4.00	3.0	697.0	5.0	0.51	6.0	745.0	0.0
STA 210+70 to 221+30	1060	10636	0.0067	0.050	4.00	3.0	697.0	5.0	0.51	6.0	745.0	0.0
STA 221+30 to 231+30	1000	10636	0.0067	0.050	4.00	3.0	697.0	5.0	0.51	6.0	745.0	0.0
STA 231+30 to 244+40	1310	10636	0.0067	0.050	4.00	3.0	697.0	5.0	0.51	6.0	745.0	0.0
STA 244+40 to 250+60	620	10636	0.0067	0.050	4.00	3.0	697.0	5.0	0.51	6.0	745.0	0.0
STA 250+60 to 264+50	1390	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0
STA 264+50 to 274+00	950	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0
STA 274+00 to 286+90	1290	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0
STA 286+90 to 296+60	970	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0
STA 296+60 to 310+50	1390	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0
STA 310+50 to 321+70	1120	1106	0.0067	0.050	4.00	3.0	68.4	4.6	0.50	6.0	116.4	0.0
TOTALS:	31670											

TABLE C.15
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

Channel Reach	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)	Number Of Drop Structures In This Reach	Unit Discharge For This Reach (cfs/ft)
Start @ 5+00								
STA 5+00 to 16+20	198127	0	$396,253.79	$0.00	$396,253.79	21.4	0	15.3
STA 16+20 to 25+80	169823	0	$339,646.11	$0.00	$339,646.11	18.4	1	15.3
STA 25+80 to 34+80	159209	0	$318,418.23	$0.00	$318,418.23	17.2	1	15.3
STA 34+80 to 50+40	275962	0	$551,924.93	$0.00	$551,924.93	29.9	3	15.3
STA 50+40 to 66+10	277731	0	$555,462.91	$0.00	$555,462.91	30.1	4	15.3
STA 66+10 to 80+10	247659	0	$495,317.24	$0.00	$495,317.24	26.8	3	15.3
STA 80+10 to 95+10	265349	0	$530,697.04	$0.00	$530,697.04	28.7	4	15.3
STA 95+10 to 107+50	219355	0	$438,709.56	$0.00	$438,709.56	23.7	4	15.3
STA 107+50 to 123+00	274193	0	$548,386.95	$0.00	$548,386.95	29.7	3	15.3
STA 123+00 to 128+00	88450	0	$176,899.01	$0.00	$176,899.01	9.6	2	15.3
STA 128+00 to 140+80	226431	0	$452,861.48	$0.00	$452,861.48	24.5	4	15.3
STA 140+80 to 153+20	219355	0	$438,709.56	$0.00	$438,709.56	23.7	5	15.3
STA 153+20 to 171+50	323725	0	$647,450.39	$0.00	$647,450.39	35.0	6	15.3
STA 171+50 to 177+30	92929	0	$185,857.96	$0.00	$185,857.96	10.1	2	15.3
STA 177+30 to 189+70	198676	0	$397,351.51	$0.00	$397,351.51	21.6	4	15.3
STA 189+70 to 201+00	181051	0	$362,102.58	$0.00	$362,102.58	19.7	5	15.3
STA 201+00 to 210+70	155416	0	$310,831.42	$0.00	$310,831.42	16.9	4	15.3
STA 210+70 to 221+30	169836	0	$339,671.45	$0.00	$339,671.45	18.5	5	15.3
STA 221+30 to 231+30	160222	0	$320,444.76	$0.00	$320,444.76	17.4	4	15.3
STA 231+30 to 244+40	209891	0	$419,782.64	$0.00	$419,782.64	22.8	4	15.3
STA 244+40 to 250+60	99338	0	$198,675.75	$0.00	$198,675.75	10.8	2	15.3
STA 250+60 to 264+50	28531	0	$57,061.01	$0.00	$57,061.01	4.2	4	16.2
STA 264+50 to 274+00	19499	0	$38,998.53	$0.00	$38,998.53	2.8	4	16.2
STA 274+00 to 286+90	26478	0	$52,955.90	$0.00	$52,955.90	3.9	4	16.2
STA 286+90 to 296+60	19910	0	$39,819.55	$0.00	$39,819.55	2.9	5	16.2
STA 296+60 to 310+50	28531	0	$57,061.01	$0.00	$57,061.01	4.2	3	16.2
STA 310+50 to 321+70	22989	0	$45,977.21	$0.00	$45,977.21	3.4	4	16.2
TOTALS:	4358664	0	$8,717,328.46	$0.00	$8,717,328.46	477.9	94	

TABLE C.15
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
Excavation Cost/c.y.: $2.00
Channel Lining: Earth, Banks & Bottom
Horizontal Lining Width (ft): 0.00
Lining Cost/c.y.: $0.00
Calculations Are Based On A Constant Bed-Slope Of .0067 ft/ft
Drop Structure Height = 3 Ft

Channel Reach	Drop Structure Cost Per LF Of Channel Bottomwidth	Total Drop Structure Cost For This Reach	Bank Lining Cost At Each Drop Structure ($/Drop)	Cost Of Bank Lining Cutoff Wall At Each Drop Structure ($/Drop)	Total Bank Lining Cost For This Reach	Total Drop Structure Cost With Bank Lining
Start @ 5+00						
STA 5+00 to 16+20	$99	$0	$15,550	$740	$0	$0
STA 16+20 to 25+80	$99	$76,433	$15,550	$740	$16,290	$92,723
STA 25+80 to 34+80	$99	$76,433	$15,550	$740	$16,290	$92,723
STA 34+80 to 50+40	$99	$229,298	$15,550	$740	$48,871	$278,169
STA 50+40 to 66+10	$99	$305,730	$15,550	$740	$65,161	$370,892
STA 66+10 to 80+10	$99	$229,298	$15,550	$740	$48,871	$278,169
STA 80+10 to 95+10	$99	$305,730	$15,550	$740	$65,161	$370,892
STA 95+10 to 107+50	$99	$305,730	$15,550	$740	$65,161	$370,892
STA 107+50 to 123+00	$99	$229,298	$15,550	$740	$48,871	$278,169
STA 123+00 to 128+00	$99	$152,865	$15,550	$740	$32,581	$185,446
STA 128+00 to 140+80	$99	$305,730	$15,550	$740	$65,161	$370,892
STA 140+80 to 153+20	$99	$382,163	$15,550	$740	$81,452	$463,614
STA 153+20 to 171+50	$99	$458,595	$15,550	$740	$97,742	$556,337
STA 171+50 to 177+30	$99	$138,006	$15,550	$740	$32,581	$170,587
STA 177+30 to 189+70	$99	$276,012	$15,550	$740	$65,161	$341,174
STA 189+70 to 201+00	$99	$345,015	$15,550	$740	$81,452	$426,467
STA 201+00 to 210+70	$99	$276,012	$15,550	$740	$65,161	$341,174
STA 210+70 to 221+30	$99	$345,015	$15,550	$740	$81,452	$426,467
STA 221+30 to 231+30	$99	$276,012	$15,550	$740	$65,161	$341,174
STA 231+30 to 244+40	$99	$276,012	$15,550	$740	$65,161	$341,174
STA 244+40 to 250+60	$99	$138,006	$15,550	$740	$32,581	$170,587
STA 250+60 to 264+50	$101	$27,619	$16,000	$740	$66,961	$94,581
STA 264+50 to 274+00	$101	$27,619	$16,000	$740	$66,961	$94,581
STA 274+00 to 286+90	$101	$27,619	$16,000	$740	$66,961	$94,581
STA 286+90 to 296+60	$101	$34,524	$16,000	$740	$83,702	$118,226
STA 296+60 to 310+50	$101	$20,715	$16,000	$740	$50,221	$70,936
STA 310+50 to 321+70	$101	$27,619	$16,000	$740	$66,961	$94,581
TOTALS:		$5,293,109			$1,542,095	$6,835,204

TABLE C.16
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
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

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 1.5 Ft Freeboard	Channel Topwidth With 1.5 Ft Freeboard
Start @ 5+00											
STA 5+00 to 16+20	1120	11775	0.0089	0.035	2.00	4.0	289.6	9.9	0.88	5.5	311.6
STA 16+20 to 25+80	960	11775	0.0104	0.035	2.00	4.0	268.0	10.7	0.95	5.5	290.0
STA 25+80 to 34+80	900	11775	0.0111	0.035	2.00	4.0	259.4	11.0	0.98	5.5	281.4
STA 34+80 to 50+40	1560	11775	0.0128	0.035	2.00	4.0	241.4	11.8	1.06	5.5	263.4
STA 50+40 to 66+10	1570	11775	0.0127	0.035	2.00	4.0	242.2	11.8	1.05	5.5	264.2
STA 66+10 to 80+10	1400	11775	0.0143	0.035	2.00	4.0	228.6	12.4	1.11	5.5	250.6
STA 80+10 to 95+10	1500	11775	0.0133	0.035	2.00	4.0	236.7	12.0	1.08	5.5	258.7
STA 95+10 to 107+50	1240	11775	0.0161	0.035	2.00	4.0	215.0	13.2	1.18	5.5	237.0
STA 107+50 to 123+00	1550	11775	0.0129	0.035	2.00	4.0	240.6	11.8	1.06	5.5	262.6
STA 123+00 to 128+00	500	11775	0.0200	0.035	2.00	4.0	192.9	14.7	1.32	5.5	214.9
STA 128+00 to 140+80	1280	11775	0.0156	0.035	2.00	4.0	218.5	13.0	1.17	5.5	240.5
STA 140+80 to 153+20	1240	11775	0.0161	0.035	2.00	4.0	215.0	13.2	1.18	5.5	237.0
STA 153+20 to 171+50	1830	11775	0.0164	0.035	2.00	4.0	213.3	13.3	1.19	5.5	235.3
STA 171+50 to 177+30	580	10636	0.0172	0.035	2.00	4.0	187.6	13.6	1.22	5.5	209.6
STA 177+30 to 189+70	1240	10636	0.0161	0.035	2.00	4.0	194.0	13.2	1.18	5.5	216.0
STA 189+70 to 201+00	1130	10636	0.0177	0.035	2.00	4.0	185.1	13.8	1.24	5.5	207.1
STA 201+00 to 210+70	970	10636	0.0206	0.035	2.00	4.0	171.4	14.8	1.33	5.5	193.4
STA 210+70 to 221+30	1060	10636	0.0189	0.035	2.00	4.0	179.3	14.2	1.28	5.5	201.3
STA 221+30 to 231+30	1000	10636	0.0200	0.035	2.00	4.0	174.1	14.6	1.31	5.5	196.1
STA 231+30 to 244+40	1310	10636	0.0153	0.035	2.00	4.0	199.5	12.8	1.15	5.5	221.5
STA 244+40 to 250+60	620	10636	0.0161	0.035	2.00	4.0	194.0	13.2	1.18	5.5	216.0
STA 250+60 to 264+50	1390	1106	0.0144	0.035	2.00	4.0	18.6	10.4	1.04	5.5	40.6
STA 264+50 to 274+00	950	1106	0.0211	0.035	2.00	4.0	14.7	12.2	1.25	5.5	36.7
STA 274+00 to 286+90	1290	1106	0.0155	0.035	2.00	4.0	17.8	10.7	1.08	5.5	39.8
STA 286+90 to 296+60	970	1106	0.0206	0.035	2.00	4.0	14.9	12.1	1.24	5.5	36.9
STA 296+60 to 310+50	1390	1106	0.0144	0.035	2.00	4.0	18.6	10.4	1.04	5.5	40.6
STA 310+50 to 321+70	1120	1106	0.0179	0.035	2.00	4.0	16.3	11.4	1.16	5.5	38.3
TOTALS:	31670										

TABLE C.16
Hydraulic Data & Cost Estimates for North-South Desert Ridge Channel, With Low-Flow Culverts
City of Phoenix, Areas C & D
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

Channel Reach	Approximate Toedown Depth (ft)	Total Channel Excavation Volume For This Reach (cy)	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 12 ft Maintenance Road (acres)
Start @ 5+00							
STA 5+00 to 16+20	5.0	76175	6969	$152,350.07	$167,253.33	$319,603.40	8.6
STA 16+20 to 25+80	5.0	61066	5973	$122,132.15	$143,360.00	$265,492.15	6.9
STA 25+80 to 34+80	5.0	55679	5600	$111,357.77	$134,400.00	$245,757.77	6.3
STA 34+80 to 50+40	5.0	90778	9707	$181,555.77	$232,960.00	$414,515.77	10.2
STA 50+40 to 66+10	5.0	91609	9769	$183,217.39	$234,453.33	$417,670.72	10.3
STA 66+10 to 80+10	5.0	77814	8711	$155,628.16	$209,066.67	$364,694.83	8.8
STA 80+10 to 95+10	5.0	85843	9333	$171,686.57	$224,000.00	$395,686.57	9.7
STA 95+10 to 107+50	5.0	65494	7716	$130,988.31	$185,173.33	$316,161.64	7.4
STA 107+50 to 123+00	5.0	89949	9644	$179,898.91	$231,466.67	$411,365.57	10.1
STA 123+00 to 128+00	5.0	24156	3111	$48,312.35	$74,666.67	$122,979.02	2.7
STA 128+00 to 140+80	5.0	68512	7964	$137,023.38	$191,146.67	$328,170.04	7.7
STA 140+80 to 153+20	5.0	65494	7716	$130,988.31	$185,173.33	$316,161.64	7.4
STA 153+20 to 171+50	5.0	96002	11387	$192,004.10	$273,280.00	$465,284.10	10.8
STA 171+50 to 177+30	5.0	27396	3609	$54,792.58	$86,613.33	$141,405.91	3.1
STA 177+30 to 189+70	5.0	60195	7716	$120,389.20	$185,173.33	$305,562.53	6.8
STA 189+70 to 201+00	5.0	52808	7031	$105,615.38	$168,746.67	$274,362.05	5.9
STA 201+00 to 210+70	5.0	42614	6036	$85,227.08	$144,853.33	$230,080.41	4.8
STA 210+70 to 221+30	5.0	48265	6596	$96,530.54	$158,293.33	$254,823.87	5.4
STA 221+30 to 231+30	5.0	44473	6222	$88,946.78	$149,333.33	$238,280.11	5.0
STA 231+30 to 244+40	5.0	65048	8151	$130,096.43	$195,626.67	$325,723.09	7.3
STA 244+40 to 250+60	5.0	30097	3858	$60,194.60	$92,586.67	$152,781.27	3.4
STA 250+60 to 264+50	5.0	17802	8649	$35,604.54	$207,573.33	$243,177.88	2.0
STA 264+50 to 274+00	5.0	11407	5911	$22,814.28	$141,866.67	$164,680.94	1.3
STA 274+00 to 286+90	5.0	16304	8027	$32,608.72	$192,640.00	$225,248.72	1.8
STA 286+90 to 296+60	5.0	11686	6036	$23,372.58	$144,853.33	$168,225.91	1.3
STA 296+60 to 310+50	5.0	17802	8649	$35,604.54	$207,573.33	$243,177.88	2.0
STA 310+50 to 321+70	5.0	13816	6969	$27,632.87	$167,253.33	$194,886.20	1.6
TOTALS:		1408287	197058	$2,816,573.35	$4,729,386.67	$7,545,960.02	158.5

APPENDIX D

MAILING LIST FOR PUBLIC MEETING ATTENDEES
AND
PERIPHERAL AREA C & D PLANNING ADVISORY COMMITTEE MEMBERS

PERIPHERAL AREA C & D PLANNING ADVISORY COMMITTEE

NAME	BUSINESS	RESIDENCE
Mr. Jones Osborn II Chairman	Meyer, Hendricks, Victor Osborn & Maledon 2700 North 3rd Street Phoenix, Arizona 85004 263-8700	
Mr. Bernie Cain	John Hall Associates 11209 N. Tatum Blvd. #260 Phoenix, Arizona 85028 953-4000	3101 East Yucca St. Phoenix, AZ. 85028
Mr. Michael Cantor	Law Office of Michael Cantor 111 W. Monroe Suite 12000 Phoenix, Arizona 85003 254-4187	10838 N. 35th St. Phoenix, AZ. 85028 996-9522
Mr. Joe Cantadino	Coventry Homes 3875 N. 44th St. St. 201 Phoenix, Arizona 85018 840-8300	
Mr. Kimball J. Corson	Lewis & Roca Lawyers First Interstate Bank Plaza 100 W. Washington St. Phoenix, Arizona 85003 262-5311	Summit Ranch, Box 1358 Phoenix, AZ. 85029
Ms. Peggy DeMarco	SunCor Development Partners 2828 North Central #1212 Phoenix, Arizona 85004 224-0046	
Ms. Nicki Hansen	State Land Department 1624 West Adams Phoenix, Arizona 85007 255-3671	17417 N. 56th Ave. Glendale, AZ. 85308 978-2932

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Ms. Joni Hegel	Sierra National Corp. P.O. Box 15463 Phoenix, Arizona 85060 948-0200	946-2299
Ms. Penny Howe		2902 East Cholla Phoenix, AZ. 85028 992-0713
Mr. Ronald Junck	2002 East Osborn Phoenix, Arizona 85016 955-2446	5305 N. 6th St. Phoenix, AZ. 85012 265-6897
Mr. Lars Lagerman	Evans, Kitchel & Jenckes 2600 N. Central Ave. 20th Pl. Phoenix, Arizona 85004 234-8815	7507 N. 22nd St. Phoenix, AZ. 85020 234-8815
Ms. Tara Laman	Realty Executives P.O. Box 969 Carefree, Arizona 85377 488-9360	5054 E. Calle De Los Arboles Phoenix, AZ. 488-9061
Mr. Guy Leohnis	Wahlers Construction Inc. P.O. Box 19127 Phoenix, Arizona 85005 257-9797	2402 E. Shangri-La Rd. Phoenix, AZ. 85028 971-1778
Mr. G. Noel Lesniak	Motorola 3013 South 52nd Street Tempe, Arizona 85282 438-3249	31042 N. Rancho Moreno Phoenix, AZ. 488-2972
Mr. David Lewis	Luke Land 7801 N. Black Canyon Hyw. Phoenix, Arizona 85021 995-1936	863-1101

NAME	BUSINESS	RESIDENCE
Mr. John F. Long	John F. Long Properties 5035 West Camelback Road Phoenix, Arizona 85032 272-0421	
Mr. Joseph McGarry	Lewis & Roca 100 West Washington Phoenix, Arizona 85003 262-5311	4517 N. Rubicon Ave. Phoenix, AZ. 85018 840-6666
Mr. Bruce McKinney	Glenelyn Corp. 493-0081	815 East Grovers Phoenix, AZ. 85023 992-0183
Mr. Herman Middleton	P.O. Box 1139M Black Canyon Stage 1 Phoenix, Arizona 85029 582-5109	25438 N. 17th Ave. Phoenix, AZ. 85029 582-5109
Mr. Samuel Morse Vice-Chairman	Western Landscape Architects 3509 E. Shea Blvd. Ste 117 Phoenix, Arizona 85028 953-2845	2207 W. Coolbrook Ave. Phoenix, AZ. 85023 992-3400
Mr. Clark Nisbet	Sunstrand Aviation Operations 18008 N. Balck Canyon Hyw. Phoenix, Arizona 85023 439-6400	2906 E. Union Hills Phoenix, AZ. 85024 992-5042
Ms. Mike Pehlman		19042 N. 22nd St. Phoenix, AZ. 85024 992-9794
Ms. Linda Powers		7510 N. 1st St. Phoenix, AZ. 85020 997-1887

NAME	BUSINESS	RESIDENCE
Mr. Robert Smith	DFYS Architects 5110 N. 40th St. Ste.107 Phoenix, Arizona 85018 954-9060	4240 E. Acoma Dr. Phoenix, AZ. 85032 992-3559
Ms. Jan Triplitt	Nail Consultants 621 W. Lone Cactus Drive Phoenix, Arizona 85027 1-582-0258	4726 E. Beverly Ln. Phoenix, AZ. 85032 867-1685
Ms. Linda Verges	Mark V. Financial Com. Group, Ltd 1001 North Central #725 Phoenix, Arizona 85004 941-0533	3209 E. Pershing Phoenix, AZ. 85032 971-0533
Mr. Donald Viehmann	Viehmann, Martin & Associates 2402 East Arizona Biltmore Circle Phoenix, Arizona 85016 957-0660	
Mr. Tom Graham	Vanguard Management 2929 East Camelback Road Ste.250 Phoenix, Arizona 85016	
Mr. Gil Lapainis	Pensus Group 2201 East Camelback Road Ste. 226-B Phoenix, Arizona 85016	
Mr. John Kuhn	Evans, Kuhn & Associates 727 East Bethany Home Road #225 Phoenix, Arizona 85014	
Mr. Frosty Taylor	Paradise Valley Voice 10440 North 32nd Street Ste. #104 Phoenix, Arizona 85028	
Ms. Jerri Robertson	Foothills Sentinel P.O. Box 1569 Cave Creek, Arizona 85331	

NAME	BUSINESS	RESIDENCE
Mr. George Riley	Box 4130, Cave Creek Stage Cave Creek, 85331	
Mr. Jim Bugbee	17239 North 59th Place Scottsdale, Arizona 85254	
Mrs. Jane Rau	8148 East Dale Lane Scottsdale, Arizona 85262 585-4446	
Ms. Clare Gramer	P.O. 1950 Phoenix, Arizona 85001	
Mr. Phillip Ernsberger	730 East Marlitte Avenue Phoenix, Arizona 85014	
Ms. Jane White	15252 Cave Creek Stage Cave Creek, Arizona 85331	
Ms. Karen Butler	Slavin, Kane & Patterson 2198 East Camelback Road Ste. 285 Phoenix, Arizona 85016	
Mr. Fred Pearson	Gruen Associates 3900 East Camelback Road Ste. 611 Phoenix, Arizona 85018	
Ms. Linda Millican	Cranes Homes 7430 East Butherus Ste. B Scottsdale, Arizona 85260	
Mr. Howard Forman	Star Route 2 Box 470 Cave Creek, Arizona 85331	

ATTENDANCE SHEET

NAME	ADDRESS
Jane Rau	8148 E. Dale Lane Scottsdale, Arizona 85262
Herb Chaney	23742 N. 24th St. Phoenix, Arizona 85024
Bill St.Clair	2701 E. Utopia Phoenix, Arizona 85024
Gerald DeHoog	23631 N. 23st Phoenix, Arizona
Roscoe Bowers	3901 E. Pinnacle Peak #226 Phoenix, Arizona 85024
Lillian Moodey	1616 W. Adams Phoenix, Arizona 85007
Dempsey Helms	1616 W. Adams Phoenix, Arizona 85007
Bill Rogers	23029 N. Cave Creek Rd. Phoenix, Arizona 85024
Tim Campbell	2700 N. Central Avenue #1000 Phoenix, Arizona 85004
Dick Perreart	Flood Control District
Amir Motamedi	Flood Control District
Iris & Jim McCoy	3901 E. Pinnacle Peak #320 Phoenix, Arizona 85024
George & Mary Van Diangelo	3901 E. Pinnacle Peak Rd. #341 Phoenix, Arizona 85024

ATTENDANCE SHEET
(Continued)

NAME	ADDRESS
George Knauff	3901 E. Pinnacle Peak Rd #391 Phoenix, Arizona 85024
Audrey Young	3901 E. Pinnacle Peak Rd. #409 Phoenix, Arizona 85024
Pat Waring	3901 E. Pinnacle Peak Rd. #132 Phoenix, Arizona 85024
Jean Shaner	3901 E. Pinnacle Peak Rd. #258 Phoenix, Arizona 85024
Howard & Lyn Bickerdyke	3901 E. Pinnacle Peak Rd. #264 Phoenix, Arizona 85024
Noel Lesniak	31042 N. Rancho Moreno Drive Cave Creek, Arizona 85331 438-3249
Francisco Badilla	City of Phoenix
Paul Kienow	City of Phoenix
Roy E. Peckat	3901 E. Pinnacle Peak Rd. #339 Phoenix, Arizona 85024