ARIA
AT SILVERSTONE

Preliminary Drainage Report

3 engineering Job #: 5315

Original Date: July 15, 2024, REV October 10, 2024

COS #: 15ZN-2005#4, 2-PP-2024

3engineering
planning civil engineering surveying

ARIA AT SILVERSTONE

PRELIMINARY DRAINAGE REPORT

Prepared for:

K Hovnanian Great Western Homes, LLC
8800 E. Raintree Drive, Suite 300
Scottsdale, Arizona 85260
Contact: Chuck Chisholm
Phone: (480) 824-4175

Expires 12/31/2024

Matthew J. Mancini, P.E.

July 15, 2024

REV October, 10, 2024

Submittal to:

City of Scottsdale
7447 E. Indian School Road, Suite 105
Scottsdale, AZ 85251

Prepared by:

3 engineering, L.L.C.
6370 E. Thomas Road, Suite 200
Scottsdale, Arizona 85251
Contact: Matthew J. Mancini, P.E.

Job Number 5315

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Table of Contents

Appendix

1. Introduction

The purpose of this report is to present the existing and proposed drainage plan for the project site, Silverstone Parcel D. It is our opinion that the proposed grading and drainage concept is in accordance with the City of Scottsdale drainage requirements (Ref. 2) and the Master Drainage Report for Silverstone (Ref. 4).

The project site, Silverstone Parcel D, is located in Section 14, Township 14 North, Range 4 East of the Gila and Salt River Meridian, Maricopa County, Arizona within the City of Scottsdale. The project is located North of Williams Drive, West of 74th Street, and East of Scottsdale Road at 22602 N. 74th Street, Scottsdale, Arizona 85255. The site is bounded on the north by an existing apartment complex, on the west by Scottsdale Road and vacant land, on the south by Williams Road and commercial property, and east by 74th Street and an existing senior living facility. See Appendix A for a site map. The site is a proposed 100-lot single-family attached residential subdivision project.

2. Site Description

Existing

The site is currently vacant desert land. The project size is 13.5 acres. The existing topography generally slopes from northeast to southwest at approximately 1.5 percent (1.5 %). The site does not currently retain storm water. The existing berm onsite is a remnant of the old Westworld site, and contains a large passthrough culvert which allows any waters to pass through the berm. Therefore, no water is retained. Based on site reconnaissance, aerial photography (Appendix E), and excerpts from the Final Drainage Report for Mark Taylor San Portales Apartments, Kimley Horn and Associates March 2016 (Ref. 5), the Drainage Report for Williams Drive (Scottsdale Road to Miller), Wood Patel, January 2007 (Ref. 6), and the Rawhide Wash Flood Hazard Mitigation – Conditional Letter of Map Revision, FCD2018C15, Dec 2021, Revised March 2023, Prepared by Flood Control District of Maricopa County, and JE Fuller (Ref. 7) and (Appendix I), storm water enters the site along at the northwest corner from the outfall of the San Portales apartments, as well at the southeast corner where the existing Williams Drive channel lies along the southern boundary of the site. As you can see in Appendix E, upstream (north) of the site is completely development, and therefore the minimal flow shown impacting the site from the north that is shown in Ref. 7 and Appendix I is not realistic. The site is also affected by the floodplain ponding at the outlet of the Rawhide Wash crossing of Scottsdale Road. Per Ref. 7 the floodplain has been revised via a CLOMR application, and shall be used as the best available data. In addition, the site does not show any signs of containing waters of the US (404 washes).

Federal Emergency Management Agency (FEMA) Designation

According to FEMA Flood Insurance Rate Map (FIRM) # 04013C1310M, updated July 20, 2021, along with the updated FIRM in Ref/7's CLOMR, dated February 2023, the site is located within the "Zone X" and "Zone AO1 & AO2" floodplain designations.

"Zone X" is described as follows:

> "Area of Minimal Flood Hazard"

"Zone AO1" is described as follows:

> "River or stream flood hazard areas, and areas with a 1% or greater chance of shallow flooding each year, usually in the form of sheet flow, with an average depth of 1 feet."

"Zone AO2" is described as follows:

> "River or stream flood hazard areas, and areas with a 1% or greater chance of shallow flooding each year, usually in the form of sheet flow, with an average depth of 2 feet."

Refer to the updated Flood Insurance Rate Map information in Appendix B.

Proposed

The Site, Silverstone Parcel D is proposed as a 100-lot residential subdivision, with private streets and gated access from Williams Drive. The project proposes approx. 13 acres of disturbance.

3. ***Drainage Design - Offsite***

The site is currently vacant desert land. The existing topography generally slopes from northeast to southwest at approximately 1.5 percent (1.5 %). To the east and south, 74th Street and Williams Drive, respectively, provide drainage barriers to the site.

Per Ref 4, 5, 6 and 7, there are three areas of offsite storm water impacting the site; at the site's northwest corner, along the northern boundary, and at the site's southeast corner, The apartment complex development directly north of the site on Parcel E retains pre-vs-post storm water volume, and releases it at the subject site's northwest corner. It also has 1-ft deep basins and small landscape areas along the south that overflow through block wall weep holes into the subject site when storms exceed 100-year events. The Miller Road and Williams Drive Improvement Areas impact the site's southeast corner, in which there are already channel improvements in place that convey and release this flow at the site's southwest corner.

Per Ref 5, the outflow rate for Parcel E is 49 CFS. The 1-ft deep basins and small landscape areas along the south of Parcel E capture small amounts of water, and overflow discharges are negligible. Per Ref. 6, The flow rate that discharges at the Site's southeast corner within the Williams Drive Channel is 130 CFS. Based on Ref. 6 this flow increases as the channel traverses west; however, due to existing development, and the development of this site, the flow only increases via the flow coming off Williams Drive via scuppers, and 100-year basin overflow from the Site itself. Referring to Appendix G, this flow increases to 140 cfs once it flows past the easterly entrance to the Site.

Referring to Appendix E, upstream (north) of the site is completely development, and therefore the minimal flow shown impacting the site from the north that is shown in Ref. 7 and Appendix I is not realistic. Therefore, the proposed development will mitigate the overflow at the northwest corner of the site, the flow entering the Williams Drive channel, and any minimal flow in excess of the 100-year storm the retention basins to the north do not store that pass through the weep holes along the wall at the north boundary of the site.

To accommodate the Parcel E overflow, an existing accepting basin was placed with the apartment complex development. This basin is to remain, and discharge flow to Scottsdale Road R/W as it currently does today.

For the 1-ft deep basins and small landscape areas along the south of Parcel E, open space has been provided at the north end of the subject site and is designed to provide relief from minimal runoff that may occur from Parcel E during storm events exceeding 100-year conditions.

The offsite flow from the Miller Road and Williams Drive Improvement Area are proposed to be conveyed within the existing channel along the north side of Williams Drive. Grading and Landscape Improvements to this channel are proposed with this project to improve the efficiency of the channel. For slopes exceeding 4:1, rip rap is proposed on the channel. A pipe culvert system is designed under the site's entrance off of Williams Drive. Refer to Appendix G for the channel and culvert calculations.

Based on the drainage design of this project, the historical drainage patterns, and outfall points are maintained, and downstream properties are not adversely affected by this project.

4. *Drainage Design – Onsite*

The City of Scottsdale Design Standards and Policies Manual and the Drainage Design Manual for Maricopa County, Volume 1 was followed in designing on-site drainage facilities for the site. The following standards shall be met as part of this project:

- 10-year peak discharges shall be contained below the top of curb elevations.
- 100-year peak discharges shall be contained within the private street tract.
- Sump condition catch basins and storm drain shall be designed for the 10-year storm event with 100-year overflowing the sump.
- Flow-By condition catch basins storm drain shall be designed for the 100-year storm event.
- Channels and channel culverts shall be designed for the 100-year event.
- Due to this site being a previously developed parcel, the retention of the site shall following criteria; Pre-Vs-Post Runoff or First Flush (first 0.5" of runoff) whichever is greater.
- Retention basins shall drain within 36-hour. 0.1 cfs shall be used as a drywell design rate. (post construction percolation tests shall be used to determine higher rates)
- Drainage shall enter and exit in a similar and/or historical manner as existing conditions.
- PADs within the Zone AO flood zone shall be elevated a minimum of 1-ft above the base flood elevation. Refer to Appendix G for Base Flood Elevation and PAD elevations.

Refer to the Drainage Maps in Appendix F for the following discussion:

On-site drainage areas will be conveyed via surface drainage from the lots to the private accessways' curb and gutter for flow draining to the front of the lots, and directly into retention basins for flow draining to the rear of lots. Storm water exiting the lots in the front flows and into the curb and gutter flows into storm drain systems and then into the surface retention basins and underground tanks.

Retention for Scottsdale Road, Williams Drive and 74th Street are not provided/required based on the Master Drainage Report, and the previous infrastructure in place. Site peak flows shall be calculated using the Rational Method, as established in Ref. 1. The calculations determine the amount of flow generated on-site and directly to the catch basins. Drainage areas were determined based on preliminary grading design, and are shown on the Drainage Map in Appendix F.

At the time of final design, StormCAD shall be used to design storm drain sizes. Weir Calculations were used to determined catch basin sizes. Refer to Appendix G for Weir Calculations.

Because the Site was previously developed, the retention of the site shall adhere to the following criteria; Pre-Vs-Post Runoff or First Flush (first 0.5" of runoff) whichever is greater. Based on Ref. 2. the Site's 100-year runoff coefficient for the site is 0.94 and the existing coefficient is 0.45. Based on NOAA14, the site's precipitation value is 2.38 inches. Based on the calculations, pre-vs-post is greater than first flush. Full 100-year volumes were also calculated, as these are utilized to determine the 100-year overflows for each of the basins. For required and proposed retention volume calcs, refer to Appendix G .

All basins, due to being pre-vs-post retention based, are designed to discharge 100 year storm water flows. The following are descriptions of each basin's overflow:

- Basin A – Basin A will fill up and excess flow will over top via a rip-rap weir at the site's ultimate outfall at the southwest corner of the site and into Scottsdale Road.
- Basin B – Basin B retains the 100-year runoff volume. Any overflow in excess of the 100-year will cascade to Basin D.
- Basin C - Basin C retains the 100-year runoff volume. Any overflow in excess of the 100-year will over top via a rip-rap weir at the site's NW corner and into Scottsdale Road.

- Basin D - Basin D will fill up and 100-year discharge will outlet to an overflow strom drain system that drains to Basin A, and will over top via a rip-rap weir at the site's ultimate outfall at the southwest corner of the site and into Scottsdale Road.
- Basin E – Basin E will fill up and excess flow will over top via a rip-rap weir into the Williams Drive channel.

Refer to Appendix G for Overflow Calculations.

The surface basins shall drain via basin infiltration and use of drywells. A drywell rate of 0.1 cfs is used for the purposes of this design report; however, Geotechnical percolation tests shall be completed after construction of the basins to determine if the drywell systems can be reduced or eliminated. Refer to Appendix G for percolation calculations.

Drainage easements shall be dedicated over the tracts with retention areas and areas of offsite flow with Q's above 25 cfs. In addition, the private street (Tract A), shall have a drainage easement as part of its use. This will ensure that the basins and storm drain systems can be maintained in order to perform properly during storm events. There is currently a temporary drainage easement on the site, which will be abandoned as a part of this project

For the purpose of design, finished floors for the project have been placed a minimum of 12-inches above lot outfalls, and a minimum of 18-inches above ultimate outfalls at the southwest corner of the site. For lots within the Zone AO flood zone, the pads are designed a minimum of 1-ft above the base flood elevation. Refer to Appendix B for the FIRM, and Appendix G for the base flood elevation and minimum PAD calculations. The proposed project disturbs over 1.0 acre and therefore a SWPP Plan, NOI and Authorization to Discharge Letter will be required from ADEQ.

5. ***Conclusions***

The following is a summary of the Scottsdale Heights Phase 2 Drainage Report.

- The site currently lies within “Zone X” and “Zone AO” floodplain designations.
- PADs within the Zone AO zone are elevated a minimum 1-ft above the base flood elevation.
- Retention is provided for Pre-vs-Post storm event.
- Retention shall dissipate within 36 hours via drywells.
- Offsite drainage is accepted and discharged in its historical locations.
- Finished floors are set a minimum of 12-inches above lot outfalls, and 18-inches above ultimate outfalls.

6. ***References***

1. Maricopa County, *Drainage Design Manual, Volume I, Hydrology*, Flood Control District of Maricopa County.

2. City of Scottsdale, *Design Standards and Policies Manual,* 2018.

3. Maricopa County Drainage Design Manual, Hydraulics, Flood Control District of Maricopa County, 2013.

4. Master Drainage Report for Silverstone, Wood Patel, Phoenix Arizona, March 2007, and Addendums 2 & 3 to Master Drainage Report for Silverstone, Kimley Horn and Associates, Inc. April 2015, and December 2015, respectively.

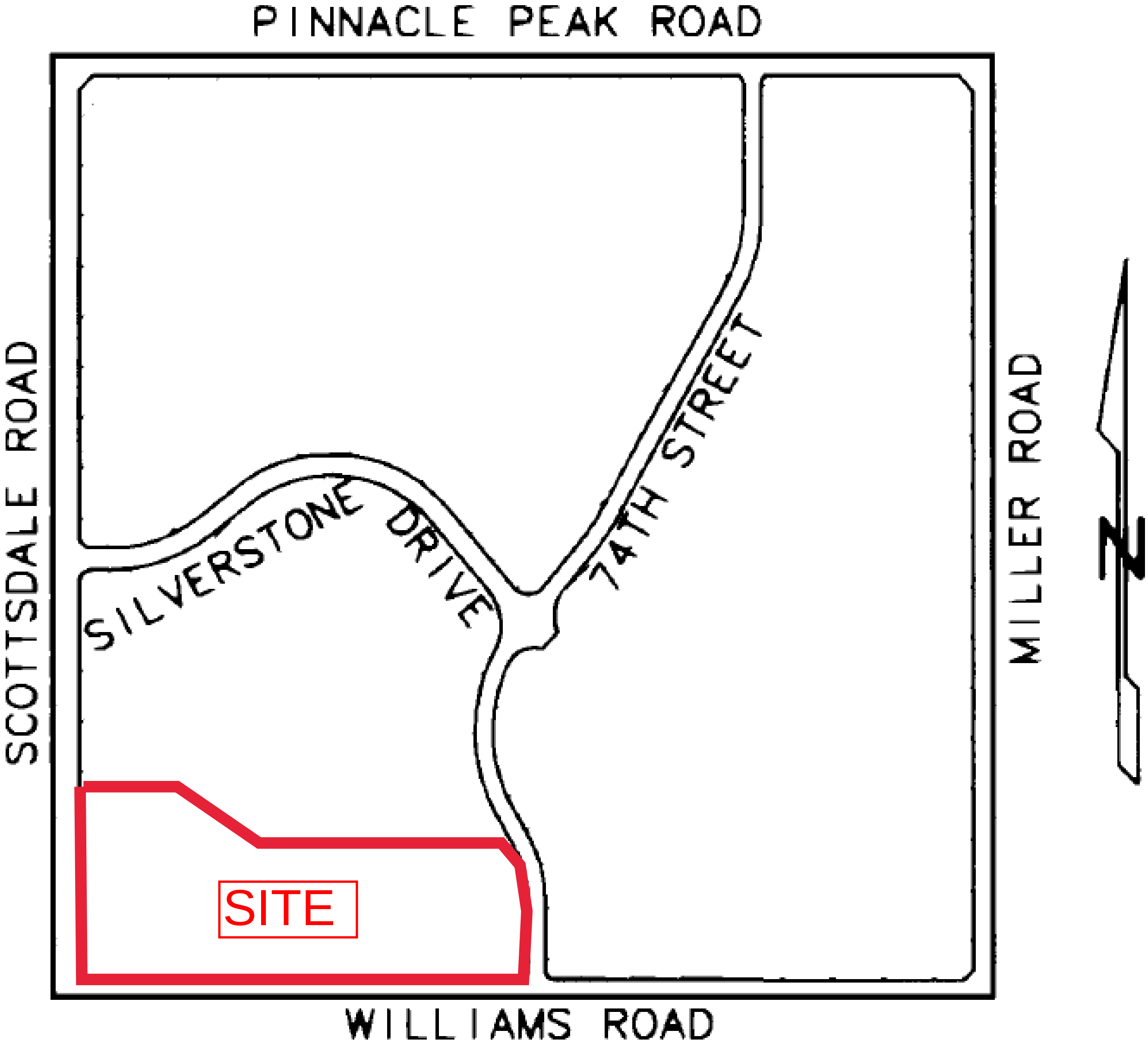
5. Final Drainage Report for Mark Taylor San Portales Apartments, Kimley Horn and Associates March 2016.

6. Drainage Report for Williams Drive (Scottsdale Road to Miller), Wood Patel, January 2007.

7. Rawhide Wash Flood Hazard Mitigation – Conditional Letter of Map Revision, FCD2018C15, Dec 2021, Revised Marc 2023, Prepared by Flood Control District of Maricopa County, and JE Fuller

APPENDIX A

Vicinity Map



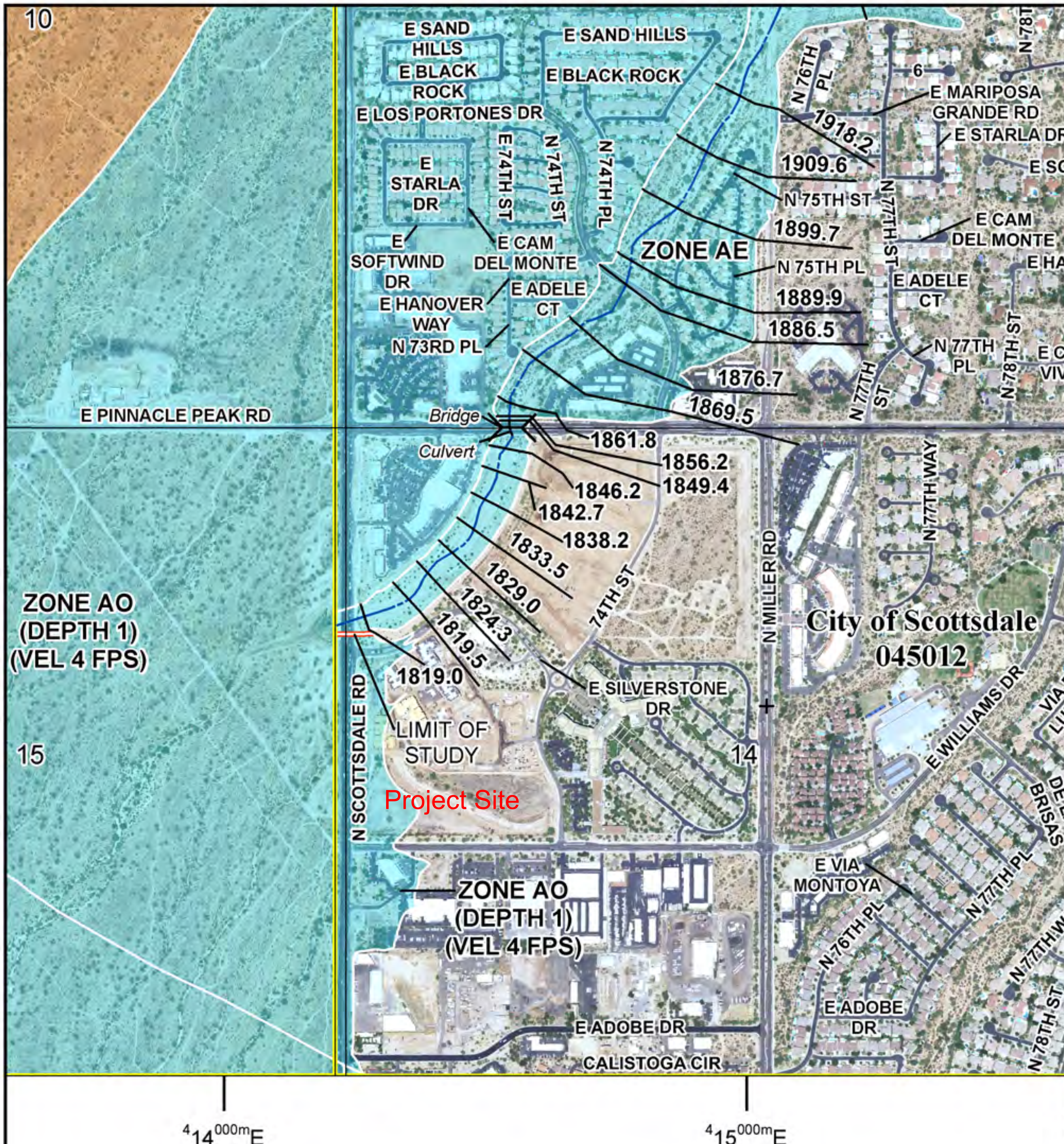
SECTION 14
T.4N, R.4.E

VICINITY MAP

N.T.S.

APPENDIX B

FEMA FIRM



National Flood Insurance Program

NATIONAL FLOOD INSURA

FLOOD INSURANCE RATE

MARICOPA COUNTY, ARIZONA

And Incorporated Areas

PANEL **1310** OF **4425**

FEMA

Panel Contains:

COMMUNITY	NUMBER	PANEL	SUFFIX
PHOENIX, CITY OF	040051	1310	M
SCOTTSDALE, CITY OF	045012	1310	M

VERSION NUMBER
2.3.3.4

MAP NUMBER
04013C1310M

MAP REVISED
JULY 20, 2021

This is an official FIRMette showing a portion of the above-referenced flood map created from the MSC FIRMette Web tool. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For additional information about how to make sure the map is current, please see the Flood Hazard Mapping Updates Overview Fact Sheet available on the FEMA Flood Map Service Center home page at https://msc.fema.gov.

Floodplain and Elevation Certificate Map

Elevation Certificate
Local
Arterial

Highway

Interstate Highway
State\US\Other Highway
Interchange\Ramp
Parcel

Floodplain (Pending FEMA Approval)

Floodway
100-Year Flood Zone

Floodplain (FEMA Effective)

Floodway
100-Year Flood Zone

N 1:4,514 1 inch = 148 feet

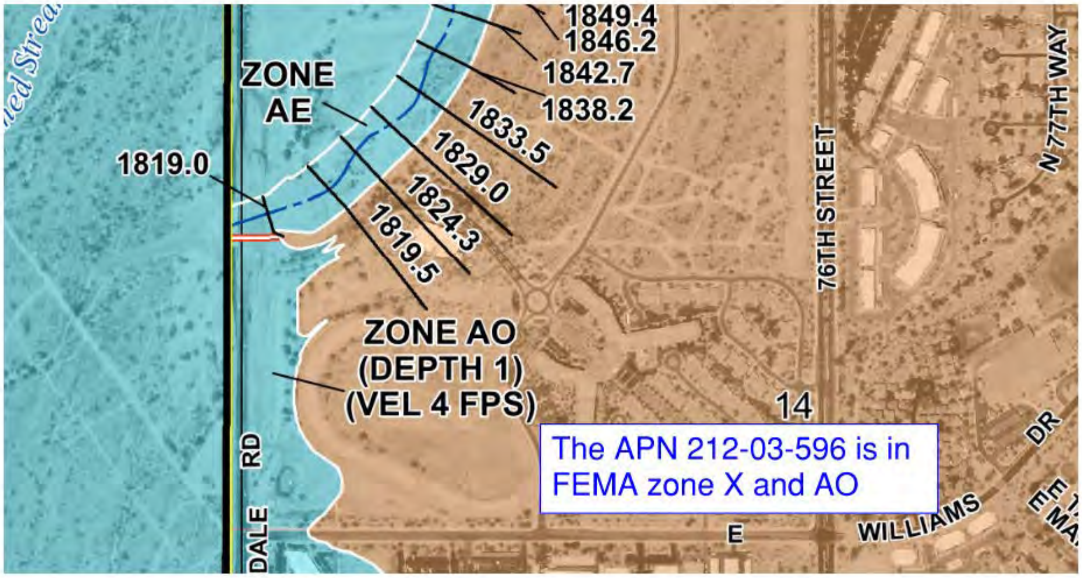
0 0 0 0 mi

MARICOPA COUNTY

Flood Control District of Maricopa County
2801 W Durango St (602) 506-2419
Phoenix, AZ 85009 http://www.fcd.maricopa.gov

Unofficial Document
This document cannot be used for floodplain determinations. Current studies, erosion setbacks and other factors may also affect the floodplain status of the property. The information shown for pending floodplains are the best technical information available at this time to determine the 1% chance flood and are subject to change.

From:	Stormwater Group
To:	Dan Castro
Subject:	RE: (Request for Flood Hazard Determination Letter) 5315
Date:	Tuesday, March 12, 2024 2:20:20 PM
Attachments:	image002.png



Flood Insurance Rate Map (FIRM) Information

Community Number	Panel # Panel Date	Suffix	Date of FIRM (Index Date)	FIRM Zone	Base FLOOD Elevation (In AO Zone Use Depth)
045012	1310 7/20/2021	M	7/20/2021	X, AO	depth=1' velocity=4ft/s

From: Dan Castro <danc@3engineering.com>
Sent: Thursday, March 7, 2024 12:14 PM
To: Stormwater Group <stormwatergroup@scottsdaleaz.gov>
Subject: (Request for Flood Hazard Determination Letter) 5315

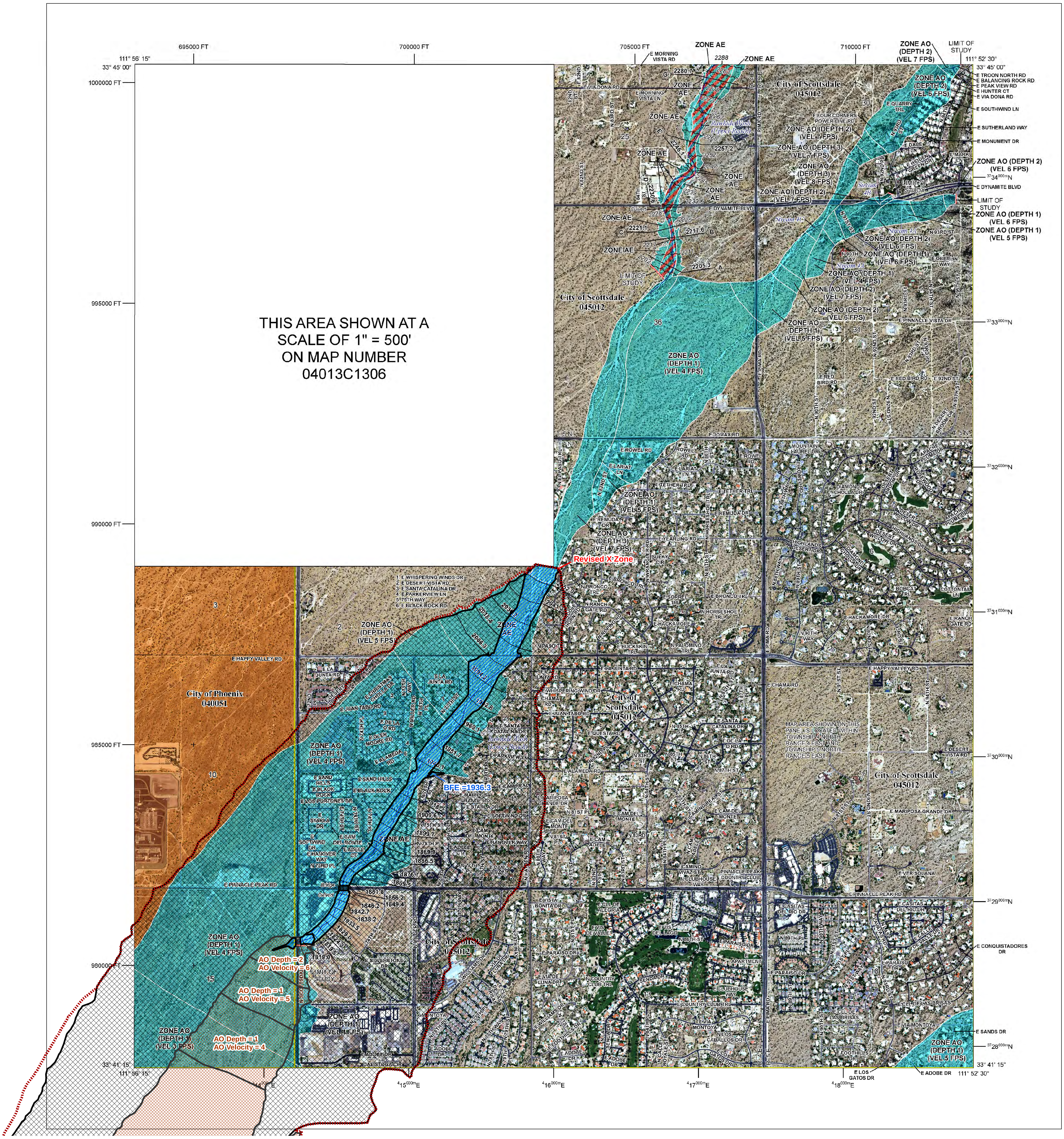
⚠ **External Email: Please use caution if opening links or attachments!**

Per direction from email response below, I am requesting a Flood Hazard Determination letter for parcel **212-03-596** located at the northeast corner of Scottsdale Road and Williams Drive. Please let me know if you need any other information.

Sincerely,

Dan Castro | *Director of Planning*
3engineering
6370 E. Thomas Rd., Suite # 200 | Scottsdale, AZ 85251
O: (602) 334-4387 | C: (520) 307-7065 | F: (602) 490-3230
danc@3engineering.com | www.3engineering.com

From: DoNotReply@scottsdaleaz.gov <DoNotReply@scottsdaleaz.gov>
Sent: Thursday, March 07, 2024 11:47 AM
To: Dan Castro <danc@3engineering.com>
Subject: PRR# 24-004302



FLOOD HAZARD INFORMATION

SEE FIS REPORT FOR ZONE DESCRIPTIONS AND INDEX MAP

THE INFORMATION DEPICTED ON THIS MAP AND SUPPORTING DOCUMENTATION ARE ALSO AVAILABLE IN DIGITAL FORMAT AT

HTTPS://MSC.FEMA.GOV

SPECIAL FLOOD HAZARD AREAS	Without Base Flood Elevation (BFE) *Zone A,V, A99*
	With BFE or Depth *Zone AE, AO, AH, VE, AR*
	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee See Notes. *Zone X*
	Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN Area of Minimal Flood Hazard *Zone X*
	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
OTHER FEATURES	E 18.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
	8 Coastal Transect
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
	513 Base Flood Elevation Line (BFE)
	Limit of Study
	Jurisdiction Boundary

NOTES TO USERS

For information and questions about this Flood Insurance Rate Map (FIRM), available products associated with this FIRM, including historic versions, the current map date for each FIRM panel, how to order products, or the National Flood Insurance Program (NFIP) in general, please call the FEMA Map Information eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Flood Map Service Center website at https://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website.

Communities annexing land on adjacent FIRM panels must obtain a current copy of the adjacent panel as well as the current FIRM Index. These may be ordered directly from the Flood Map Service Center at the number listed above.

For community and countywide map dates refer to the Flood Insurance Study Report for this jurisdiction.

To determine if flood insurance is available in this community, contact your Insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

Base map information shown on this FIRM was provided in digital format by the United States Geological Survey (USGS). This information was derived from digital orthophotography at a 5-meter resolution from photography dated 2017.

LEGEND FOR

Limit of Revision

Revised Riverine Floodzones

AE

Revised Alluvial Fan Floodzones

AO with Velocity

Added X Floodzone (Due to Levee)

X, AREA WITH REDUCED FLOOD RISK DUE TO LEVEE

Revised X Floodzone

X, 1 PCT DEPTH LESS THAN 1 FOOT

SCALE

Map Projection:
NAD 1983 StatePlane Arizona Central FIPS 0202 Feet;
Western Hemisphere: Vertical Datum: NAVD 88

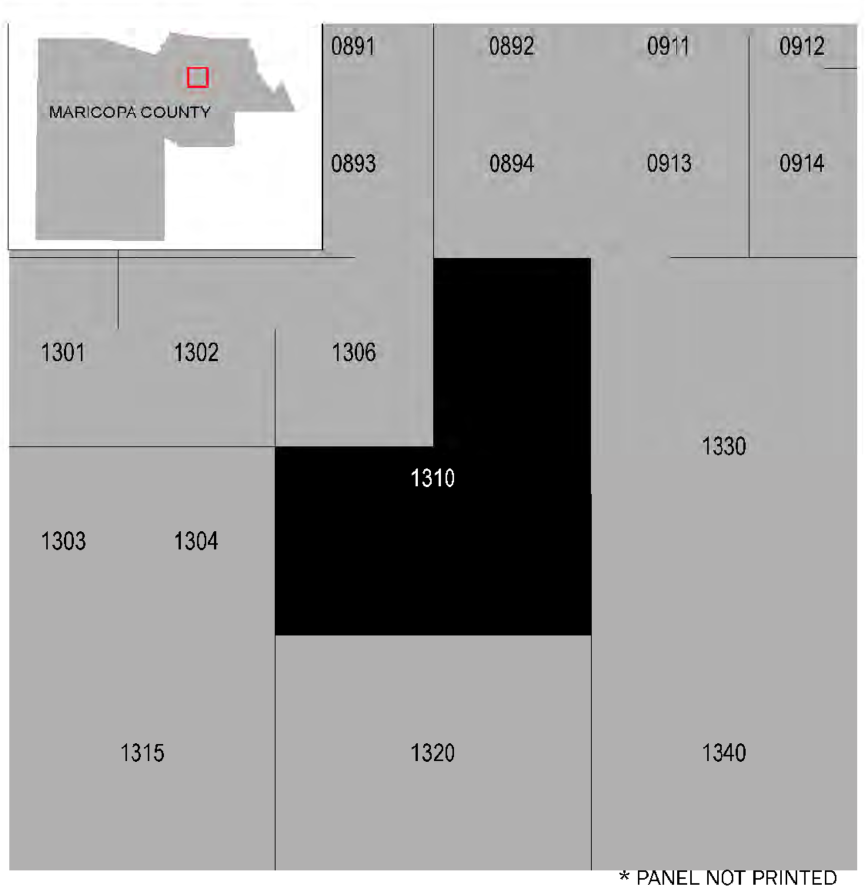
N

1 inch = 1,000 feet 1:12,000

0 1,000 2,000 4,000 Feet

0 250 500 1,000 Meters

PANEL LOCATOR



* PANEL NOT PRINTED

FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

MARICOPA COUNTY, ARIZONA

And Incorporated Areas

PANEL **1310** OF **4425**

FEMA

Panel Contains:

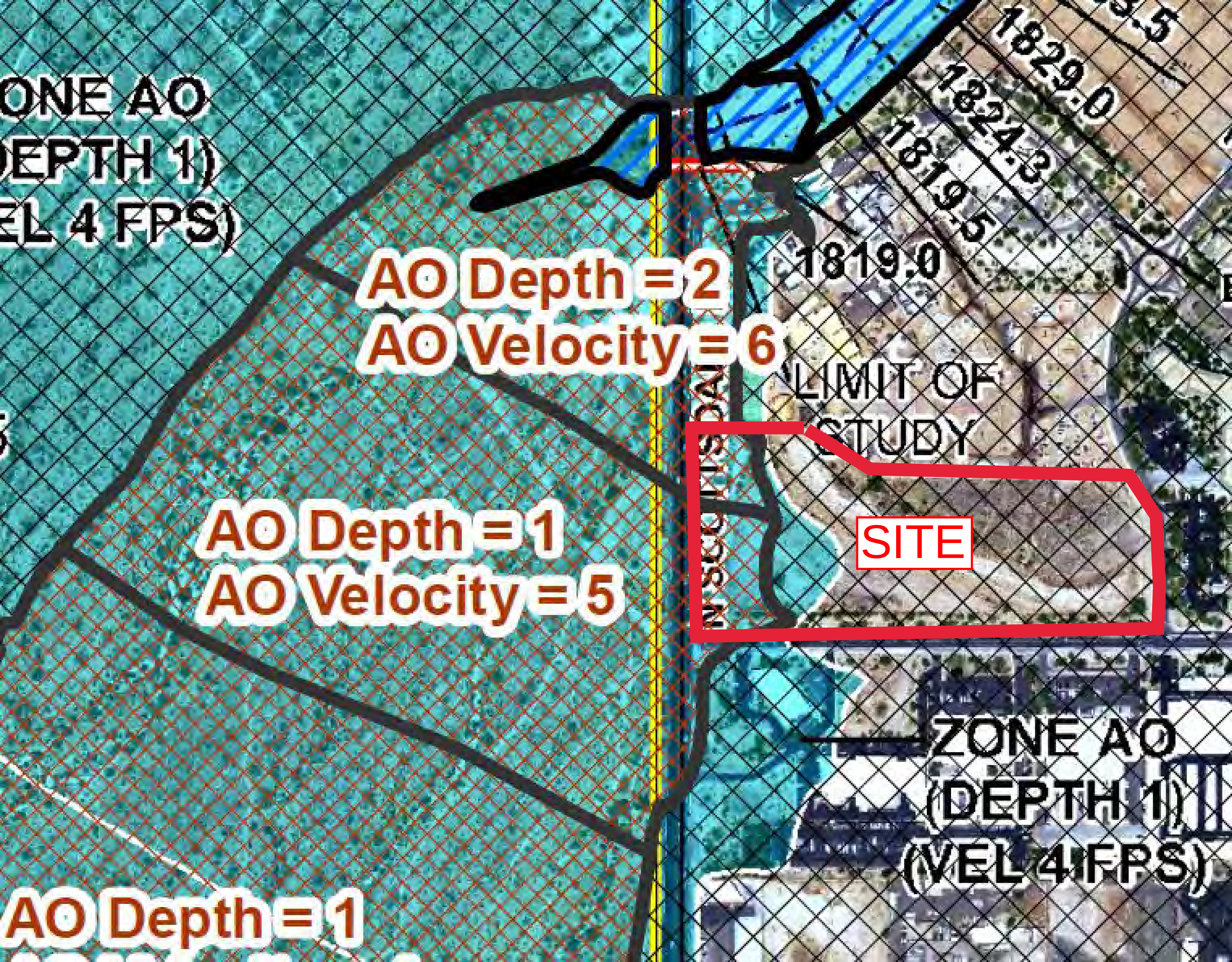
COMMUNITY	NUMBER	PANEL	SUFFIX
PHOENIX, CITY OF	040051	1310	M
SCOTTSDALE, CITY OF	045012	1310	M

ANNOTATED: February 2023

VERSION NUMBER
2.3.3.4

MAP NUMBER
04013C1310M

MAP REVISED
JULY 20, 2021


1829.0
1824.3
1819.5
1819.0
AO Depth = 2
AO Velocity = 6
LIMIT OF STUDY
SITE
AO Depth = 1
AO Velocity = 5
ZONE AO
(DEPTH 1)
(VEL 4 FPS)
AO Depth = 1

APPENDIX C

Warning and Disclaimer of Liability

GRADING & DRAINAGE LANGUAGE

WARNING AND DISCLAIMER OF LIABILITY

The City's Stormwater and Floodplain Management Ordinance is intended to minimize the occurrence of losses, hazards and conditions adversely affecting the public health, safety and general welfare which might result from flooding.

The Stormwater and Floodplain Management Ordinance identifies floodplains, floodways, flood fringes and special flood hazard areas. However, a property outside these areas could be inundated by floods. Also, much of the city is a dynamic flood area; floodways, floodplains, flood fringes and special flood hazard areas may shift from one location to another, over time, due to natural processes.

WARNING AND DISCLAIMER OF LIABILITY

The flood protection provided by the Stormwater and Floodplain Management Ordinance is considered reasonable for regulatory purposes and is based on scientific and engineering considerations. Floods larger than the base flood can and will occur on rare occasions. Floodwater heights may be increased by constructed or natural causes. The Stormwater and Floodplain Management Ordinance does not create liability on the part of the city, any officer or employee thereof, or the federal, state or county government for any flood damages that result from reliance on the Ordinance or any administrative decision lawfully made thereunder.

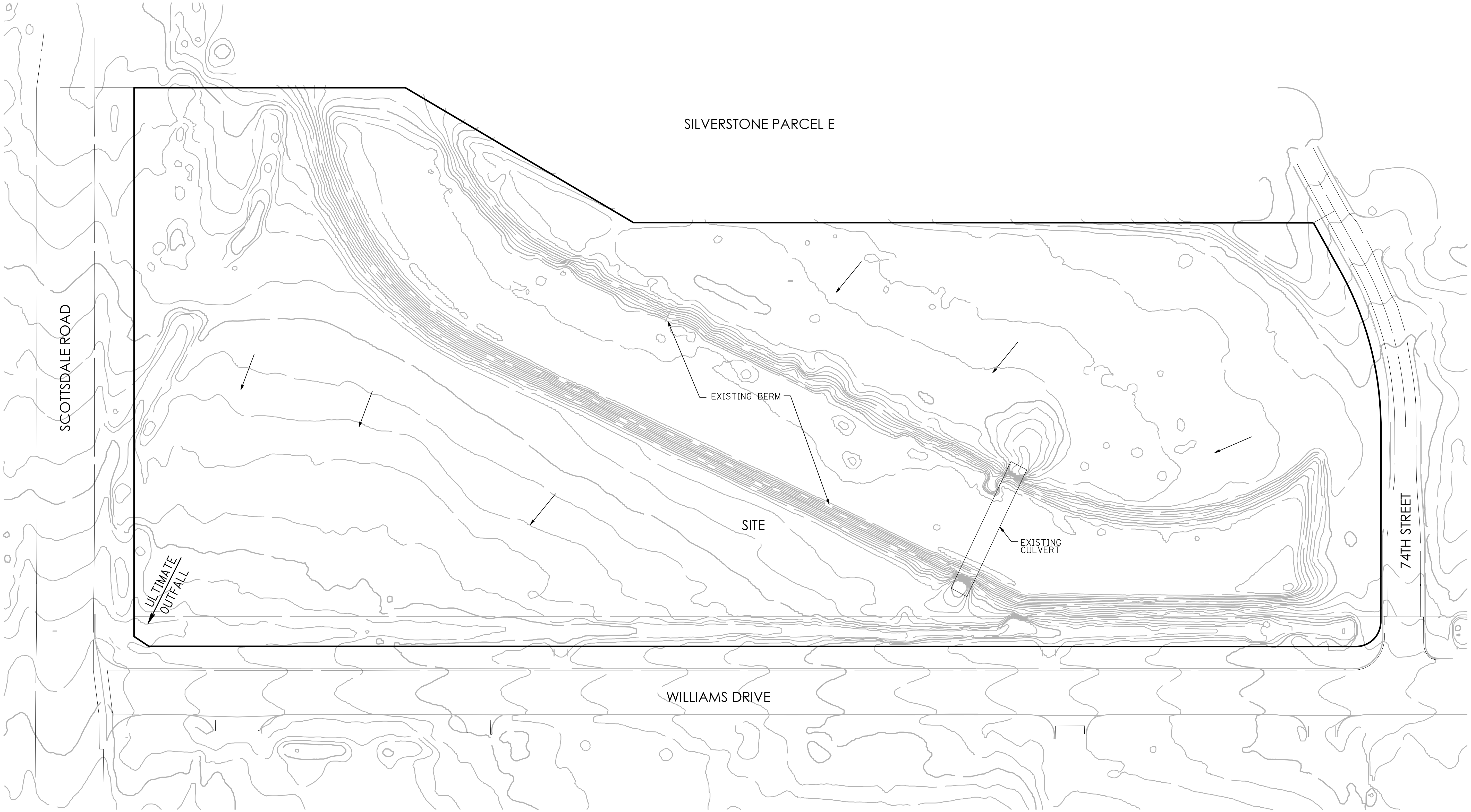
Compliance with the Stormwater and Floodplain Management Ordinance does not ensure complete protection from flooding. Flood-related problems such as natural erosion, streambed meander, or constructed obstructions and diversions may occur and have an adverse effect in the event of a flood. You are advised to consult your own engineer or other expert regarding these considerations.

I have read and understand the above.

15-ZN-2005#4		3/27/24
Plan Check #	Owner	Date

APPENDIX D

Topographic Map of Onsite Conditions



SCALE: 1"=100'

0' 50' 100' 200'

SILVERSTONE PARCEL D
SCOTTSDALE, ARIZONA

TOPOGRAPHIC MAP

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planning civil engineering surveying
3 ENGINEERING, LLC.
6370 E. THOMAS ROAD, SUITE # 200 - SCOTTSDALE, ARIZONA 85251
PHONE: (602) 334-4387 - FAX: (602) 490-3230
WWW.3ENGINEERING.COM

DATE: 3/27/24

PROJECT NO. 5315

APPENDIX E

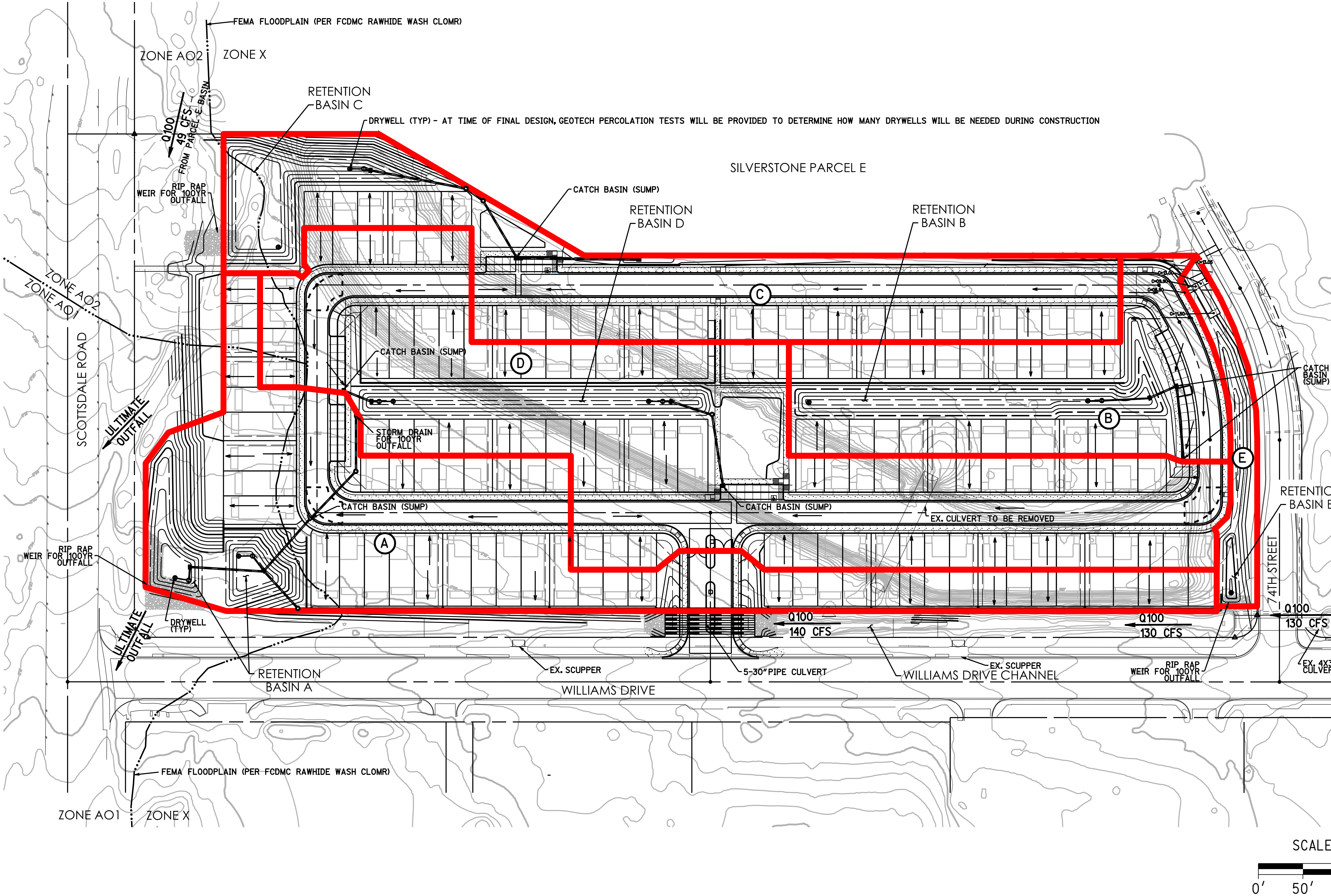
Aerial Photographs of Site

AERIAL - OVERALL SILVERSTONE
TOWNHOMES
DUPLEXES
RAWHIDE WASH
DUPLEXES
TOWNHOMES
APARTMENTS
SITE


AERIAL PHOTO OF SITE
SITE
PARCEL D
212-03-596
SILVERSTONE AT PINNACLE PEAK MOD
MCR: 883-17
PARCEL D
SILVERSTONE AT PINNACLE PEAK MOD
MCR: 883-17
N 74TH ST
N 74TH ST
E WILLIAMS DR
215-04-715
330'+/-

APPENDIX F

Drainage Maps



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DATE: 7/15/24

SILVERSTONE PARCEL D
SCOTTSDALE, ARIZONA

DRAINAGE MAP

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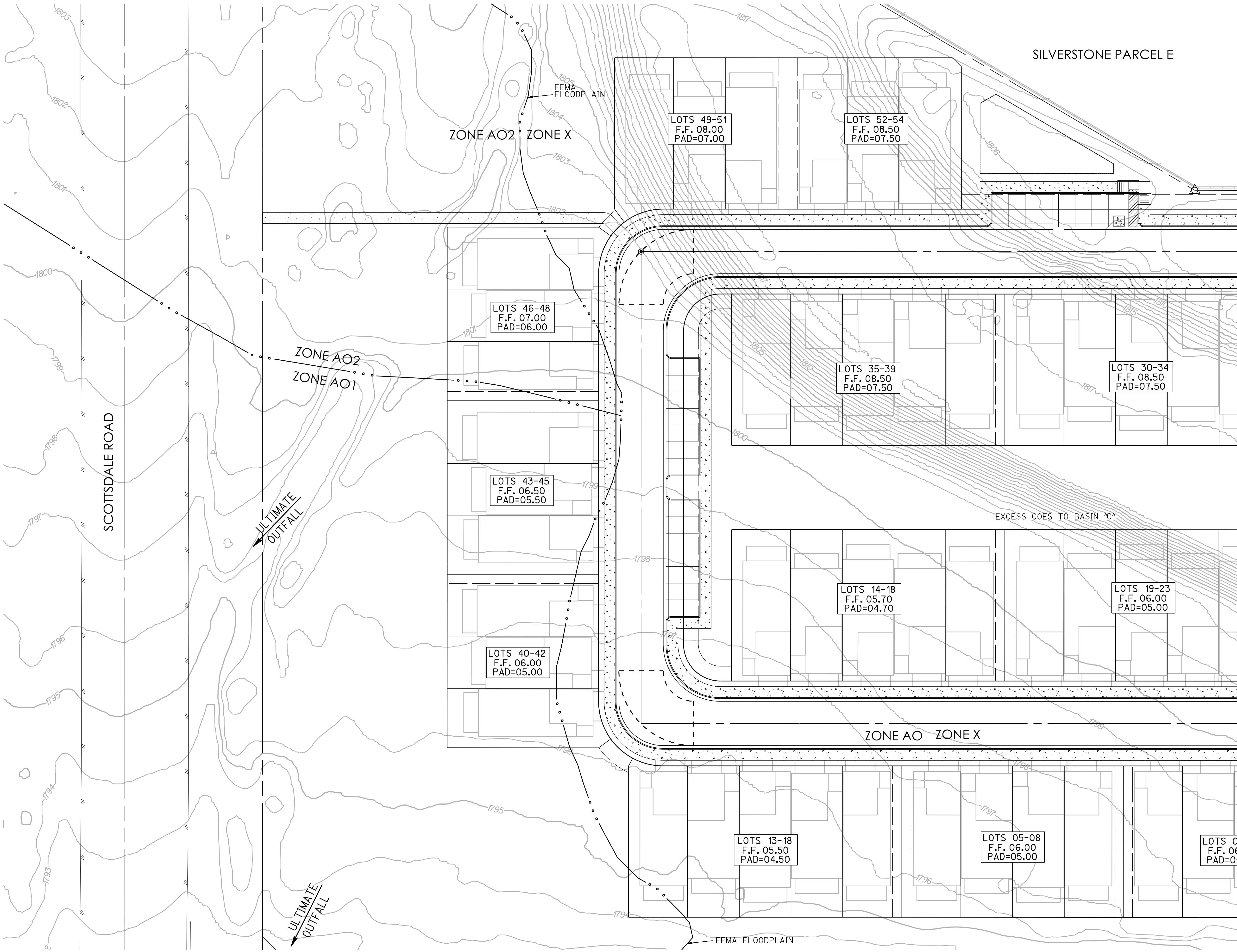

SCALE: 1"=50'

0' 25' 50' 100'

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planning civil engineering surveying

3 ENGINEERING, LLC.
6370 E. THOMAS ROAD, SUITE # 200 - SCOTTSDALE, ARIZONA 85251
PHONE: (602) 334-4387 - FAX: (602) 490-3230
WWW.3ENGINEERING.COM

DATE: 7/15/24

PROJECT NO. 5315

SILVERSTONE PARCEL D
SCOTTSDALE, ARIZONA

FEMA FLOODPLAIN vs FF MAP

C:\My SecuriSync\Projects_5000\5315_Silverstone Parcel D\Reports\Drainage Report - Preliminary\5315_DR MAP_FFs.dgn

APPENDIX G

Drainage Calculations

NOAA Atlas 14, Volume 1, Version 5
Location name: Scottsdale, Arizona, USA*
Latitude: 33.6923°, Longitude: -111.9235°
Elevation: m/ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.205 (0.171-0.252)	0.268 (0.224-0.329)	0.362 (0.300-0.442)	0.434 (0.357-0.528)	0.529 (0.429-0.642)	0.603 (0.482-0.726)	0.677 (0.533-0.814)	0.752 (0.582-0.904)	0.852 (0.643-1.03)	0.929 (0.687-1.12)
10-min	0.312 (0.259-0.383)	0.408 (0.341-0.500)	0.550 (0.456-0.673)	0.660 (0.543-0.803)	0.805 (0.652-0.977)	0.917 (0.734-1.10)	1.03 (0.811-1.24)	1.14 (0.886-1.38)	1.30 (0.979-1.56)	1.41 (1.05-1.71)
15-min	0.387 (0.322-0.475)	0.505 (0.423-0.620)	0.682 (0.565-0.834)	0.818 (0.673-0.996)	0.998 (0.809-1.21)	1.14 (0.910-1.37)	1.28 (1.00-1.54)	1.42 (1.10-1.70)	1.61 (1.21-1.94)	1.75 (1.30-2.12)
30-min	0.521 (0.433-0.639)	0.681 (0.570-0.835)	0.919 (0.761-1.12)	1.10 (0.906-1.34)	1.34 (1.09-1.63)	1.53 (1.22-1.84)	1.72 (1.35-2.07)	1.91 (1.48-2.30)	2.17 (1.64-2.60)	2.36 (1.75-2.85)
60-min	0.645 (0.536-0.791)	0.842 (0.705-1.03)	1.14 (0.941-1.39)	1.36 (1.12-1.66)	1.66 (1.35-2.02)	1.89 (1.52-2.28)	2.13 (1.68-2.56)	2.36 (1.83-2.84)	2.68 (2.02-3.22)	2.92 (2.16-3.52)
2-hr	0.750 (0.632-0.901)	0.970 (0.818-1.17)	1.29 (1.08-1.55)	1.54 (1.28-1.84)	1.87 (1.54-2.23)	2.12 (1.72-2.52)	2.38 (1.90-2.83)	2.64 (2.08-3.13)	3.00 (2.30-3.55)	3.27 (2.45-3.89)
3-hr	0.817 (0.687-0.998)	1.05 (0.883-1.28)	1.37 (1.15-1.67)	1.62 (1.35-1.97)	1.98 (1.62-2.38)	2.26 (1.82-2.71)	2.55 (2.02-3.06)	2.86 (2.23-3.42)	3.28 (2.48-3.92)	3.61 (2.67-4.33)
6-hr	0.979 (0.843-1.16)	1.24 (1.07-1.47)	1.58 (1.35-1.86)	1.85 (1.57-2.17)	2.22 (1.86-2.59)	2.51 (2.07-2.92)	2.81 (2.28-3.27)	3.12 (2.48-3.63)	3.53 (2.74-4.11)	3.86 (2.93-4.50)
12-hr	1.12 (0.972-1.31)	1.41 (1.22-1.66)	1.78 (1.54-2.08)	2.07 (1.78-2.41)	2.47 (2.09-2.86)	2.77 (2.32-3.20)	3.08 (2.54-3.56)	3.39 (2.76-3.92)	3.81 (3.03-4.43)	4.13 (3.22-4.83)
24-hr	1.32 (1.16-1.53)	1.68 (1.47-1.94)	2.18 (1.90-2.51)	2.57 (2.24-2.97)	3.13 (2.70-3.60)	3.57 (3.04-4.11)	4.04 (3.40-4.66)	4.52 (3.77-5.23)	5.21 (4.25-6.04)	5.75 (4.63-6.72)
2-day	1.45 (1.26-1.67)	1.85 (1.61-2.13)	2.43 (2.11-2.79)	2.89 (2.50-3.32)	3.54 (3.04-4.06)	4.06 (3.45-4.66)	4.60 (3.87-5.31)	5.18 (4.31-6.00)	5.98 (4.89-6.96)	6.62 (5.33-7.77)
3-day	1.55 (1.36-1.78)	1.98 (1.74-2.28)	2.62 (2.29-3.00)	3.14 (2.73-3.58)	3.87 (3.34-4.42)	4.46 (3.82-5.10)	5.09 (4.32-5.84)	5.76 (4.83-6.64)	6.72 (5.53-7.78)	7.49 (6.07-8.75)
4-day	1.65 (1.46-1.89)	2.12 (1.87-2.42)	2.82 (2.48-3.20)	3.38 (2.96-3.84)	4.20 (3.65-4.77)	4.87 (4.19-5.53)	5.58 (4.76-6.38)	6.35 (5.35-7.29)	7.46 (6.17-8.61)	8.36 (6.81-9.73)
7-day	1.88 (1.65-2.16)	2.40 (2.11-2.75)	3.20 (2.80-3.66)	3.85 (3.36-4.40)	4.79 (4.14-5.47)	5.56 (4.76-6.36)	6.39 (5.42-7.33)	7.28 (6.10-8.41)	8.56 (7.04-9.95)	9.61 (7.78-11.3)
10-day	2.05 (1.80-2.34)	2.63 (2.31-3.00)	3.49 (3.06-3.98)	4.19 (3.65-4.77)	5.19 (4.49-5.90)	6.00 (5.15-6.84)	6.88 (5.84-7.87)	7.82 (6.56-8.99)	9.15 (7.55-10.6)	10.2 (8.32-12.0)
20-day	2.56 (2.25-2.91)	3.29 (2.90-3.75)	4.36 (3.83-4.95)	5.18 (4.54-5.88)	6.31 (5.49-7.16)	7.19 (6.22-8.18)	8.11 (6.96-9.26)	9.05 (7.70-10.4)	10.3 (8.68-12.0)	11.4 (9.43-13.3)
30-day	3.01 (2.65-3.43)	3.88 (3.42-4.41)	5.14 (4.51-5.82)	6.10 (5.34-6.90)	7.41 (6.44-8.39)	8.43 (7.29-9.55)	9.48 (8.14-10.8)	10.6 (9.00-12.0)	12.0 (10.1-13.8)	13.2 (11.0-15.3)
45-day	3.54 (3.13-4.02)	4.57 (4.04-5.18)	6.04 (5.33-6.84)	7.16 (6.29-8.10)	8.65 (7.56-9.80)	9.80 (8.50-11.1)	11.0 (9.46-12.5)	12.2 (10.4-13.9)	13.8 (11.7-16.0)	15.1 (12.6-17.5)
60-day	3.95 (3.49-4.46)	5.10 (4.52-5.76)	6.73 (5.94-7.59)	7.93 (6.98-8.95)	9.52 (8.34-10.8)	10.7 (9.34-12.1)	11.9 (10.3-13.6)	13.2 (11.3-15.0)	14.8 (12.6-17.1)	16.1 (13.5-18.6)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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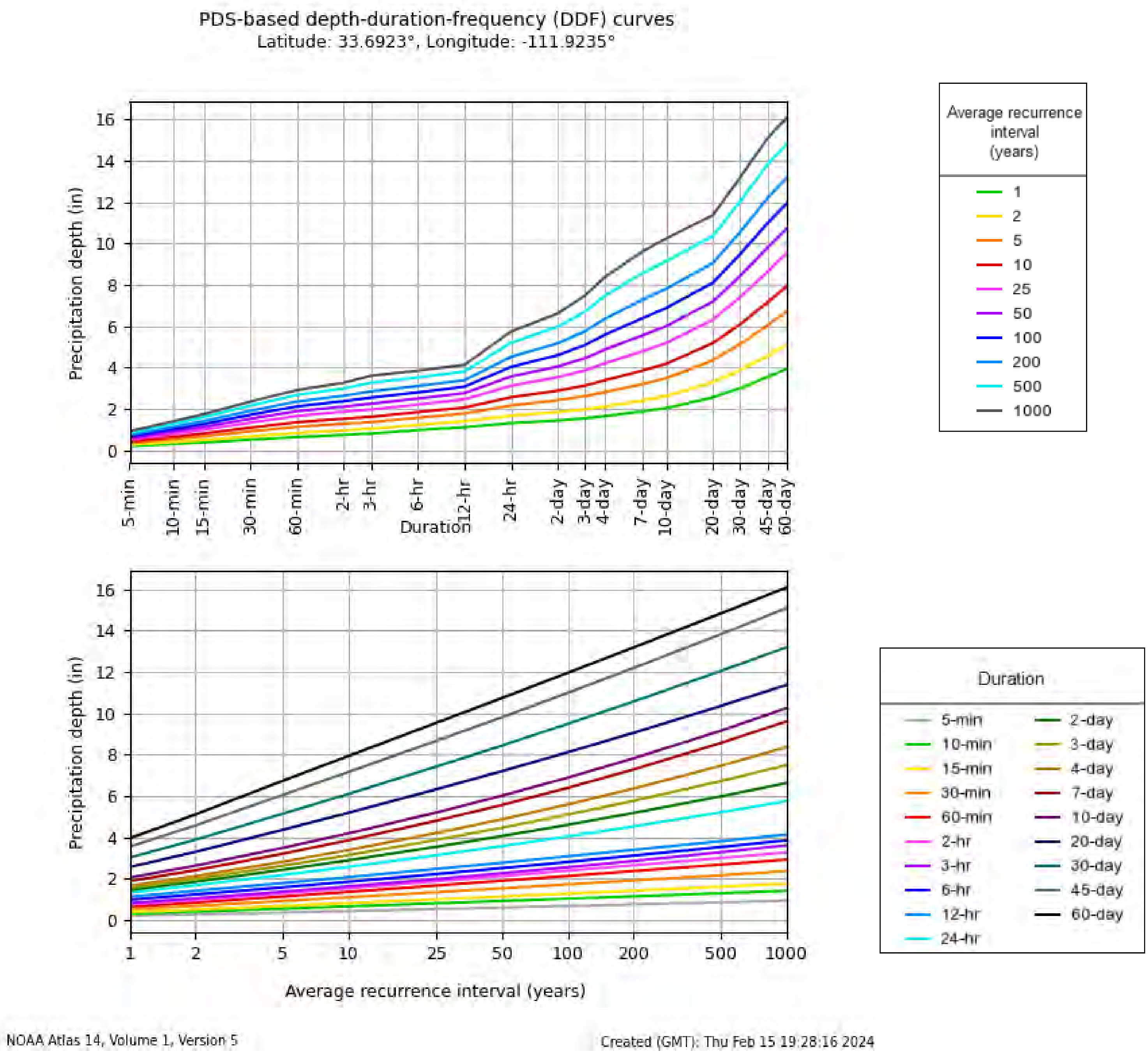
PF graphical



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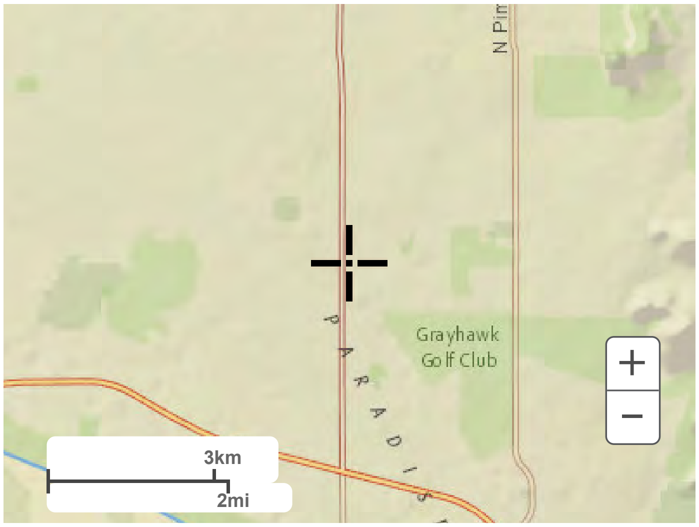
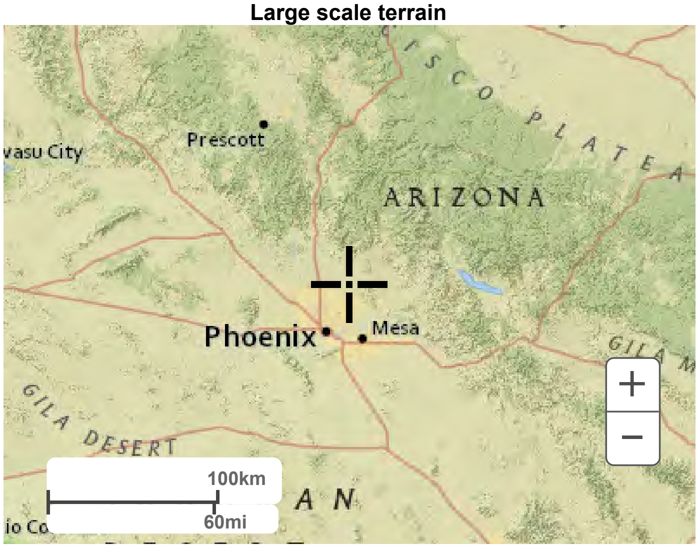
Maps & aerials

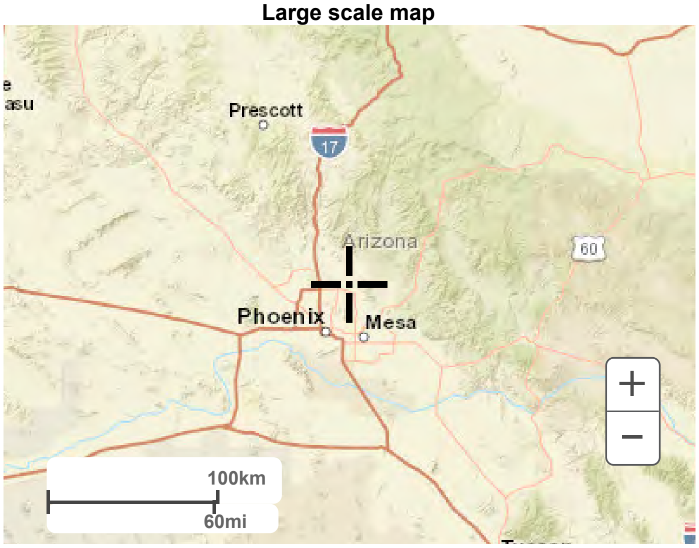
Small scale terrain



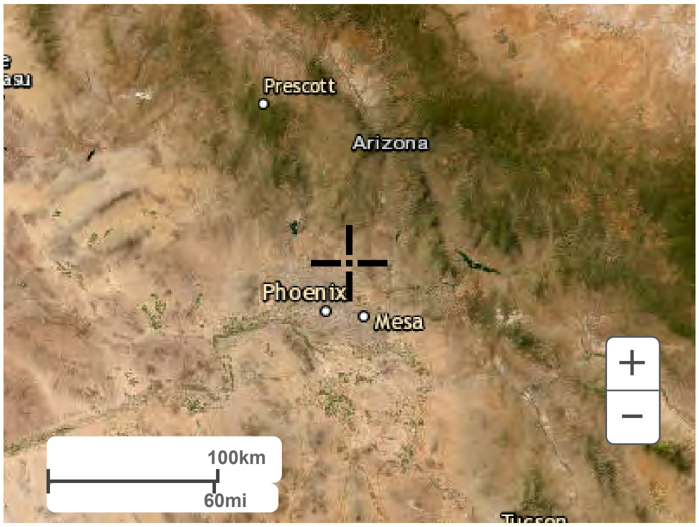
Large scale terrain



Large scale map



Large scale aerial



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Questions?: HDSC.Questions@noaa.gov

Disclaimer

NOAA Atlas 14, Volume 1, Version 5
Location name: Scottsdale, Arizona, USA*
Latitude: 33.6923°, Longitude: -111.9235°
Elevation: m/ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches/hour)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	**2.46** (2.05-3.02)	**3.22** (2.69-3.95)	**4.34** (3.60-5.30)	**5.21** (4.28-6.34)	**6.35** (5.15-7.70)	**7.24** (5.78-8.71)	**8.12** (6.40-9.77)	**9.02** (6.98-10.8)	**10.2** (7.72-12.3)	**11.1** (8.24-13.5)
10-min	**1.87** (1.55-2.30)	**2.45** (2.05-3.00)	**3.30** (2.74-4.04)	**3.96** (3.26-4.82)	**4.83** (3.91-5.86)	**5.50** (4.40-6.63)	**6.18** (4.87-7.43)	**6.87** (5.32-8.25)	**7.78** (5.87-9.37)	**8.48** (6.28-10.2)
15-min	**1.55** (1.29-1.90)	**2.02** (1.69-2.48)	**2.73** (2.26-3.34)	**3.27** (2.69-3.98)	**3.99** (3.24-4.84)	**4.54** (3.64-5.48)	**5.11** (4.02-6.14)	**5.68** (4.39-6.82)	**6.43** (4.86-7.74)	**7.01** (5.19-8.46)
30-min	**1.04** (0.866-1.28)	**1.36** (1.14-1.67)	**1.84** (1.52-2.25)	**2.20** (1.81-2.68)	**2.69** (2.18-3.26)	**3.06** (2.45-3.69)	**3.44** (2.71-4.14)	**3.82** (2.96-4.59)	**4.33** (3.27-5.21)	**4.72** (3.49-5.70)
60-min	**0.645** (0.536-0.791)	**0.842** (0.705-1.03)	**1.14** (0.941-1.39)	**1.36** (1.12-1.66)	**1.66** (1.35-2.02)	**1.89** (1.52-2.28)	**2.13** (1.68-2.56)	**2.36** (1.83-2.84)	**2.68** (2.02-3.22)	**2.92** (2.16-3.52)
2-hr	**0.375** (0.316-0.450)	**0.485** (0.409-0.584)	**0.645** (0.541-0.774)	**0.768** (0.637-0.919)	**0.935** (0.767-1.11)	**1.06** (0.859-1.26)	**1.19** (0.950-1.41)	**1.32** (1.04-1.57)	**1.50** (1.15-1.78)	**1.63** (1.23-1.95)
3-hr	**0.272** (0.228-0.332)	**0.348** (0.294-0.426)	**0.455** (0.382-0.556)	**0.540** (0.448-0.656)	**0.658** (0.538-0.793)	**0.751** (0.607-0.902)	**0.849** (0.673-1.02)	**0.951** (0.741-1.14)	**1.09** (0.826-1.30)	**1.20** (0.890-1.44)
6-hr	**0.163** (0.140-0.193)	**0.206** (0.178-0.244)	**0.263** (0.225-0.310)	**0.308** (0.261-0.362)	**0.370** (0.310-0.433)	**0.418** (0.345-0.487)	**0.468** (0.381-0.545)	**0.520** (0.414-0.606)	**0.589** (0.458-0.686)	**0.643** (0.488-0.750)
12-hr	**0.093** (0.080-0.108)	**0.117** (0.101-0.137)	**0.147** (0.127-0.172)	**0.172** (0.147-0.200)	**0.204** (0.173-0.237)	**0.229** (0.192-0.265)	**0.255** (0.210-0.295)	**0.281** (0.229-0.325)	**0.316** (0.251-0.367)	**0.342** (0.267-0.401)
24-hr	**0.055** (0.048-0.063)	**0.069** (0.061-0.080)	**0.090** (0.079-0.104)	**0.107** (0.093-0.123)	**0.130** (0.112-0.150)	**0.148** (0.126-0.171)	**0.168** (0.141-0.194)	**0.188** (0.156-0.217)	**0.216** (0.177-0.251)	**0.239** (0.192-0.280)
2-day	**0.030** (0.026-0.034)	**0.038** (0.033-0.044)	**0.050** (0.043-0.058)	**0.060** (0.052-0.069)	**0.073** (0.063-0.084)	**0.084** (0.071-0.097)	**0.095** (0.080-0.110)	**0.107** (0.089-0.124)	**0.124** (0.101-0.144)	**0.137** (0.111-0.161)
3-day	**0.021** (0.018-0.024)	**0.027** (0.024-0.031)	**0.036** (0.031-0.041)	**0.043** (0.037-0.049)	**0.053** (0.046-0.061)	**0.061** (0.053-0.070)	**0.070** (0.059-0.081)	**0.080** (0.067-0.092)	**0.093** (0.076-0.108)	**0.104** (0.084-0.121)
4-day	**0.017** (0.015-0.019)	**0.022** (0.019-0.025)	**0.029** (0.025-0.033)	**0.035** (0.030-0.039)	**0.043** (0.038-0.049)	**0.050** (0.043-0.057)	**0.058** (0.049-0.066)	**0.066** (0.055-0.075)	**0.077** (0.064-0.089)	**0.087** (0.070-0.101)
7-day	**0.011** (0.009-0.012)	**0.014** (0.012-0.016)	**0.019** (0.016-0.021)	**0.022** (0.019-0.026)	**0.028** (0.024-0.032)	**0.033** (0.028-0.037)	**0.038** (0.032-0.043)	**0.043** (0.036-0.050)	**0.050** (0.041-0.059)	**0.057** (0.046-0.067)
10-day	**0.008** (0.007-0.009)	**0.010** (0.009-0.012)	**0.014** (0.012-0.016)	**0.017** (0.015-0.019)	**0.021** (0.018-0.024)	**0.025** (0.021-0.028)	**0.028** (0.024-0.032)	**0.032** (0.027-0.037)	**0.038** (0.031-0.044)	**0.042** (0.034-0.049)
20-day	**0.005** (0.004-0.006)	**0.006** (0.006-0.007)	**0.009** (0.007-0.010)	**0.010** (0.009-0.012)	**0.013** (0.011-0.014)	**0.014** (0.012-0.017)	**0.016** (0.014-0.019)	**0.018** (0.016-0.021)	**0.021** (0.018-0.024)	**0.023** (0.019-0.027)
30-day	**0.004** (0.003-0.004)	**0.005** (0.004-0.006)	**0.007** (0.006-0.008)	**0.008** (0.007-0.009)	**0.010** (0.008-0.011)	**0.011** (0.010-0.013)	**0.013** (0.011-0.014)	**0.014** (0.012-0.016)	**0.016** (0.014-0.019)	**0.018** (0.015-0.021)
45-day	**0.003** (0.002-0.003)	**0.004** (0.003-0.004)	**0.005** (0.004-0.006)	**0.006** (0.005-0.007)	**0.008** (0.006-0.009)	**0.009** (0.007-0.010)	**0.010** (0.008-0.011)	**0.011** (0.009-0.012)	**0.012** (0.010-0.014)	**0.013** (0.011-0.016)
60-day	**0.002** (0.002-0.003)	**0.003** (0.003-0.004)	**0.004** (0.004-0.005)	**0.005** (0.004-0.006)	**0.006** (0.005-0.007)	**0.007** (0.006-0.008)	**0.008** (0.007-0.009)	**0.009** (0.007-0.010)	**0.010** (0.008-0.011)	**0.011** (0.009-0.012)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

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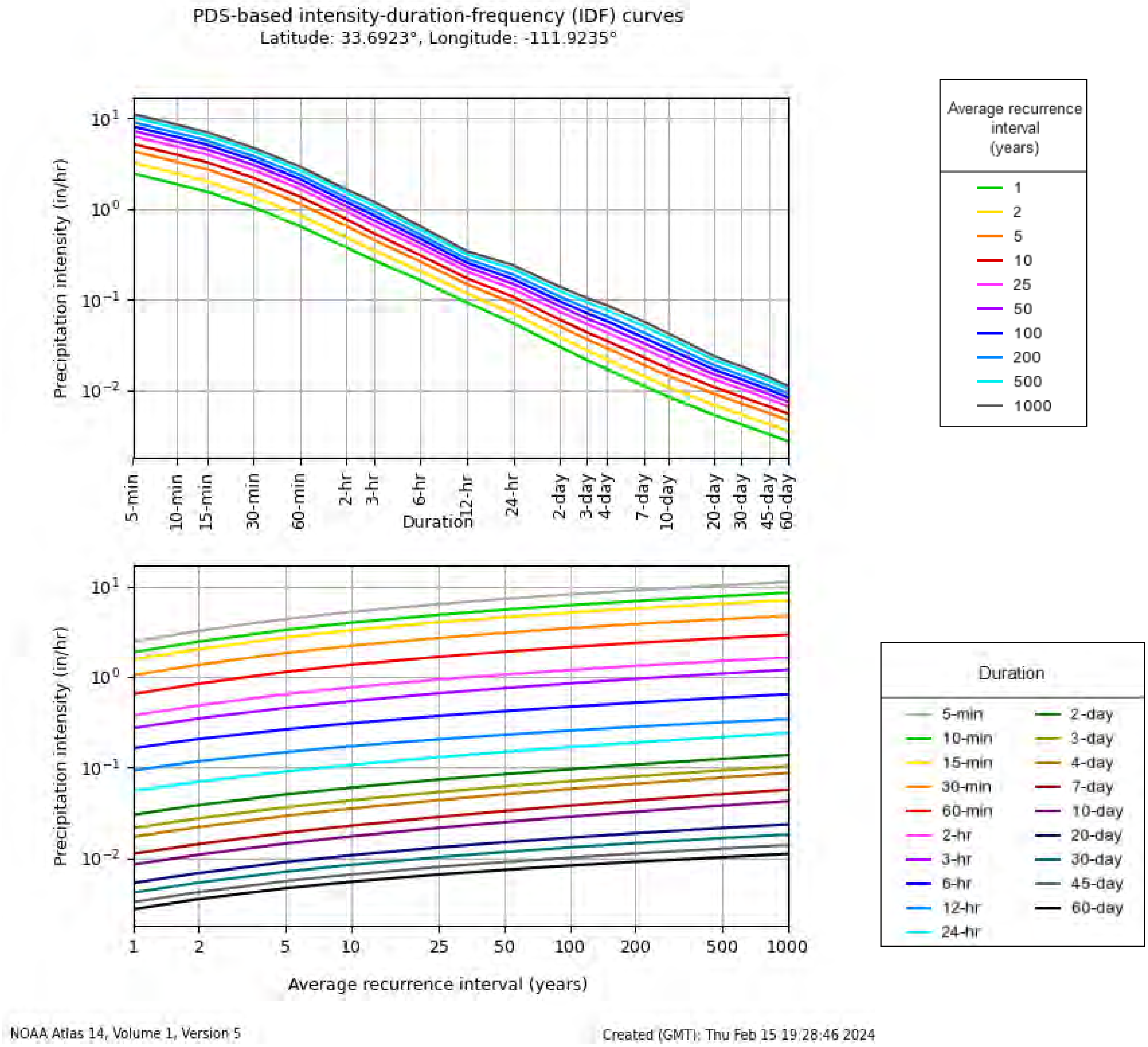
PF graphical



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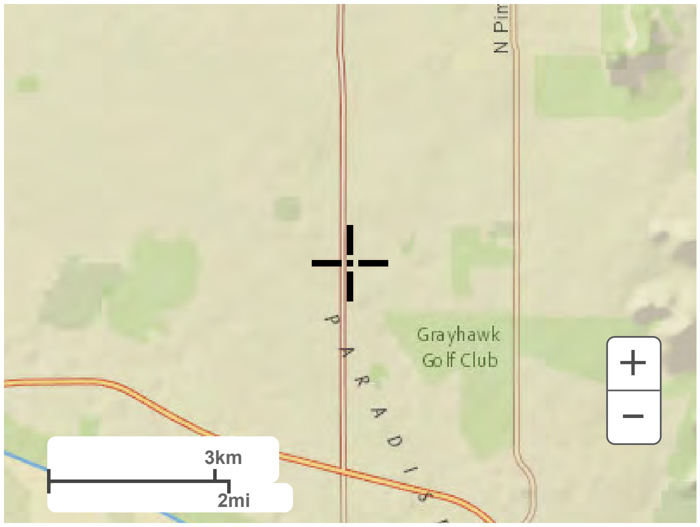
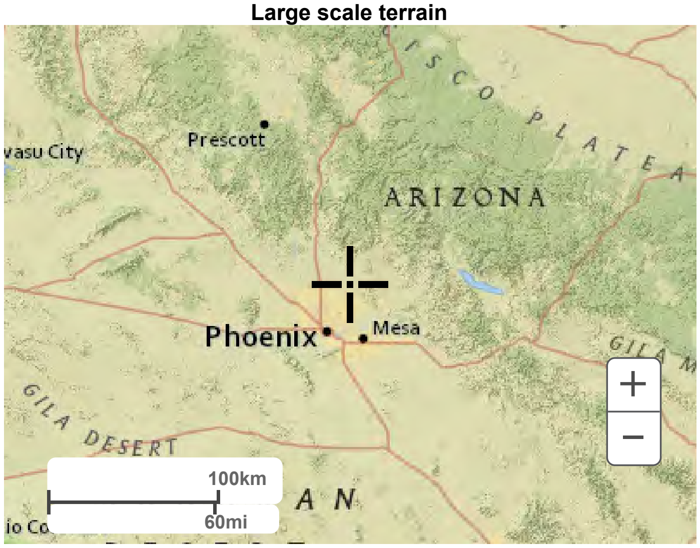
Maps & aerials

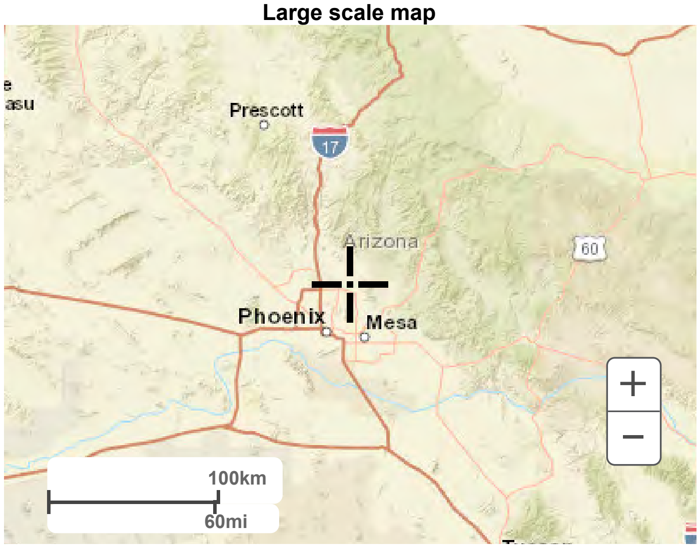
Small scale terrain



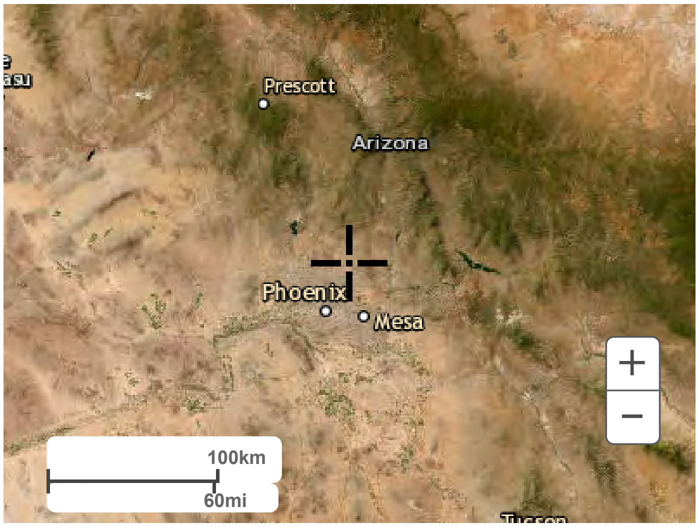
Large scale terrain



Large scale map



Large scale aerial



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National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

Disclaimer

2. A rainfall runoff model using the USACE's HEC 1 Flood Hydrograph Package (generally used for watersheds that are larger than 160 acres, irregular in shape and contour, or if routing of flows is necessary).

B. **Watershed Conditions**

Watersheds are subject to change. Grading and drainage plans shall consider all watershed conditions that would result in the greatest peak discharge rate, to:

1. Size drainage facilities, and
2. Determine lowest floor elevations.

C. **Split-Flow Conditions**

Projects in northern parts of Scottsdale must address split-flow channel conditions where applicable. These splits in the alluvial channels usually include highly erosive soils and are generally unstable and unpredictable. In setting lowest floor elevations relative to upstream splits, assume that 100% of the flow could go either direction in any given flood event. For infrastructure design, the estimate of the actual split, based on a hydraulic analysis of the current channel cross sections, must include a minimum safety factor of 30% of the total flow. If there are extenuating factors affecting the stability of the split, the safety factor should be increased accordingly.

D. **Environmentally Sensitive Lands**

For special considerations regarding Environmentally Sensitive Lands, refer to the City Zoning Ordinance and DSPM Chapter 2 Section 2-2. Modification of natural watercourses with a flow of 50 cfs or greater are addressed in the City Zoning Ordinance.

E. **The Rational Method**

1. Precipitation. Precipitation input is rainfall intensity, "i," and can be obtained directly from NOAA 14.
2. Time of Concentration. Time of concentration "t_c" is the total time of travel from the most hydraulically remote part of the watershed to the concentration point of interest. The calculation of "t_c" must follow FCDMC Hydrology Manual procedures.
3. Runoff Coefficients. Use Fig. 4-1.5, Runoff Coefficients for Use with Rational Method, or equivalent to obtain the runoff coefficients or "C" values. Composite "C" values for the appropriate zoning category or weighted average values calculated for the specific site are both acceptable approaches.

RUNOFF COEFFICIENTS – "C" VALUE

LAND USE	**STORM FREQUENCY**		
Composite Area-wide Values	2-25 Year	50 Year	100 Year
Commercial & Industrial Areas	0.80	0.83	0.86
Residential Areas – Single Family, slopes 10% or less			
R1-190	0.33	0.50	0.53
R1-130	0.35	0.51	0.59

R1-70	0.37	0.52	0.60
R1-43	0.38	0.55	0.61
R1-35	0.40	0.56	0.62
R1-18	0.43	0.58	0.64
R1-10	0.47	0.62	0.70
R1-7	0.51	0.66	0.80
R1-5	0.54	0.69	0.86
Residential Areas – Single Family, slopes greater than 10%			
R1-190	0.65	0.74	0.82
R1-130	0.68	0.76	0.84
R1-70	0.69	0.77	0.85
R1-43	0.70	0.77	0.85
R1-35	0.70	0.78	0.85
R1-18	0.71	0.79	0.86
R1-10	0.75	0.82	0.88
R1-7	0.81	0.86	0.91
R1-5	0.85	0.89	0.92
Townhouse (R-2, R-4)	0.63	0.74	0.94
Apartments & Condominiums (Condos) (R-3, R-5)	0.76	0.83	0.94
Specified Surface Type Values			
Paved streets, parking lots (concrete or asphalt), roofs, driveways, etc.	0.90	0.93	0.95
Lawns, golf courses, & parks (grassed areas)	0.20	0.25	0.30
Undisturbed natural desert or desert landscaping (no impervious weed barrier)	0.37	0.42	0.45
Desert landscaping (with impervious weed barrier)	0.63	0.73	0.83
Mountain terrain - slopes greater than 10%	0.60	0.70	0.80
Agricultural areas (flood irrigated fields)	0.16	0.18	0.20
Gravel floodways and shoulders	0.68	0.78	0.82

FIGURE 4-1.5 RUNOFF COEFFICIENTS FOR RATIONAL METHOD

F. **HEC-1 Model**

1. Minimum submittals
   a. A printout of the input data.
   b. A schematic (routing) diagram of the stream network.
   c. The runoff summary output table, including drainage basin name, area, 2, 10, and 100- year flow values.
   d. Electronic input file(s) on compact disc (CD) or digital versatile/video disc (DVD).
   e. Supporting documentation and source material for parameter selection.

RETENTION REQUIRED (100yr 2hr)

Sub-Area	Area	C-Value	P	Volume	Volume
-	(SF)	#	inches	CF	AF
A	136915	0.94	2.38	25526	0.59
B	70925	0.94	2.38	13223	0.30
C	106305	0.94	2.38	19819	0.45
D	187009	0.94	2.38	34865	0.80
E	13262	0.94	2.38	2472	0.06
Total	**514416**			**95904**	**2.20**

RETENTION REQUIRED (Pre-Vs Post)

Sub-Area	Area	C-Value	C-Value	C-Value	P	Volume	Volume
-	(SF)	Pre	Post	Pre-v-Post	inches	CF	AF
A	136915	0.45	0.94	0.49	2.38	13306	0.31
B	70925	0.45	0.94	0.49	2.38	6893	0.16
C	106305	0.45	0.94	0.49	2.38	10331	0.24
D	187009	0.45	0.94	0.49	2.38	18174	0.42
E	13262	0.45	0.94	0.49	2.38	1289	0.03
Total	**514416**					**49993**	**1.15**

RETENTION REQUIRED (First Flush)

Sub-Area	Area	C-Value	P	Volume	Volume
-	(SF)	#	inches	CF	AF
A	136915	1.00	0.50	5705	0.13
B	70925	1.00	0.50	2955	0.07
C	106305	1.00	0.50	4429	0.10
D	187009	1.00	0.50	7792	0.18
E	13262	1.00	0.50	553	0.01
Total	**514416**			**21434**	**0.49**

NOTE: Sub Areas A, & V are not included in table, as these are associated offsite drainage areas

Basin Volume - Basin A

Elevation	Area	Average Area	TOTAL	TOTAL
FT	SF	SF	CF	AF
91	2278			0.00
92	3680	2979	2979	0.07
93	5719	4700	7679	0.18
94	8295	7007	14686	0.34
		TOTAL	**14686**	**CF**

Volume Required (Pre-vs-Post)	
Subbasin	
A	
(CF)	
13306	
Volume OK	CF

Basin Volume - Basin B

G	Area	Average Area	TOTAL	TOTAL
FT	SF	SF	CF	AF
2	1696			0.00
3	4926	3311	3311	0.08
4	8353	6640	9951	0.23
5	12028	10191	20141	0.46
		TOTAL	**20141**	**CF**

Volume Required (Pre-vs-Post)	
Subbasin	
B	
(CF)	
6893	
Volume OK	CF

Basin Volume - Basin C

Elevation	Area	Average Area	TOTAL	TOTAL
FT	SF	SF	CF	AF
1	5326			0.00
2	7112	6219	6219	0.14
3	9347	8230	14449	0.33
4	11825	10586	25035	0.57
		TOTAL	**25035**	**CF**

Volume Required (Pre-vs-Post)	
Subbasin	
C	
(CF)	
10331	
Volume OK	CF

Basin Volume - Basin D

Elevation	Area	Average Area	TOTAL	TOTAL
FT	SF	SF	CF	AF
1	1665			0.00
2	4705	3185	3185	0.07
3	7880	6293	9478	0.22
4	11168	9524	19002	0.44
		TOTAL	**19002**	**CF**

Volume Required (Pre-vs-Post)	
Subbasin	
D	
(CF)	
18174	
Volume OK	CF

Basin Volume - Basin E

Elevation	Area	Average Area	TOTAL	TOTAL
FT	SF	SF	CF	AF
3	0			0.00
4	50	25	25	0.00
5	1,289	670	695	0.02
6	1785	1537	2232	0.05
		TOTAL	**2232**	**CF**

Volume Required (Pre-vs-Post)	
Subbasin	
E	
(CF)	
1289	
Volume OK	CF

Basin Percolation Rates - FOR VOLUME PROVIDED

Sub-Area	Rate of Bleedoff	Total Volume Provided	Dry-Up Time	# drywells for 36 hour dry up	
	(cfs)	(cf)	(hr)	#	
Basin A	0.1	14,686	40.8	1.1	**USE 2 DRYWELLS**
Basin B	0.1	20,141	55.9	1.6	**USE 2 DRYWELLS**
Basin C	0.1	25,035	69.5	1.9	**USE 2 DRYWELLS**
Basin D	0.1	19,002	52.8	1.5	**USE 2 DRYWELLS**
Basin E	0.1	2,232	6.2	0.2	**USE 1 DRYWELL**

Post-Development Rational Method Calculations for Inlets

Sub-Area	Area	Area	C_{10}	C_{100}	Tc	i_{10}	i_{100}	Local Q_{10}	Local Q_{100}
	(SF)	(AC)	-	-	(min)	(in/hr)	(in/hr)	(cfs)	(cfs)
Portion of A	49500	1.14	0.74	0.94	5	5.21	8.12	4.38	8.67
Portion of B	13000	0.30	0.76	0.94	5	5.76	8.98	1.31	2.52
Portion of C	69000	1.58	0.76	0.94	5	5.76	8.98	6.93	13.37
Portion of D	32500	0.75	0.76	0.94	5	5.76	8.98	3.27	6.30

Curb Opening Catch Basin Capacity Calculations - Weir Condition

Inlet Type	Inlet Area	Q	Inlet Capacity W/25% Clogging	d	Cw	L	
		(cfs)	(cfs)	(ft)		(ft)	
Curb Opening CB	Portion of A	4.38	5.09	0.50	3	6	Designed for 10yr

Q=Cw*L*d^1.5

L=(Q/(Cw*d^1.5))*1.25

Cw= 3 weir coefficient
Q = discharge capacity
L = curb opening length
d = flow depth
CF = clogging factor = 25% (1.25xL)

Q10 within curb
Q100 to overtop into basin

Type F Catch Basin - Grated Inlet Capacity - Weir Condition

Inlet Type	Inlet Area	Q	Inlet Capacity W/50% Clogging	d	Cw	P[1]	
		(cfs)	(cfs)	(ft)		(ft)	
Type F CB	Portion of B	2.52	8.37	0.50	3	11.83	Designed for 100yr
Type F CB	Portion of C	13.37	18.03	0.50	3	25.50	Designed for 100yr
Type F CB	Portion of D	6.30	8.37	0.50	3	11.83	Designed for 100yr

Q=Cw*P*d^1.5

[1] **Wetted Perimeter**	**ft**
1 Type F Catch Basins	11.83
2 Type F Catch Basins	18.67
3 Type F Catch Basins	25.50
4 Type F Catch Basins	32.33

Cw= 3.0 weir coefficient
Q = discharge capacity
P = inlet perimeter
d = flow depth

100YR Over-Flow Q Calculations - for Basins that don't retain 100yr

Overflow ID	Overflow Type	Basin	Overflow Volume	A	P	C	i_{100}	Q_{100}
#	#	-	CF	AC	in	(in/hr)	(in/hr)	(cfs)
1	RIP RAP WEIR	A	10840	1.33	2.38	0.94	6.18	7.75
2	CATCH BASIN	B	-6918	-0.85	2.38	0.94	6.18	-4.95
3	RIP RAP WEIR	C	-5216	-0.64	2.38	0.94	6.18	-3.73
4	CATCH BASIN	D	15863	1.95	2.38	0.94	6.18	11.35
5	RIP RAP WEIR	E	241	0.03	2.38	0.94	6.18	0.17

NOTE: BASIN B&C RETAIN 100-YR VOLUME -NO OVERFLOW NECESSARY

V = Overflow Volume
V = Vr(100 yr) - Vp
V = P/12 x C x A   Solve for A

Q = C x i x A   Solve for Q (use 10min of Tc)

Finished Floor Requirements

Lot #	Highest Elevation from 3 engineering Field Survey	AO Flood Zone Depth PAD	Flood Elevation PAD	Min. PAD Elev. Based on Flood Elev.	Min. FF Elev. Based on Flood Elev.	Proposed PAD	Proposed Finished Floor Main	Proposed Finished Floor Face of Garage	Prop. FF Meets Flood Depth
#	FT	FT	FT	FT	FT	FT	FT	FT	(Y/N)
13-18	1797.30	1.00	1795.00	1796.00	1797.00	1804.50	1805.50	1805.00	Y
40-42	1797.80	1.00	1798.80	1799.80	1800.80	1805.00	1806.00	1805.50	Y
43-45	1799.90	2.00	1801.90	1802.90	1803.90	1805.50	1806.50	1806.00	Y
46-48	1801.20	2.00	1803.20	1804.20	1805.20	1806.00	1807.00	1806.50	Y

Williams Drive Channel

Rip Rap Calculations

Per SSA 7-98 May 1998

$D50 = 0.0648Q^{0.4}$

Q=	140	cfs	
D50 =	0.47	feet	Use 6-inches

T = 2 x D50
T = 2 x 0.50
T= 20-inches **RIP RAP SPEC - 12" Thick Rip Rap D50 = 6"**

Williams Drive Channel

Scour Depth Calculations

Per SSA 5-96 September 1996

$d_s = d_{gs} + d_{lts}$	Q_{100} =	140	cfs	
d_s = scour depth	d_{gs} =	1.13	feet	
d_{gs} = general degradation	d_{lts} =	0.39	feet	
d_{lts} = long term degrdation	**d_s =**	**1.52**	**feet**	**USE MINIMUM of 3-feet below flow line**

$d_{gs} = 0.157(Q_{100})^{0.4}$

$d_{lts} = 0.02(Q_{100})^{0.6}$

Culvert Calculator Report
West Entry

Solve For: Headwater Elevation

Culvert Summary					
Allowable HW Elevation	100.00	ft	Headwater Depth/Height	1.21	
Computed Headwater Eleva	99.53	ft	Discharge	140.00	cfs
Inlet Control HW Elev.	99.47	ft	Tailwater Elevation	3.00	ft
Outlet Control HW Elev.	99.53	ft	Control Type	Outlet Control	

Grades					
Upstream Invert	96.50	ft	Downstream Invert	96.00	ft
Length	100.00	ft	Constructed Slope	0.005000	ft/ft

Hydraulic Profile					
Profile	M2		Depth, Downstream	1.80	ft
Slope Type	Mild		Normal Depth	1.98	ft
Flow Regime	Subcritical		Critical Depth	1.80	ft
Velocity Downstream	7.38	ft/s	Critical Slope	0.006148	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	2.50	ft
Section Size	30 inch		Rise	2.50	ft
Number Sections	5				

Outlet Control Properties					
Outlet Control HW Elev.	99.53	ft	Upstream Velocity Head	0.71	ft
Ke	0.50		Entrance Loss	0.36	ft

Inlet Control Properties					
Inlet Control HW Elev.	99.47	ft	Flow Control	Transition	
Inlet Type	Square edge w/headwall		Area Full	24.5	ft²
K	0.00980		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	1	
C	0.03980		Equation Form	1	
Y	0.67000				

Culvert Calculator Report
Existing Culvert - East

Solve For: Headwater Elevation

Culvert Summary			
Allowable HW Elevation	106.00 ft	Headwater Depth/Height	1.47
Computed Headwater Elevε	106.84 ft	Discharge	130.00 cfs
Inlet Control HW Elev.	106.84 ft	Tailwater Elevation	105.10 ft
Outlet Control HW Elev.	106.78 ft	Control Type	Inlet Control

Grades			
Upstream Invert	103.90 ft	Downstream Invert	102.60 ft
Length	100.00 ft	Constructed Slope	0.013000 ft/ft

Hydraulic Profile			
Profile	CompositePressureProfileS1S2	Depth, Downstream	1.19 ft
Slope Type	N/A	Normal Depth	1.16 ft
Flow Regime	N/A	Critical Depth	1.59 ft
Velocity Downstream	10.06 ft/s	Critical Slope	0.005687 ft/ft

Section			
Section Shape	Horizontal Ellipse	Mannings Coefficient	0.013
Section Material	Concrete	Span	3.15 ft
Section Size	24x38 inch	Rise	2.00 ft
Number Sections	4		

Outlet Control Properties			
Outlet Control HW Elev.	106.78 ft	Upstream Velocity Head	0.86 ft
Ke	0.50	Entrance Loss	0.43 ft

Inlet Control Properties			
Inlet Control HW Elev.	106.84 ft	Flow Control	Submerged
Inlet Type	Square edge with headwall (horizontal ellipse)	Area Full	20.4 ft²
K	0.01000	HDS 5 Chart	29
M	2.00000	HDS 5 Scale	1
C	0.03980	Equation Form	1
Y	0.67000		

Worksheet for Williams Drive Channel

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.00670	ft/ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	5.00	ft
Discharge	140.00	ft³/s

Results

Normal Depth	2.48	ft
Flow Area	30.86	ft²
Wetted Perimeter	20.69	ft
Hydraulic Radius	1.49	ft
Top Width	19.88	ft
Critical Depth	1.99	ft
Critical Slope	0.01725	ft/ft
Velocity	4.54	ft/s
Velocity Head	0.32	ft
Specific Energy	2.80	ft
Froude Number	0.64	
Flow Type	Subcritical	

GVF Input Data

Downstream Depth	0.00	ft
Length	0.00	ft
Number Of Steps	0	

GVF Output Data

Upstream Depth	0.00	ft
Profile Description		
Profile Headloss	0.00	ft
Downstream Velocity	Infinity	ft/s
Upstream Velocity	Infinity	ft/s
Normal Depth	2.48	ft
Critical Depth	1.99	ft
Channel Slope	0.00670	ft/ft

Worksheet for Williams Drive Channel

GVF Output Data

Critical Slope	0.01725	ft/ft

Cross Section for Williams Drive Channel

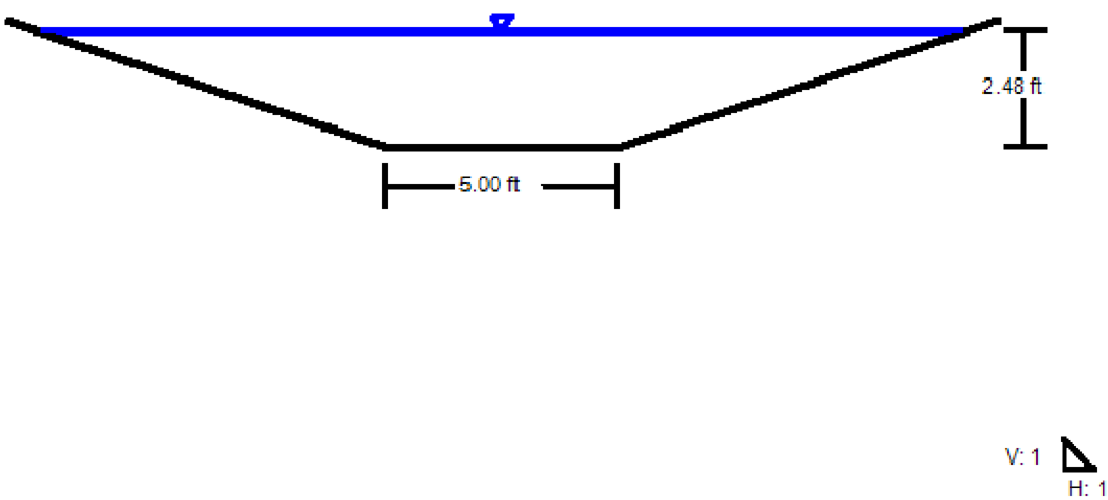
Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.035	
Channel Slope	0.00670	ft/ft
Normal Depth	2.48	ft
Left Side Slope	3.00	ft/ft (H:V)
Right Side Slope	3.00	ft/ft (H:V)
Bottom Width	5.00	ft
Discharge	140.00	ft³/s

Cross Section Image



APPENDIX H

Excerpts from Existing Drainage Studies

ADDENDUM No. 2 TO THE MASTER DRAINAGE REPORT FOR SILVERSTONE

Stormwater Review By:
Richard Anderson
Phone 480-312-2729
FAX 480-312-9202
E-MAIL rianderson@ScottsdaleAZ.gov
Review Cycle ______ Date 5/21/18

3476-06-18

Prepared For:

Approved

Mark-Taylor Residential
6623 North Scottsdale Road
Scottsdale, Arizona 85250

Prepared By:

Kimley-Horn and Associates, Inc.
7740 North 16th Street
Suite 300
Phoenix, Arizona 85020

Registered Professional Engineer (Civil)
CERTIFICATE NO. 37059
STEPHEN E. HANEY
Date Signed 4/28/15
ARIZONA, U.S.A.
Exp. 12/31/14

191769002
April 2015

Kimley»Horn

DRAINAGE

3476-06-18

4/30/15

ADDENDUM No.2

COS #425-SA-2006, 315-ZN-2006, PC#3476-06-2

- Addendum No.2 to the Master Drainage Report Silverstone- March 2007
- City of Scottsdale, Stormwater Management Division, Approved 3/18/07 (Original) Addendum No. 1 approved 3/6/14.

- Justification #1: Parcel E as shown is will provide a single detention basin in the scenic corridor along Scottsdale Road. Prior to the Rawhide wash improvements, storm water was conveyed south along Scottsdale Road to Williams Road. With the completed improvements to Rawhide Wash Channel upstream of Parcel E, the existing channel in the scenic corridor is now underutilized. To be consistent with the natural land form, the existing channel along Scottsdale Road will be utilized as the detention basin area. The single location of detention along Scottsdale Road in lieu or the two previous basins, provides for more developable area of the adjacent parcel and contains basin overflow to the Scenic Corridor area; therefore eliminating the unnecessary storm water burden on the downstream parcels. This single basin along Scottsdale Road also eliminates the proposed drainage corridor that divides the parcel to the south, therefore allowing for increased development opportunities. The ultimate outfall for the Master Drainage Plan remains unchanged from the intersection of Scottsdale Road and Williams Road. See attached exhibit Plate 3A for parcels E and D revised drainage map.

- Justification #2: The single retention basin for Parcel E will provide a detention for the pre-versus-post development. Per the City of Scottsdale Policy, areas of the proposed development that were previously developed are required to provide a storage volume equal to the increase in the runoff volume generated by the proposed development during the design storm event (100-year 2-hr). Parcel E is part of the previously developed Rawhide development which included parking and an elevated race track.


NORTH
GRAPHIC SCALE IN FEET
0 40 80 160
Kimley»Horn
© 2015 KIMLEY-HORN AND ASSOCIATES, INC.
7878 North 16th Street, Suite 300
Phoenix, Arizona 85020 (602) 944-5500
PLATE 3A - SILVERSTONE DRAINAGE MAP
(ENLARGED PARCEL E AND D)
LEGEND
PROPOSED DETENTION BASIN (POSSIBLE LOCATIONS)
PROPOSED FLOW DIRECTION
TEMPORARY/PROPOSED DRAINAGE CORRIDOR PER FINAL PARCEL DETAILS (APPROXIMATE LOCATION)
SILVERSTONE DRIVE
SCOTTSDALE ROAD
74TH STREET
WILLIAMS DRIVE
E
PARCEL AREA=16.8± AC
D
PARCEL AREA=13.5 AC
DETENTION REQ.=1.00 AC-FT
DETENTION REQ.=2.44 AC-FT

EXHIBIT 4 - POST-DEVELOPED CONDITION MAP PARCEL E
NORTH
GRAPHIC SCALE IN FEET
0 20 40 80
Kimley»Horn
© 2015 KIMLEY-HORN AND ASSOCIATES, INC.
7878 North 16th Street, Suite 300
Phoenix, Arizona 85020 (602) 944-5500
LEGEND
10 PROPOSED MAJOR CONTOUR
12 PROPOSED MINOR CONTOUR
10 EXISTING MINOR CONTOUR
12 EXISTING MAJOR CONTOUR
00 DRAINAGE AREA ID
SILVERSTONE DRIVE
SCOTTSDALE ROAD
74TH STREET
BASIN A
HW= 11.00
BTM= 8.00
Vreq= 22,513 CF
Vprov= 22,954 CF
BASIN B
HW= 5.50
BTM= 8.50
Vreq= 21,021 CF
Vprov= 21,545 CF

FINAL DRAINAGE REPORT

Mark-Taylor San Portales Apartments
Scottsdale, Arizona

Case No. 53-DR-2014

Prepared For:

Mark-Taylor Residential

191769002
March 2016

FINAL DRAINAGE REPORT

Mark-Taylor San Portales Apartments
Scottsdale, Arizona

Case No. 53-DR-5014

Prepared For:

Mark-Taylor Residential
6623 North Scottsdale Road
Scottsdale, Arizona 85250

Prepared By:

Kimley-Horn and Associates, Inc.
7740 North 16th Street
Suite 300
Phoenix, Arizona 85020

191769002
March 2016

Refer to Figures 2 and 4 in Appendix F for the Grading and Drainage Plan and the Site Basin Delineation, respectively. Refer to Appendix C for the Hydrologic/Hydraulic Calculations.

3.6 Pre- and Post-Development Runoff Characteristics at Concentration Points

The existing site consists of approximately 16.7 acres of vacant land that drains from the northeast to the southwest. Due to the current topography, there is no single concentration point for the storm water flows generated by the project site. As previously discussed, in the pre-developed condition approximately 37 cfs exits the site at the existing Scottsdale Road channel and approximately 43 cfs exits the site as sheet flow along the southern property line.

The post-development flow at the Scottsdale Road channel will be approximately 49.4 cfs for the 100-year, 2-hour rainfall event, which is less than the total pre-development flow of 80 cfs. Refer to Section 3.7 for the distribution of the flows at this location.

For events where the storage capacity of the surface basins along Scottsdale Road is not exceeded, storm water will be discharged from these basins to the Scottsdale Road channel through six-inch bleed-off pipes. The flow from these pipes will be approximately 2 cfs and will drain the detention basins in 36 hours or less.

The proposed outfall elevations of the surface detention basins will be set above existing grade, which will allow the future development on the south parcel the option of continuing this storm drain pipe, or providing a channel within the Scottsdale Road scenic corridor setback. A temporary construction easement is provided in the area surrounding the outfall to allow the adjacent property owner to connect to the proposed outfall structure. As previously mentioned, post-development flows will be not exceed pre-development flows, which will benefit the downstream development by decreasing the size of pipe or channel required to convey this flow.

3.7 Proposed Drainage Structures or Special Drainage Facilities

The storm drain pipe system is divided into two networks, with the northern network discharging to Basin A and the southern network discharging to Basin B. A portion of each pipe network contains 48-inch HDPE pipe that will be used for both conveyance and detention. Each network also has diversion structure located upstream of the outfall to the surface detention basins. Storm water flowing into the diversion structure from the 48-inch detention/conveyance pipes will flow out of the diversion structure through a six-inch pipe. This six-inch pipe will convey storm water directly to the site outfall at the southwest corner of the property. Once the capacity of the six-inch pipe is exceeded, storm water will back up into the 48-inch pipes. After the 48-inch pipes are filled, storm water will exit the diversion structure through an overflow pipe, which will convey storm water to the surface detention basins. The invert of the overflow pipes are set to the crowns of the incoming 48-inch detention/conveyance pipes, thereby allowing the 48-inch pipes to detain storm water before overtopping to the surface detention basins.

Based on this design, storm water from smaller rainfall events will be detained entirely within the underground storm drain system, and slowly discharged to the Scottsdale Road channel at the southwest corner of the site. Larger rainfall events will fill the surface detention basins along Scottsdale Road, and these basins will also drain to the Scottsdale Road channel south of the site.

As previously noted, the site will provide detention for the difference between the pre-development and post-development storm water volume. Storm water in excess of this volume will overtop the basins and flow south into the Scottsdale Road channel, in accordance with pre-development drainage patterns.

The computer program Pond Pack, by Bentley Systems, was used to analyze the routing and storage for the surface and underground detention systems. This approach was used to ensure that the basins were filling to capacity, to determine the peak discharges from the basins, and to determine the attenuation provided by the system.

The site was divided into eight catchment areas, and a unit hydrograph was developed for each catchment area. The catchment areas represented the rational flows generated by several smaller drainage areas. Combining several small drainage areas into a single catchment area represents a negligible approximation due to the small contributing areas and the minimal times of concentration.

These hydrographs are then combined and routed through the storm drain system to determine the peak flows and storage capacities of the basins. The diversion structures were modeled as composite outlet structures with two orifices at different elevations. A six-inch orifice was used to model the bleed-off pipe, and a 30-inch orifice was used to model the overflow pipe that discharges to the surface detention basin. Similarly, a composite outlet structure was used to model the discharge from the surface basins. A six-inch orifice was used for the bleed-off pipe, and the rip-rap spillway was modeled as a weir with the same dimensions as the spillway.

The results of the Pond Pack analysis are summarized in Table 2 below.

Table 2: Pond Pack Analysis Results

Basin	100-Year Peak Inflow	100-Year Peak Discharge	100-Year Depth*	100-Year Storage Volume		Time to Max. Volume
	cfs	cfs	Ft	ac-ft	cf	hr.
PO-UA	53.73	39.37	4.00	0.312	13,591	0.20
PO-UB	36.12	25.32	4.00	0.103	4,487	0.20
PO-A	36.91	31.61	2.92	0.249	10,846	0.35
PO-B	52.71	45.36	2.94	0.326	14,201	0.40
Total				0.990	43,124	

Table 2: Pond Pack Analysis Results (cont'd)

Basin	10-Year Peak Inflow	10-Year Peak Discharge	10-Year Depth*	10-Year Storage Volume		Time to Max. Volume
	cfs	cfs	ft	ac-ft	cf	hr.
PO-UA	32.96	23.52	2.34	0.287	12,502	0.30
PO-UB	21.67	19.50	3.51	0.101	4,400	0.25
PO-A	21.37	14.11	2.55	0.206	8,973	0.40
PO-B	23.47	19.05	2.53	0.261	11,369	0.45
Total				0.855	37,244	

Basin	2-Year Peak Inflow	2-Year Peak Discharge	2-Year Depth*	2-Year Storage Volume		Time to Max. Volume
	cfs	cfs	ft	ac-ft	cf	hr.
PO-UA	20.77	11.66	1.35	0.236	10,280	0.50
PO-UB	13.73	10.94	2.72	0.090	3,920	0.25
PO-A	9.72	1.94	2.04	0.148	6,447	0.45
PO-B	9.29	2.01	2.03	0.179	7,797	0.50
Total				0.653	28,445	

* For underground pipe storage, depth is taken at upstream pipe invert.

Refer to Appendix G for Pond Pack analysis results.

A weir and spillway was designed with Pond Pack at each basin outfall to control the flows discharging from the basins. For both basins, a weir with a crest height of two feet above the basin bottom was used, and the spillway is ten feet wide with 4:1 side slopes. This spillway design results in a flow depth of approximately 0.9 feet above the weir crest, thereby providing approximately three feet in depth for retention storage volume.

The flow from the 100-year, 2-hour storm entering Basin A from the storm drain system is 36.9 cfs. A unit hydrograph was used to route this flow through the basin, and the peak discharge from Basin A is 31.6 cfs. This storm water exits Basin A over the weir and flows to Basin B. Based on the Pond Pack results, the 100-year flow discharging from Basin B is 45.4 cfs. Combined with the pipe discharge of 4 cfs noted above, the 100-year, 2-hour site discharge is 49.4 cfs which is less than the pre-development flow of 80 cfs.

Refer to Appendix G for the Pond Pack output.

Storm water quality will be maintained by elevating the outfall headwalls six inches above the basin bottom. This will allow the first flush of storm water to remain in the basin, where it will be disposed of via natural percolation. A temporary construction

STORMCAD LAYOUT

Kimley»Horn

K:\PHX_Civil\191769002\CADD\Exhibits\2015-10-28 StormCAD Exhibit.dwg Mar 28, 2016 trevor.jones

Plan # 4632-06-1
Case # 247-SA-2006
Q-S # 44-45

☑ Accepted
☐ Corrections

Richard W. Orr — 4/12/07
Reviewed By — Date

WOOD/PATEL

DRAINAGE REPORT
FOR
WILLIAMS DRIVE
SCOTTSDALE ROAD TO MILLER ROAD

January 2007

WP #042309.05

Prepared for:

Silverstone Development, Inc.
1550 East Missouri Avenue
Suite 300
Phoenix, Arizona 85011
Phone: (602) 230-1051
Fax: (602) 230-2826
Attn: Mr. Mike Pacheco

Submitted to:

City of Scottsdale
7447 East Indian School Road
Scottsdale, Arizona 85251

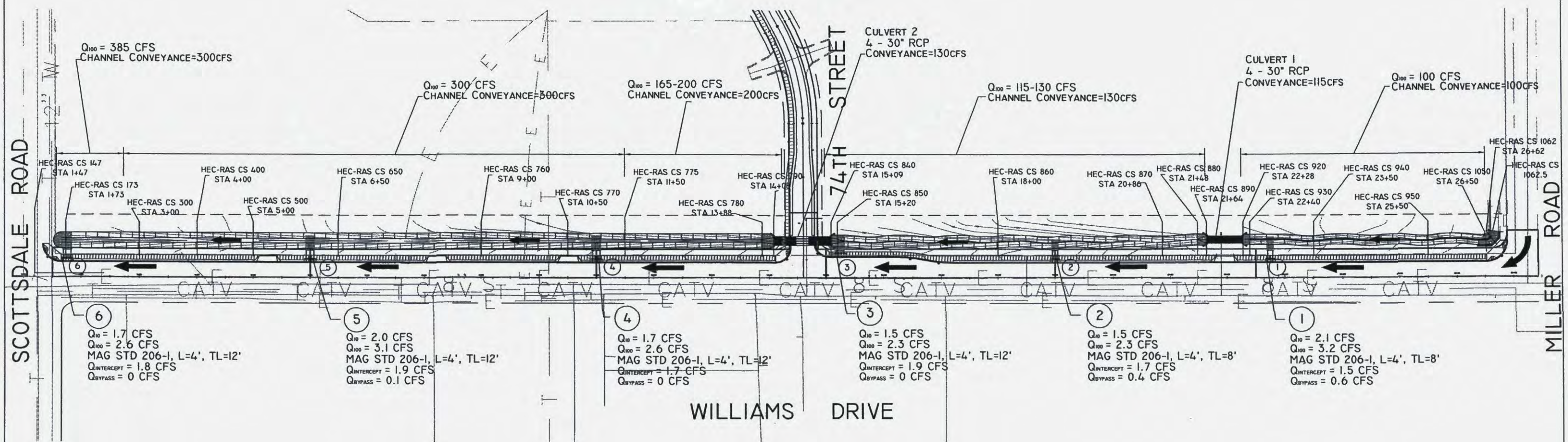
Prepared by:

Wood, Patel & Associates, Inc.
2051 West Northern Avenue
Suite 100
Phoenix, Arizona 85021
Phone: (602) 335-8500
Fax: (602) 335-8580
Website: www.woodpatel.com

Registered Professional Engineer (Civil)
CERTIFICATE NO.
13138
DARREL E.
WOOD
Date Signed 3...
ARIZONA, U.S.A.

Engineer-in-Training
Darren Forstie

Engineer



LEGEND

(3)	CONCENTRATION POINT/SCUPPER NUMBER
←	FLOW DIRECTION
Q_{10} = 2.1 CFS	10-YEAR DISCHARGE
Q_{100} = 4.2 CFS	100-YEAR DISCHARGE
MAG STD 206-1, L=4', TL=8'	SCUPPER TYPE, OPENING LENGTH (L), TOTAL SCUPPERS LENGTH (TL) (TL = TYPICAL SCUPPER LENGTH + NUMBER OF SCUPPERS)
$Q_{INTERCEPT}$ = 1.7 CFS	10-YEAR DISCHARGE, INTERCEPTED AT SCUPPER
Q_{BYPASS} = 0.5 CFS	10-YEAR DISCHARGE BYPASSED TO DOWNSTREAM SCUPPER

N

NTS

N:\2004\042309.05\Project Support\Hydro\Exhibits\ Drainage Map_12-08-06.dwg - Dec 18,2006 3:31pm

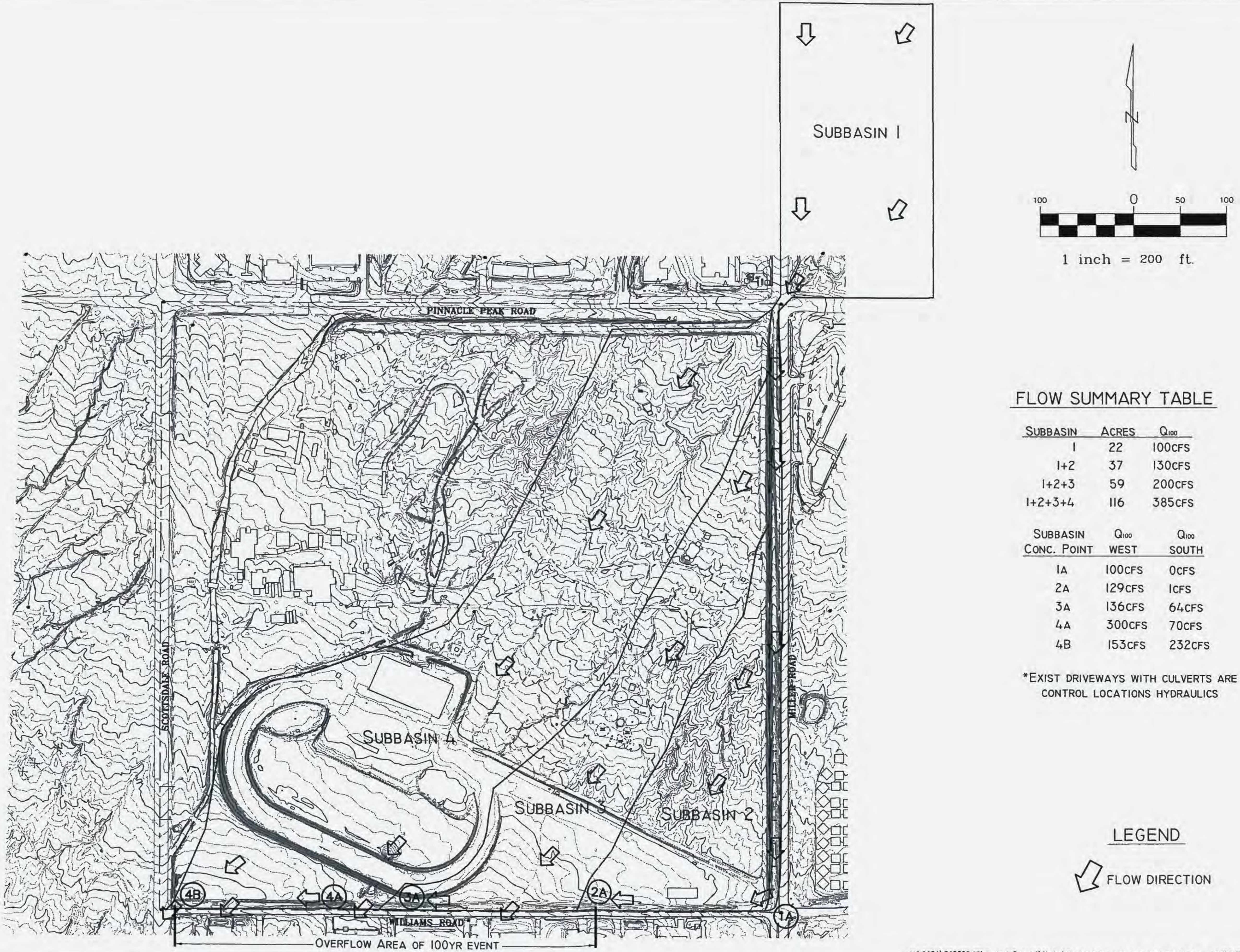
ROADWAY DRAINAGE MAP

WILLIAMS DRIVE, SCOTTSDALE, ARIZONA
SCOTTSDALE RD TO MILLER RD

Wood, Patel & Associates, Inc.
2051 West Northern, Suite 100
Phoenix, Arizona 85021 (602) 335-8500

DRAWN BY: DF
JOB NO: WP# 042309

PLATE 3



FLOW SUMMARY TABLE

SUBBASIN	ACRES	Q_{100}
1	22	100CFS
1+2	37	130CFS
1+2+3	59	200CFS
1+2+3+4	116	385CFS

SUBBASIN CONC. POINT	Q_{100} WEST	Q_{100} SOUTH
1A	100CFS	0CFS
2A	129CFS	1CFS
3A	136CFS	64CFS
4A	300CFS	70CFS
4B	153CFS	232CFS

*EXIST DRIVEWAYS WITH CULVERTS ARE CONTROL LOCATIONS HYDRAULICS

LEGEND

FLOW DIRECTION

N:\2004\042309.10\Project Support\Hydro\Exhibits\Drainage Map_Williams Dr.dwg 10/26/2006 11:19

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LAND DEVELOPMENT • WATER RESOURCES
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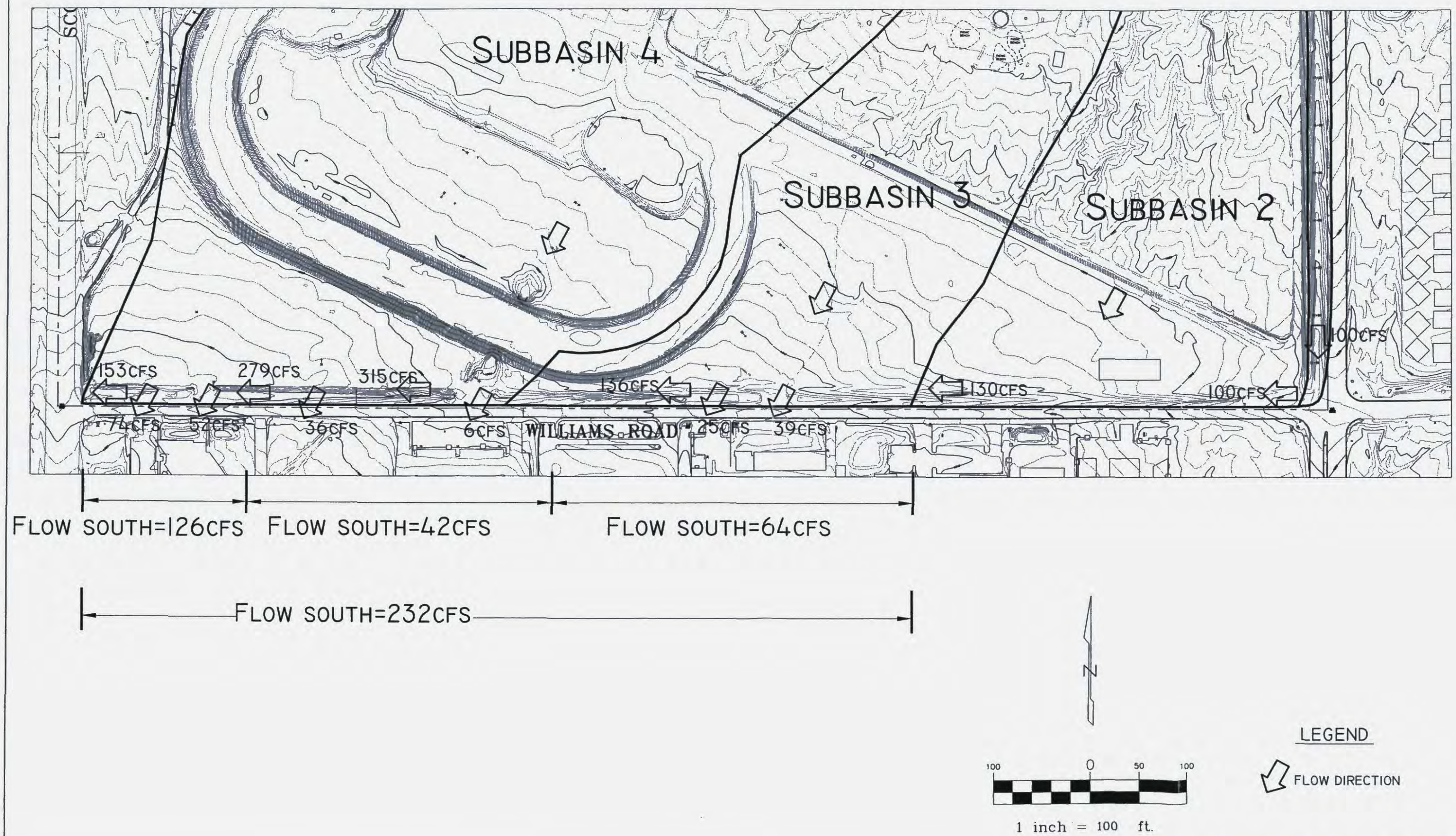
CONSTRUCTION MANAGEMENT
2051 W. Northern Ave.
Phoenix, AZ 85021
(602) 335-8500
www.woodpatel.com

SILVERSTONE
SCOTTSDALE
PRE-EXISTING DRAINAGE EXHIBIT

PRELIMINARY
NOT
FOR
CONSTRUCTION
OR RECORDING

DRAWN	
CHECKED	DF
DATE	25 MAY 06
SCALE	1" = 100'
JOB NO.	042309.10
SHEET	1OF 2



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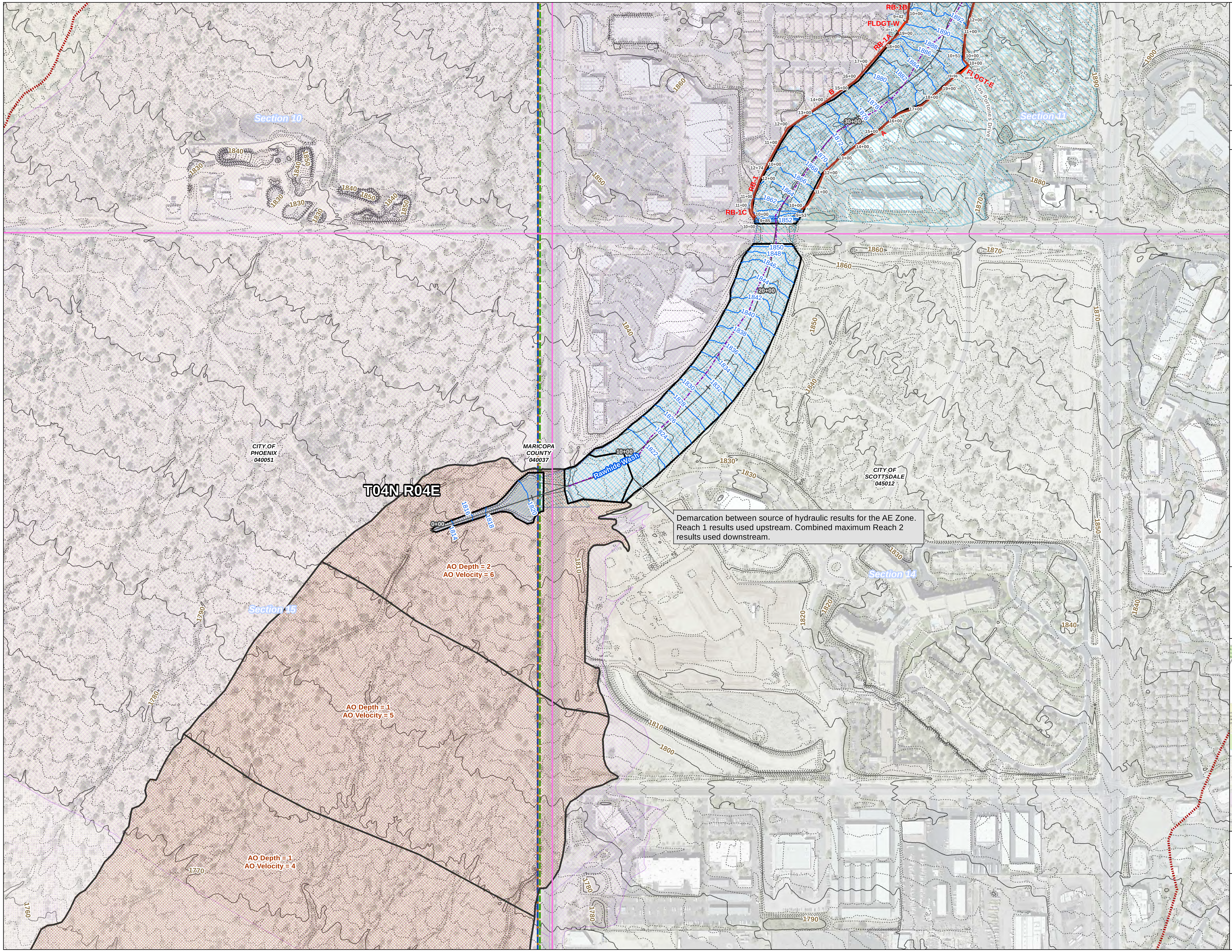
OR RECORDING

DRAWN	
CHECKED	DF
DATE	25 MAY 06
SCALE	1" = 100'
JOB NO.	042309.10
SHEET	2 OF 2

APPENDIX I

Excerpts from FCDMC CLOMR Application



Flood Control District of Maricopa County, City of Phoenix, and City of Scottsdale

RAWHIDE WASH FLOOD HAZARD MITIGATION FINAL DESIGN CLOMR CONTRACT NUMBER 2018C015

Exhibit F

LEGEND

- Limit of Study
- Base Flood Elevations
- Levee Alignments (with Stations)
- Studied FIS Streamline
- HEC-RAS Thalweg (for Reference)
- Limit of Revision
- Sections
- Township/Range

Contour Data

- Major (10-ft)
- Minor (2-ft)
- HVR Grading Contours (2-ft)

Jurisdictional Boundaries

- Phoenix
- Scottsdale
- Maricopa County

Revised Riverine Floodzones

- AE

Revised Ponding Floodzones

- AH

Revised Alluvial Fan Floodzones

- AO with Velocity

Effective FEMA Riverine Floodzones

- A
- Floodways
- AE

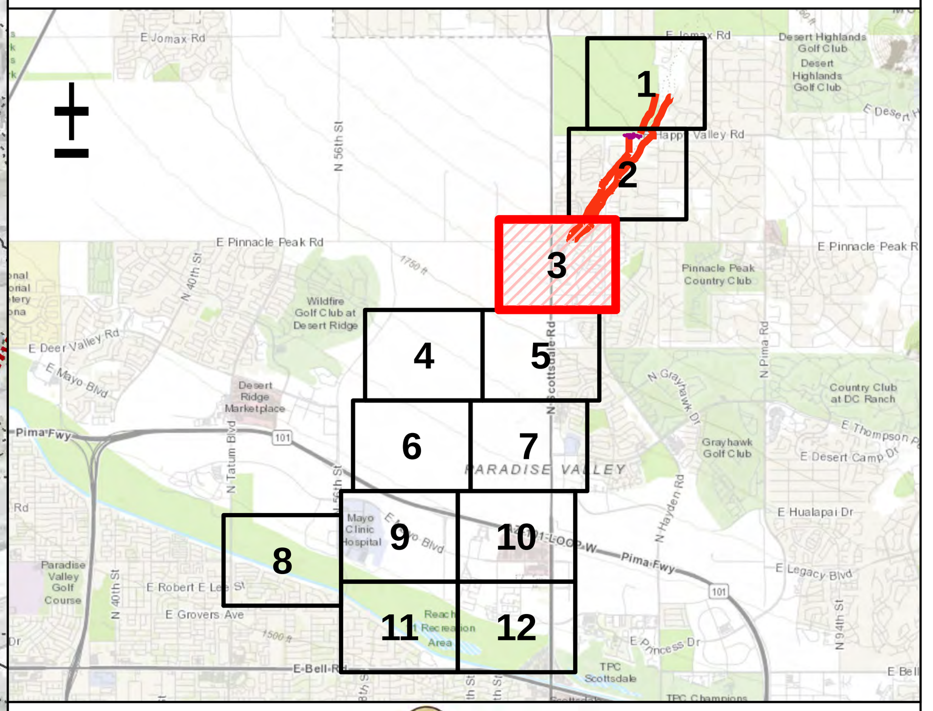
Effective FEMA Alluvial Fan Floodzones

- AO, Depth 3 ft, Velocity 7 ft/s
- AO, Depth 1 ft, Velocity 5 ft/s
- AO, Depth 1 ft, Velocity 4 ft/s
- AO, Depth 1 ft, Velocity 3 ft/s

NOTES

1. SECTION/TOWNSHIP RANGE BOUNDARIES PROVIDED BY MARICOPA COUNTY
2. HORIZONTAL PROJECTION IS ARIZONA STATE PLANE, CENTRAL ZONE, NORTH AMERICAN DATUM OF 1983 (FEET).
3. ALL ELEVATIONS ARE BASED ON NORTH AMERICAN VERTICAL DATUM OF 1988 (FEET).
4. CONTOUR DATA GENERATED FROM COMBINED TOPOGRAPHIC DATA FOR DISPLAY PURPOSES (SEE TSDN FOR MORE INFORMATION).
5. CONTOUR DATA IS 2-FOOT CONTOUR INTERVAL. MAJOR CONTOUR INTERVAL IS 10-FOOT.

SHEET INDEX MAP



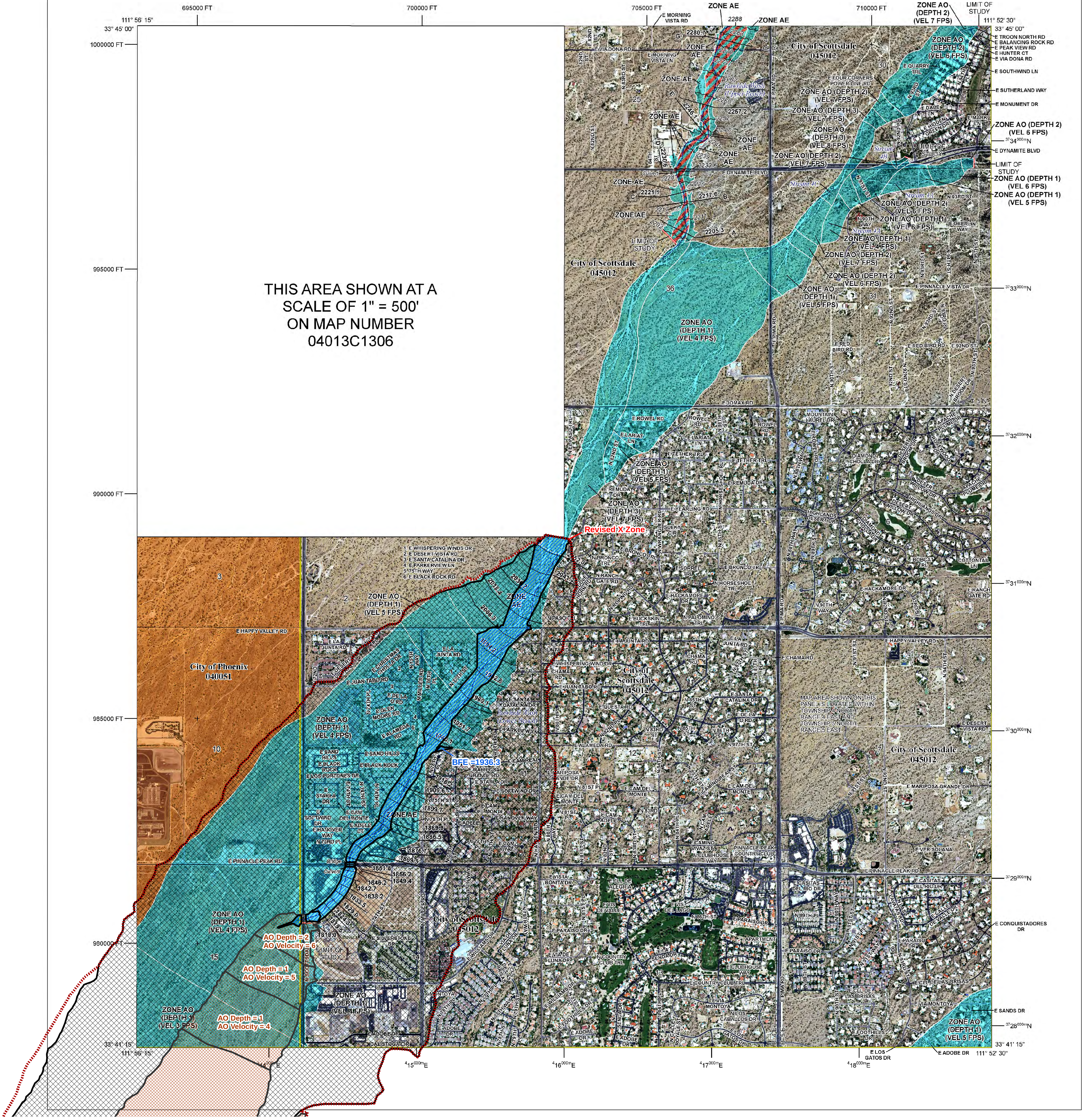
JE FULLER

Registered Professional Engineer (Civil)
CERTIFICATE NO. 52467
RICHARD M. WASKOWSKY
Date Signed 1-30-2024
ARIZONA, U.S.A.

0 200 400 Feet

Design by:	CC	SUBMITTED: January 2024
Checked by:	RW/WSO	SHEET 3 OF 12

Print Date: 1/30/2024



FLOOD HAZARD INFORMATION

SEE FIS REPORT FOR ZONE DESCRIPTIONS AND INDEX MAP

THE INFORMATION DEPICTED ON THIS MAP AND SUPPORTING DOCUMENTATION ARE ALSO AVAILABLE IN DIGITAL FORMAT AT

HTTPS://MSC.FEMA.GOV

SPECIAL FLOOD HAZARD AREAS	Without Base Flood Elevation (BFE) *Zone A,V, A99* With BFE or Depth *Zone AE, AO, AH, VE, AR*
	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee See Notes. *Zone X*
	Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN Area of Minimal Flood Hazard *Zone X*
	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
	E 18.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
	8 Coastal Transect
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
	513 Base Flood Elevation Line (BFE)
OTHER FEATURES	Limit of Study
	Jurisdiction Boundary

NOTES TO USERS

For information and questions about this Flood Insurance Rate Map (FIRM), available products associated with this FIRM, including historic versions, the current map date for each FIRM panel, how to order products, or the National Flood Insurance Program (NFIP) in general, please call the FEMA Map Information eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Flood Map Service Center website at https://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website.

Communities annexing land on adjacent FIRM panels must obtain a current copy of the adjacent panel as well as the current FIRM Index. These may be ordered directly from the Flood Map Service Center at the number listed above.

For community and countywide map dates refer to the Flood Insurance Study Report for this jurisdiction.

To determine if flood insurance is available in this community, contact your Insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

Base map information shown on this FIRM was provided in digital format by the United States Geological Survey (USGS). This information was derived from digital orthophotography at a 5-meter resolution from photography dated 2017.

LEGEND FOR

Limit of Revision

Revised Riverine Floodzones

AE

Revised Alluvial Fan Floodzones

AO with Velocity

Added X Floodzone (Due to Levee)

X, AREA WITH REDUCED FLOOD RISK DUE TO LEVEE

Revised X Floodzone

X, 1 PCT DEPTH LESS THAN 1 FOOT

SCALE

Map Projection:
NAD 1983 StatePlane Arizona Central FIPS 0202 Feet;
Western Hemisphere: Vertical Datum: NAVD 88

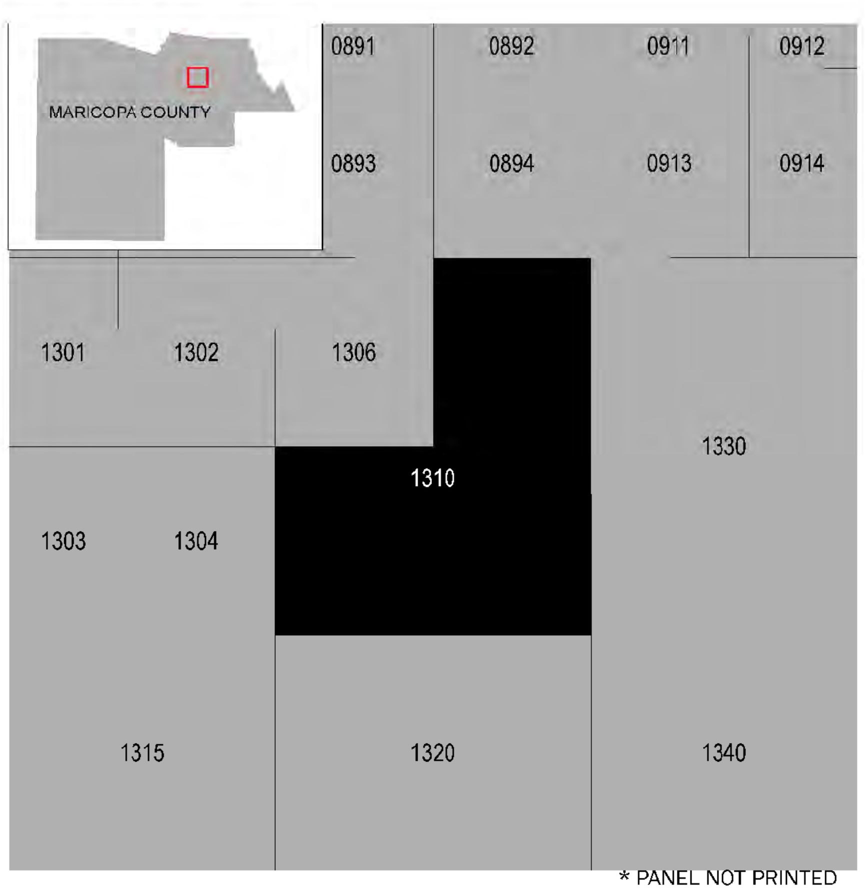
N

1 inch = 1,000 feet 1:12,000

0 1,000 2,000 4,000 Feet

0 250 500 1,000 Meters

PANEL LOCATOR



* PANEL NOT PRINTED

FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM
FLOOD INSURANCE RATE MAP

MARICOPA COUNTY, ARIZONA
And Incorporated Areas

PANEL **1310** OF **4425**

FEMA

Panel Contains:

COMMUNITY	NUMBER	PANEL	SUFFIX
PHOENIX, CITY OF	040051	1310	M
SCOTTSDALE, CITY OF	045012	1310	M

ANNOTATED: February 2023

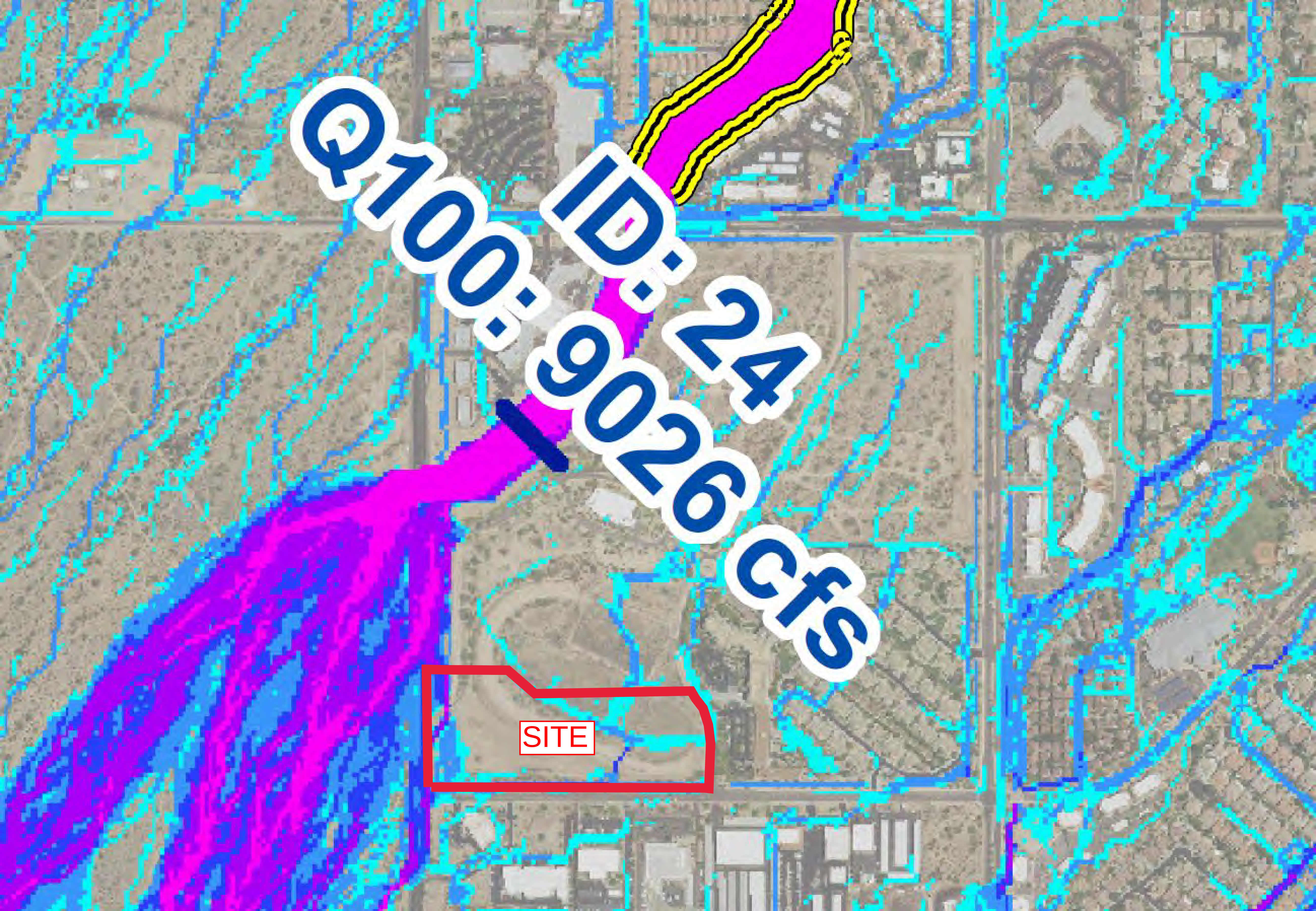
VERSION NUMBER
2.3.3.4

MAP NUMBER
04013C1310M

MAP REVISED
JULY 20, 2021

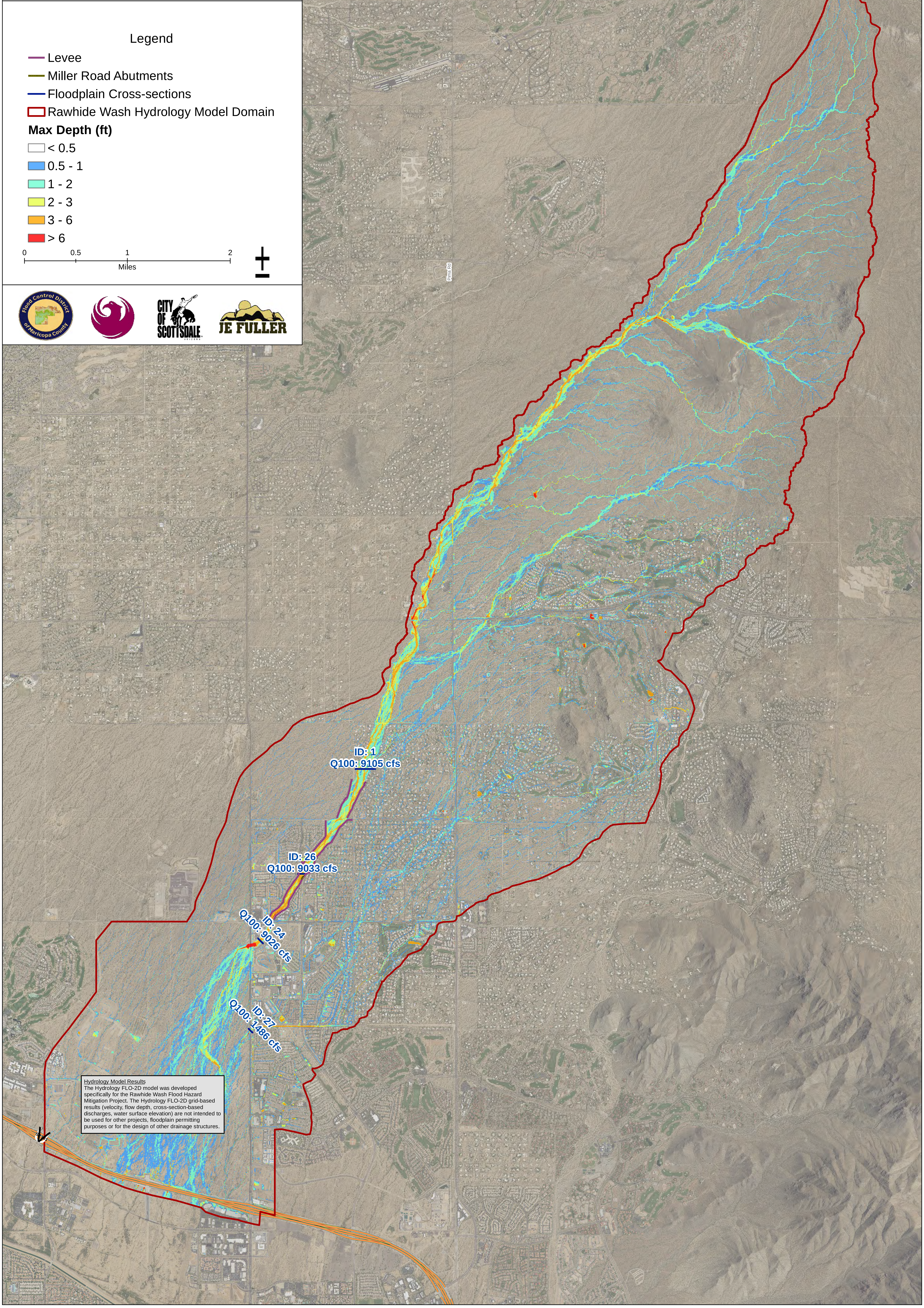
Rawhide Wash Flood Hazard Mitigation
Final Design CLOMR

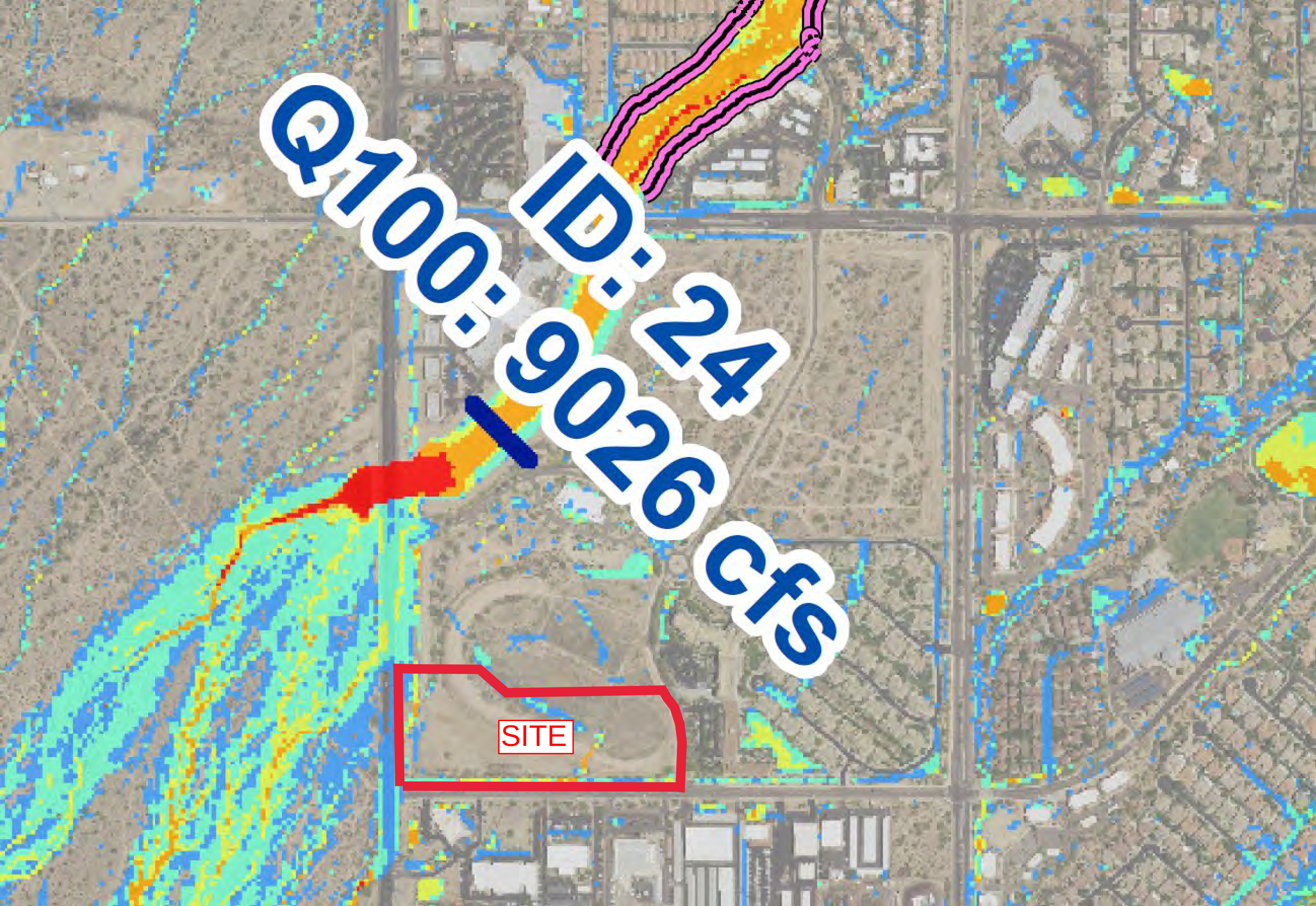
Exhibit D.2
100-yr 24-hr Maximum Discharge (Proposed)


ID: 24
Q100: 9026 cfs
SITE

Rawhide Wash Flood Hazard Mitigation
Final Design CLOMR

Exhibit E.2
100-yr 24-hr Maximum Depth (Proposed)




ID: 24
Q100: 9026 cfs
SITE

Rawhide Wash
Flood Hazard Mitigation

Conditional Letter of Map Revision
Technical Study Data Notebook

FCD 2018C015, Work Assignment No. 2

December 2021

Revised March 2023

Prepared for:

Flood Control District of Maricopa County

Flood Control District of Maricopa County
2801 West Durango Street
Phoenix, AZ 85009
(602) 506-1501

Prepared by:

JE FULLER
define•communicate•solve

JE Fuller / Hydrology and Geomorphology, Inc.
8400 South Kyrene Road, Suite 201
Tempe, Arizona 85284
(480) 752-2124

Registered Professional Engineer (Civil)
CERTIFICATE NO.
52467
RICHARD M.
WASKOWSKY
Date Signed 3-4-2023
ARIZONA, U. S. A.

W. Scott Ogden

Registered Professional Engineer (Civil)
CERTIFICATE NO.
28857
W. SCOTT
OGDEN
Date Signed 3-4-2023
ARIZONA, U.S.A.

Table of Contents

List of Figures

List of Tables

List of Appendices

List of Abbreviations

ADMS – Area Drainage Master Study

ALERT – Automated Local Evaluation in Real Time

ARF – Area Reduction Factor

CAP – Central Arizona Project

DDM – Drainage Design Manual (of Maricopa County)

DDMS – Drainage Design Management System

FCDMC – Flood Control District of Maricopa County

FDS – Floodplain Delineation Study

FLO-2D – two-dimensional rainfall-runoff model developed by Dr. Jim O'Brien

FEMA – Federal Emergency Management Agency

HEC-1 – HEC-1 Flood Hydrograph Package

MAG – Maricopa Association of Governments

NID – National Inventory of Dams

NFIP – National Flood Insurance Program

NAVD 88 – North American Vertical Datum of 1988

NOAA – National Oceanic and Atmospheric Administration

NVP – Natural Valley Procedure

PPS – Pinnacle Peak South

PPW – Pinnacle Peak West

TSDN – Technical Support Data Notebook

TDS – Terrain Data Set

USACE – United States Army Corps of Engineers

USBR – United States Bureau of Reclamation

USGS – United States Geological Survey

1 Introduction

1.1 Purpose

The purpose of this study is to develop new floodplain maps for Rawhide Wash from its alluvial fan apex between Jomax and Happy Valley Roads in Scottsdale, Arizona to the Central Arizona Project (CAP) Reach 11 Dike, which was built to protect the Hayden-Rhodes Aqueduct (see Figure 1-1). This study will update the floodplain maps using new hydrology and hydraulics based on the FEMA certification of the Rawhide Wash Levees and the proposed Miller Road Bridge. The final design of the levees is documented in the *Rawhide Wash Flood Hazard Mitigation Final Design Report* (Design Data Report) (JEF, 2023a). The final construction plans for the proposed Miller Road Bridge are included with the CLOMR digital data in Appendix G.

1.2 Authority for Study

This study was authorized under contract FCD2018C015, Work Assignment No. 2 for the Flood Control District of Maricopa County (FCDMC).

1.3 Location

The study is located in the watershed of Rawhide Wash and its tributaries that drain portions of Scottsdale and Phoenix in northern Maricopa County, Arizona. The studied portion of Rawhide Wash ranges from just upstream of the apex of the alluvial fan landform about ½ mile north of Happy Valley Road to the CAP Reach 11 Dike. A vicinity map is shown as Figure 1-1.

1.4 Brief Statement of Methods

Rainfall-runoff modeling was performed to compute 100-year discharges for Rawhide Wash using FLO-2D PRO software. FLO-2D Pro is approved by FEMA for hydrologic and hydraulic analyses for National Flood Insurance Program (NFIP) studies in Maricopa County. The hydrologic methods used in this study follow the methods and procedures outlined in the Drainage Design Manual (DDM) for Maricopa County, Arizona – Hydrology. Rainfall data for the 100-year, 6-hour and 24-hour duration storm events come from NOAA Atlas 14. Rainfall losses were computed using the Green-Ampt method with a limiting depth applied in FLO-2D.

FLO-2D PRO was used to define the hydraulics for the leveed reach of Rawhide Wash (Reach 1), the reaches of Rawhide Wash between Scottsdale Road and the CAP ponding area (Reaches 2 and 3), while the ponding area itself was delineated with HEC-1 modeling (Reach 4).

To perform all this modeling, an overall FLO-2D model was used to define the hydrology used by the project. This model is denoted with the prefix, “Rawhide,” and used a 20-foot cell size. Three other FLO-2D models were developed to define the detailed hydraulics for Reaches 1, 2, and 3. These models all used a 10-foot cell size. The inflows for Reaches 1 and 2 were taken from the overall hydrology model, while the maximum outflow hydrographs from the L101 culverts from the Reach 2 model were used as inflows to Reach 3 for one scenario. Another scenario was developed that used the maximum outflow hydrographs scaled to the design flow of each L101 culvert. However, the hydraulic results from the Reach 2 and Reach 3 models were only used to guide the areal extents of the revised alluvial fan floodplain that is located downstream of the leveed reach (i.e., Reach 1). The model domains for the models are shown in Figure 1-1.

1.5 Brief Description of Results

The resulting levee certification package and revised regulatory floodplain delineations are documented herein.

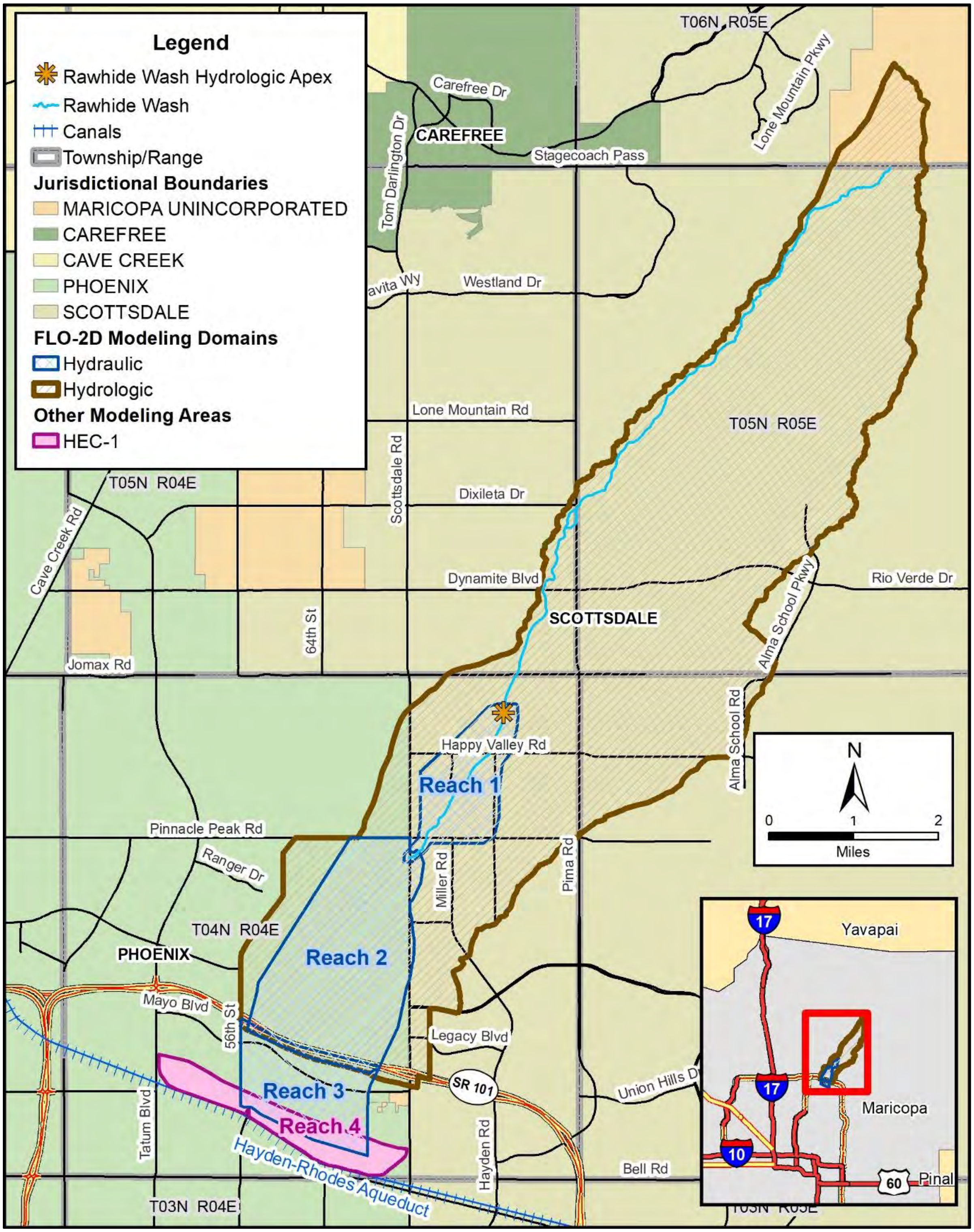


Figure 1-1. Vicinity Map

2 FEMA Forms

2.1 Overview and Concurrence Form (Form 1) ESA Documentation

Since this is a CLOMR, the requisite ESA compliance documentation is provided in Appendix B.

2.1.1 City of Scottsdale Form

2.1.1.1 Part D

Full Community Official Title is: Drainage and Flood Control Program Manager, Floodplain Administrator
Full Mailing Address is: Stormwater Management; Attn: Floodplain Administrator
7447 E Indian School Rd, Ste 125
Scottsdale, AZ 85251

2.2 Riverine Structures Form (Form 3) Explanations

2.2.1 Section B

2.2.1.1 Part 4.

Duplicate Effective Model – Natural Run: RW_SilverstoneLOMR_Asb.prj
Existing or Pre-Project Conditions Model – Natural Run: Rawhide_Reach1_100Y24H_Existing (FLO-2D Folder)
Revised or Post-Project Conditions Model – Natural Run: Rawhide_Reach1_100Y24H_Proposed (FLO-2D Folder)

2.2.2 Section C

Both the Pinnacle Peak Road and Scottsdale Road Bridges have been previously modeled and approved by FEMA into the effective FIS for Rawhide Wash. The as-built drawings for both structures are provided again with this CLOMR submittal in Appendix G. The Miller Road Bridge (MRB) is new and the plans for that structure are also provided in Appendix G. It is noted that all hydrology, hydraulic, sediment transport and scour analysis for the MRB are documented in the Design Data Report (JEF, 2023)

2.2.3 Section E. Levee/Floodwall

The project levees are composed of a combination of existing masonry and cast-in-place concrete floodwalls being retrofitted to meet FEMA standards, new masonry and cast-in-place floodwalls, two short reaches of new earthen berms immediately upstream and downstream of Happy Valley Road, FloodBreak floodgate closures at Los Portones Drive, and a raised section of Happy Valley Road with a temporary freeboard enclosure.

Three levee systems are defined for the Rawhide Wash Flood Hazard Mitigation Project (Project) as follows:

- Rawhide Wash West Bank Levee System (RW-WB)
- Rawhide Wash East Bank South Levee System (RW-EBS)
- Rawhide Wash East Bank North Levee System (RW-EBN)

Separate MT-2 Forms are provided for each system, although in most cases, the data entry for each levee system is identical due to universal design standards being applied for the Project. Explanations in this section will be detailed by each system as appropriate.

Throughout the forms, references are made to technical reports prepared to support the levee system design. ***All of the analyses and results presented in those documents represent hydrology for a future***

condition watershed that includes a yet to be designed and constructed levee system that will likely increase existing condition 100-year flows by about 340 cfs at the Rawhide Wash apex. When reviewing these documents, it is important to keep this distinction in mind. Floodplain related water surface elevations and freeboard elevations summarized herein, and on the MT-2 forms, is for existing watershed conditions mapped with this proposed CLOMR. The design scour and sediment transport analyses will not be re-analyzed for the existing condition CLOMR discharges and are deemed to be conservative.

The specifically referenced technical reports are as follows and listed in Appendix A:

- **Design Data Report (JEF, 2023a)** – this report compiles and summarizes all the design related documents, with various technical reports included as appendices. Relevant appended reports include the following:
  - **APPENDIX D: Hydrology, Hydraulics, Sediment Transport and Scour (HHSTS) (JEF, 2023b)** – this report presents and summarizes all the design HHSTS analyses including determination of design water surface elevations, freeboard elevations, scour depths, interior drainage analyses, and sediment transport analyses.
  - **APPENDIX F: Geotechnical Reports (N&M, 2021a, 2021b, and 2022)** – The primary report and two addendums address the geotechnical analyses performed to evaluate the existing and proposed embankment slope stability, foundations, seepage and seismic stability, as well as construction earthwork guidance and design.
  - **APPENDIX H: Structural Reports (CEC, 2021a and 2021b)** – These two reports summarize the existing and proposed floodwall structural and stability analyses. The existing floodwalls proposed for retrofit were evaluated for existing condition static (no loading), wind loading, and seismic static. Each wall was then modified to reflect the post-constructed geometry and then evaluated for 100-year flood loading and wind loading. The proposed floodwalls were evaluated for static, wind, flood and seismic.

The existing and proposed floodwalls and embankments are referred to by name throughout the design documents. Designations (A, B, C-1, etc.) are shown below in Figure 2-1 and on the Work Study maps of Exhibit F, the design and construction documents, and on maps and figures within the various reports.

The following are a subsection-by-subsection summary of the data entries needed for Section E not explicitly added to the form itself

2.2.3.1 Part 1b

Note that the stationing shown is based on the project thalweg with 0+00 being the downstream end of the Project HEC-RAS and HEC-6T modeling. The stationed thalweg alignment is shown on the Work Study maps for Reach 1 and the station labeled cross sections are shown in reference to the various levee system elements on the on the *Rawhide Wash Flood Hazard Mitigation Project Levee System Index Map* and *Rawhide Wash Flood Hazard Mitigation Project Levee System Plan and Profile Maps* in Appendix H. The system of floodwalls begins just upstream of Pinnacle Peak Road at river station 23+75 and end approximately 0.5 miles north of Happy Valley Road at river station 109+88.

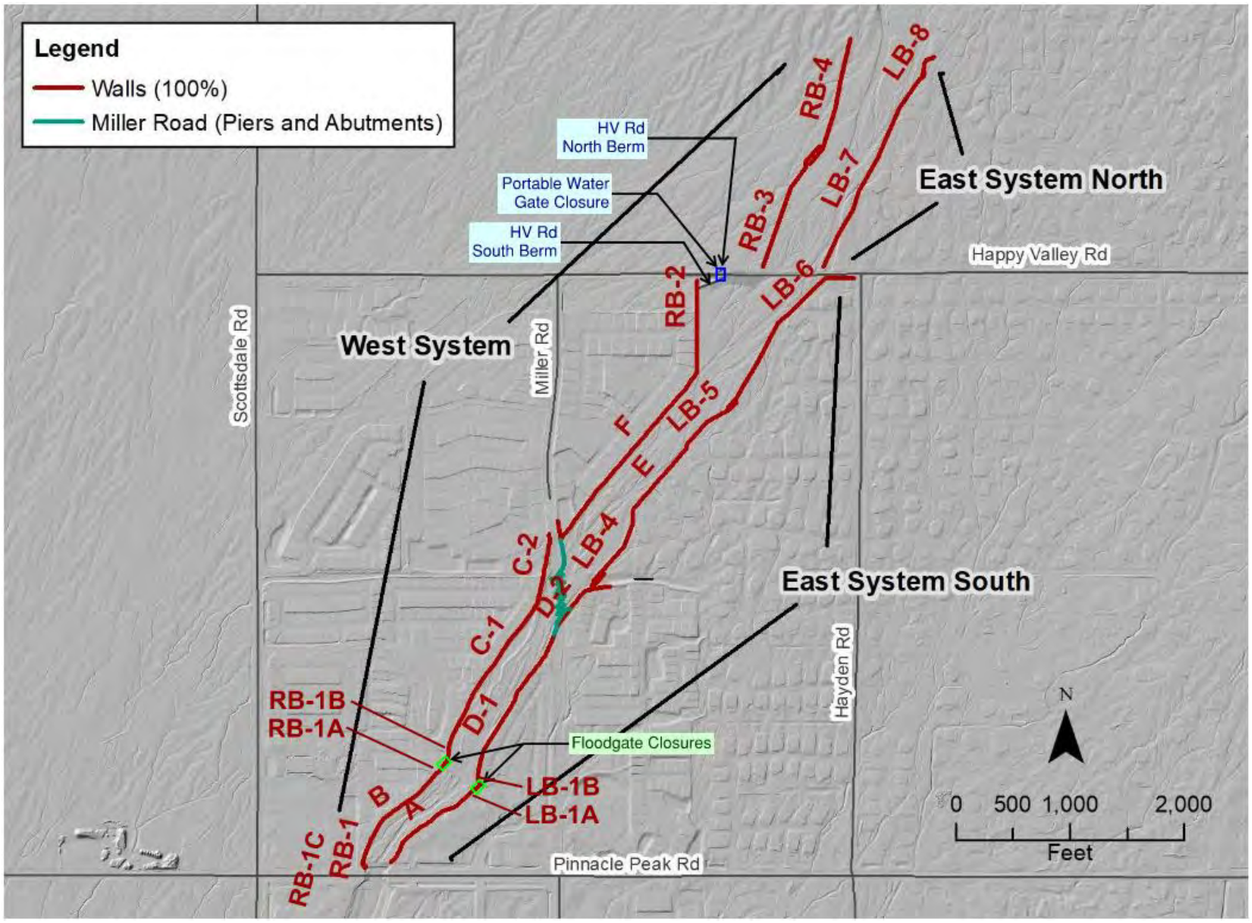


Figure 2-1. Map of Wall, Berm and Closure Designations

2.2.3.2 Part 1e

The full set of Rawhide Wash Flood Hazard Mitigation Project design plans (Phase 1 and 2) and MRB Plans all detail the various requested elements. As noted above the design water surface, freeboard, and scour profiles are all for design purposes. The CLOMR 100-year water surface elevation and freeboard have been plotted to the design plan and profile sheets that are included in Appendix H. As an aide to the review, the CLOMR BFEs and Freeboard Elevations for each river station bank (right-west or left-east) are tabularized with the floodwall description and construction control station, plan sheet number, and design top of structure elevation, and included in Appendix H.

2.2.3.3 Part 3a

The Project has three total closures. Two are located along the RW-WB system and one is located on the RW-EBS system. There are no closures for the RW-EBN system. The RW-WB system closures are located at Los Portones Drive and Happy Valley Road. The Los Portones Drive closure will be an automatically deployed floodgates designed and distributed by FloodBreak. The floodgates will also include a manual hydraulic lift as a back-up system per 44 CFR 65.10 requirements. The same FloodBreak floodgate system will be used for the RW-EBS Los Portones Drive opening. The second closure for RW-WB will at the raised portion of Happy Valley Road. The closure location is elevated above the BFE, but will rely on the deployment of portable water gates distributed by QuickDam for any freeboard needs. The specified water gate is designed to self-rise to a full height of 50-inches, which is greater than the required 3.5-foot (42-inches) of freeboard at that location. Plans, specifications, and data sheets for the FloodBreak floodgates and the QuickDam water gates are included in Appendix I of the Design Data Report (JEF, 2023a).

2.2.3.4 Part 4

The Project embankments and flood side of the floodwall system are all protected with either existing, augmented, or new gabion mattress and baskets that are generally buried below the surface. The proposed MRB improvements are also protected by gabions, although the pier and abutments are founded on drilled shafts designed to withstand the full scour and maintain structural integrity of the bridge. The erosion protection will be constructed or augmented to protect to the full design 100-year scour depth. There is no loose riprap proposed for erosion protection of the floodwalls or other levee system elements. A complete listing of scour depth calculations correlated to the thalweg stations is provided in Section 6 and Appendix C of the HHSTS Report (JEF, 2023b). Full details of the design are shown on the Project construction plans.

2.2.3.5 Part 5

The analysis of embankment slope and foundation stability were performed at 42 critical sections within the project that include both existing and proposed floodwall locations. The analyses and results are presented in Section 9 of the final Geotechnical Report (N&M, 2021a). Although most levee protection is via floodwalls, several of those existing floodwalls are located on compacted embankments. A total of 5 cases (End of Construction, Rapid Drawdown from Static Flooding, Rapid Drawdown from Transient Flooding, Steady State Seepage, and Pseudo-Static Seismic) were analyzed at all 42 locations. Table 9 of that report presents stability results and Table 10 summarizes the foundational bearing capacity and properties of soil backfill. The analysis concluded that the post-project condition slope and foundation stability all meet or exceed the required factors of safety.

It is noted that there are no analysis for the berms located immediately upstream and downstream of Happy Valley Road. This is primarily due to the fact that the berms are primarily constructed to provide freeboard connectivity between Wall RB-2 and the Happy Valley Road closure, and from the closure north to tie into dry ground and prevent any breakout along the north side of Happy Valley Road. The depth of 100-year flooding against the embanked portion of the berms is less than 1-foot with the rest serving as freeboard. The berm construction will follow geotechnical recommendations with a minimum compaction density of 95% and 3H to 1V side slopes and is therefore certified to meet the FEMA requirements.

2.2.3.6 Part 6

Detailed calculations and summaries of the floodwall and foundation stability for post-project conditions are primarily addressed in the final Structural Reports (CEC, 2021a & 2021b) included with the CLOMR package as a part of the Design Data Report Appendix H. All four loading conditions were evaluated for the existing floodwalls in pre- and post-project conditions (CEC, 2021a) and except for Wall E, all results equaled or exceeded the minimum factor of safety criteria. It is noted that Wall H is no longer part of the levee system due to other project constraints. Summaries of these results can be seen in Tables 6-1, 6-2, and 6-3 of that report. Wall E will be mitigated with stem strengtheners to bring that wall into compliance. Details for that mitigation are shown on the plans.

New cast-in-place concrete and masonry floodwalls are all designed to meet or exceed the required factors of safety for all four loading conditions (CEC, 2021b). Structural calculations and analyses for overturning and sliding, and design maximum foundation pressures are provided in the Structural Design report (CEC, 2021b). Allowable foundation bearing strengths tested by N&M range between 1,250 and 3,000 psf, with a summary included in the Geotechnical Report, Table 10 (N&M, 2021a). To achieve design minimums and mitigate the potential for differential settlement to acceptable tolerances, N&M recommended certain foundation areas to be over-excavated and compacted back to 95% density. Those limits are shown Figures 5A-5K of the report and Addendum No. 1 (N&M, 2021b) provides further guidance on Wall LB-6 near Happy Valley Road.

2.2.3.7 Part 7c

The Happy Valley Road embankment levee foundations will be over-excavated by 12-inches and compacted back to full 95% density to mitigate the possibility of foundation consolidation. Further consolidation is expected to be minimal and less than 1-inch per the Geotechnical Report (N&M, 2021a). No other embankment levees are included in the system for certification.

2.2.3.8 Part 8

Each of the five significant interior drainage areas are briefly discussed in this TSDN. Detailed design calculations are presented in the Design Data Report and HHSTS Report (JEF, 2023a and 2023b) and the construction drawings and specifications.

For Part 8c, the flow duration curve is essentially the hydrograph from the Post-Project Conditions FLO-2D model (folder name = Rawhide_Reach1_100Y24H_Proposed). See the hydrograph for FPXSEC Nos. 1 and 2, which bracket the approximate apex location.

For Part 8e, all interior drainage areas were analyzed using either direct modeling in the 100-year Post-Project FLO-2D model or were evaluated assuming a 100-year interior drainage flood that is coincident with a 10-year flood in Rawhide Wash. Details for these evaluations are summarized in the HHSTS Report (JEF, 2023a). There are no historical records to evaluate ponding probabilities and no coastal influences.

2.2.3.9 Part 10

The Operations and Maintenance Plan for the Project is included in the Design Data Report, Appendix J (JEF, 2023a)

2.2.4 Section F. Sediment Transport

A full sediment transport analysis has been performed for the design project (Reach 1) and is documented in the Section 5 of the HHSTS Report (JEF, 2023b) included with this submittal. It is noted that he HEC-6T modeling extends significantly north of the Project boundaries to attempt to allow the HEC-6T model to normalize prior to reaching the design reach. Sediment transport modeling was conducted for three separate scenarios. Two consider the design 100-year flows with and without supercritical flows allowed (HEC-6T's $SCRT=OFF or ON). The third model considers a random series of differing magnitude storms for a 50-year period to develop a long-term evaluation. The sediment volume is dependent on which of three scenarios is being considere. For the single event models, the transported volumes range between 6 and 13 ac-ft from upstream to downstream. The 50-year volume ranges between 15 and 25 ac-ft. The sediment concentration varies depending on the storm time and location in the reach and is not typically reported.

3 Survey and Mapping Information

3.1 Digital Projection Information

A Terrain Data Set (TDS) was developed using ArcGIS software tools to incorporate the mapping data as described in Section 3.3. The TDS was built with the following projection information:

- Vertical Datum: The North American Vertical Datum of 1988 (NAVD 88)
- Horizontal Datum: North American Datum of 1983 (NAD 1983), High Accuracy Reference Network (HARN), Arizona State Plane Central Coordinates, International Feet.

3.2 Field Survey Information

Field survey was conducted during the Pinnacle Peak West (PPW) Area Drainage Master Study (ADMS) (JEF, 2014). Within the Rawhide Wash Hydrology model domain, this survey was limited to hydraulic structures that can significantly affect flow characteristics such as culverts and engineered channels. The horizontal position of structures surveyed was obtained from a Trimble Juno 3D Handheld GPS (Juno 3D). The Juno 3D is a Roving GPS unit with a horizontal accuracy of less than one meter. The vertical accuracy of the Juno 3D is not sufficient for hydraulic modeling purposes; therefore, the results of this 2014 field survey were only used to refine the hydrology modeling.

3.3 Mapping

The aerial mapping data for this CLOMR study came from multiple mapping sources (see Table 3-1). The data for most mapping sources were provided by the District in the form of mass-point and break-line data with the only exceptions being the 2016 and 2020 Rawhide Wash LiDAR data. The 2016 data was provided in the original LAS format and was converted to a small cell raster, and the 2020 data was provided as a bare earth small cell raster.

The mapping data was used to develop the TDS which was in turn used to develop the FLO-2D grid cell elevations; the methodology for this process is discussed in more detail in Section 4. In areas where multiple mapping data sets overlap, the data with the most recent flight date was used to develop the TDS. The project name and the detailed information for each mapping data set are listed in Table 3-1 below, while the spatial extents are shown in Figure 3-1. All mapping used for this study meets the minimum FEMA mapping criteria for NFIP studies.

Table 3-1. Mapping Data Information

Project	Mapping ID	FCDMC Contract No.	Contour Interval	Flight Date	Vertical Datum
Pinnacle Peak North	1310	10-26	2-foot	11/2/2007	NAVD88
Pinnacle Peak South	1309	10-26	2-foot	11/02 & 11/03/2007	NAVD88
Camp Creek Mapping	1227	01-52	2-foot	4/27/2003	NAVD88
Scottsdale Mapping	1071	IGA 93-07	2-foot	9/1/1993 & 12/27/2000	NAVD88
Pinnacle Peak ADMS	1311	09-44	2-foot	6/28/2010	NAVD88
Rawhide Wash LiDAR	1412-306	2016C014	1-foot	10/19/2016	NAVD 88
Rawhide Wash Flood Hazard Mitigation Update LiDAR Mapping	-	2019C005	1-foot	1/14/2020	NAVD 88

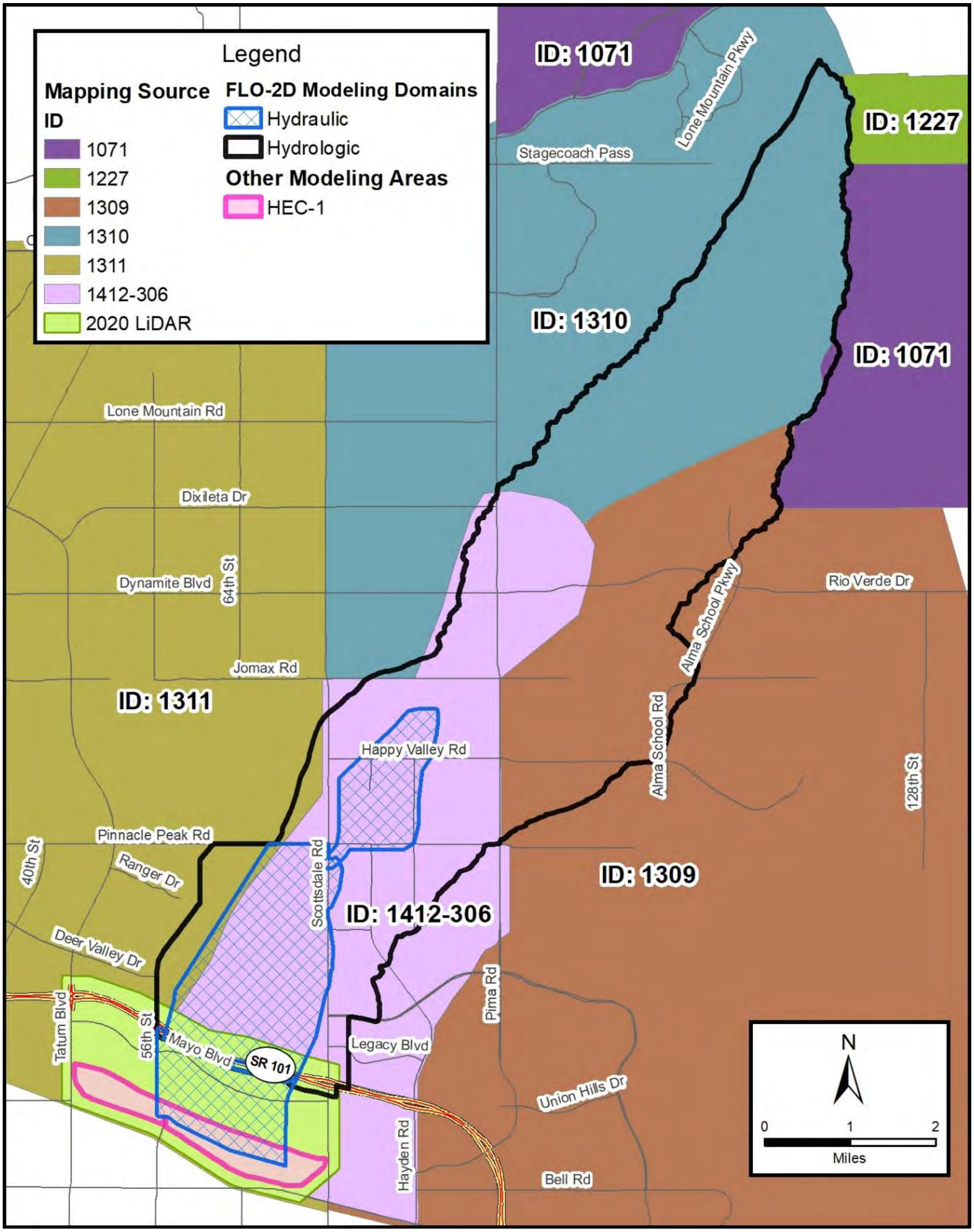


Figure 3-1. Mapping Data Sets

4 Hydrology

4.1 Method Description

The 100-year hydrology for Rawhide Wash was computed using a rainfall-runoff modeling approach with a single software package - FLO-2D. FLO-2D is a dynamic two-dimensional hydrologic and hydraulic model that conserves volume as it routes hydrographs over a system of square grid cells. The FLO-2D version used for this study is the Pro Version Build No. 16.06.16 with an executable dated February 28, 2017. This version has been approved and used by the FCDMC for other projects. The modeling procedures for the current CLOMR study followed: 1) those presented in the Drainage Design Manual (DDM) for Maricopa County – Hydrology (2018a); and 2) those used in the approved hydrology CLOMR, FEMA Case Number: 18-09-1616R (JEF, 2018).

4.2 Parameter Estimation

4.2.1 Drainage Area Boundaries

The FLO-2D model domain was set based on a review of previous studies and topographic maps of the area to ensure computation of runoff from all areas that could potentially drain to Rawhide Wash. Internal drainage area boundaries are implied based on the accumulation of runoff across the grid.

4.2.1.1 Grid Cell Size (CADPTS.DAT)

The FLO-2D surface is represented as a grid comprised of square cells that route the computed runoff and subsequent flood wave over the watershed's topographic surface. The grid cell size selected for the FLO-2D model measured 20 feet by 20 feet. The total model domain is 29.8 square miles. Therefore, with a grid size of 20 feet, the total number of grid cells modeled is 2,081,647 for the hydrology model.

4.2.1.2 Assignment of Elevation to the Grid (FPLAIN.DAT elevations)

The elevation data for the FLO-2D grid was developed starting with a TDS generated from the aerial mapping data supplied by the District (See Section 3.3). The TDS was converted to a 20-foot pixel raster using built-in ArcGIS software routines at the full resolution of the TDS. The "center" of each raster pixel was located at the exact same X-Y coordinates as the FLO-2D CADPTS.DAT input file (file relating the grid Cell ID to the X-Y location) for each sub-area model. The elevation data from the raster was written to the FPLAIN.DAT input file (file containing the Cell ID, elevation, and Manning's n-value data) for each pixel located in the Reach 1 hydrology FLO-2D model domain.

4.2.1.1 Minor Adjustment to Grid Cells

Minor adjustments were made to the averaged grid cell parameters for two reasons:

1) To mimic the actual slope near a culvert; and,
2) To reduce model run times due to excessive timestep decrements.

For 1), the elevations were adjusted at culvert structure locations to stabilize the model hydraulic structure computations by representing the true slope at these culvert inlets/outlets. The grid cell elevation at a structure inlet/outlet are taken from point elevations of the estimated culvert inverts and, in some cases, are lower than adjacent grid elevations. This is primarily due to the elevation averaging of a 20-foot grid relative to a specific point elevation of a culvert inlet or outlet.

For 2), areas such as retention basins that contain a large depth of ponded water can cause model runtimes to become extremely long due to excessive time step decrements. A relatively high Manning's n value is assigned to these areas to alleviate this issue. The initial n value used for deep ponding area grid cells was assigned per guidance in the *Drainage Policies and Standards for Maricopa County Supplemental Technical Documentation, FLO-2D Verification Report* (FCDMC, 2016) and the refined FCDMC table of ponded n values (see Table 4-1). One ponding area used a slightly higher n value than what is shown in the table to reduce high velocities.

Table 4-1. Recommended n Values for Various Ponded Depths

Depth (feet)	Manning's n Value
0 - 5	No adjustment
5 - 8	0.08
8 - 10	0.10
10 - 15	0.20
15 - 20	0.30
20 - 25	0.40
> 25	0.50

4.2.2 Watershed Work Maps

The following watershed work maps were developed to accompany this report:

- General Watershed Map (Topographic Contours and Spatially Varied Elevations)
- Soils Map
- Land Use (Surface Features) Map
- 100-year, 24-hour Maximum Discharge Map (Existing and Proposed)
- 100-year, 24-hour Maximum Depth Map (Existing and Proposed)

The source, derivation, and application of each of these parameters to the hydrologic modeling are discussed in later sections of this report. Large-scale maps for the modeled watershed are provided electronically with the digital files accompanying this report.

4.2.3 Gage Data

Rawhide Wash has one runoff gaging station (current ID: 61007, previous ID: 4863) located on its main stem on the downstream side of Dynamite Boulevard (see Figure 4-1). The drainage area at the gage is about 8.9 square miles, and the gage has been in operation for 24 years. Table 4-2 shows the annual peak discharges recorded by the station since 1999. Of the 24 years of record, during 13 of these years no flow was recorded. The minimum detection level is about 0.5 feet of depth which corresponds to a flow rate of about 40 cfs. The largest flow recorded at the gage was about 450 cfs which occurred on September 9, 2006. Due to the large number of zero flow years in the record, the effective record for statistical analyses using the Bulletin 17C approach is very short (effective record length = 24 – 13 = 11 years). Hence, meaningful statistical analyses are not possible due to the very short effective record length, and any statistical results are subject to such significant uncertainty as to be largely uninformative. Therefore, these statistics were not used to derive any peak flow information and are presented for information only.

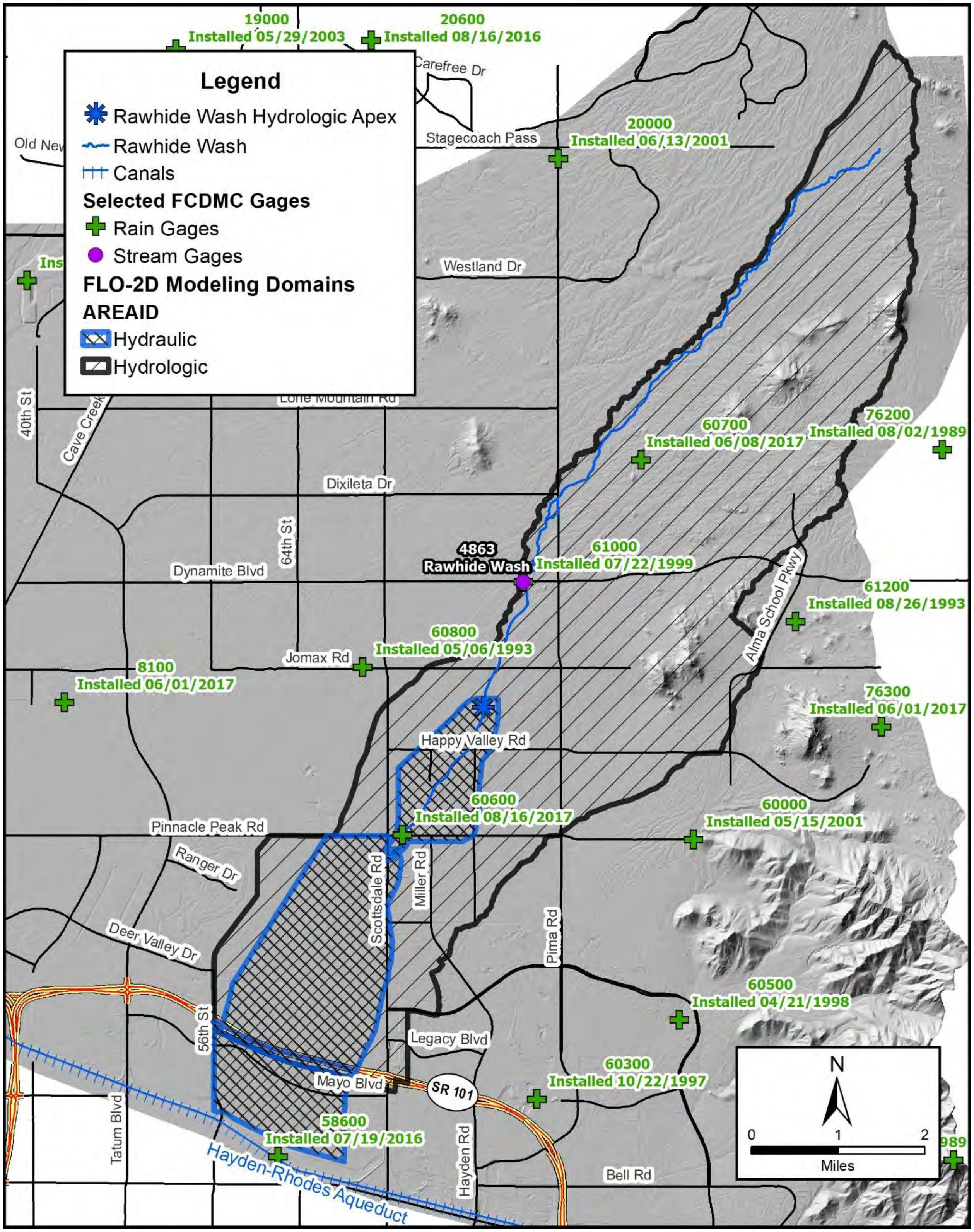


Figure 4-1. Location of Rawhide Wash Stream Gage and nearby FCDMC Rain Gages in the Watershed

Table 4-2. Annual Peak Discharges at FCD Gage #61007 - Rawhide Wash[1]

Water Year	Peak Gage Height (feet)	Peak Discharge (cfs)	Date of Peak
1999	None	0	None
2000	None	0	None
2001	None	0	None
2002	1.04	82	7/14/2002
2003	None	0	None
2004	1.07	89	3/5/2004
2005	0.67	47	8/2/2005
2006	2.07	446	9/9/2006
2007	1.92	382	7/31/2007
2008	None	0	None
2009	None	0	None
2010	None	0	None
2011	None	0	None
2012	None	0	None
2013	None	0	None
2014	1.67	283	9/8/2014
2015	0.74	47	7/31/2015
2016	1.87	361	8/5/2016
2017	None	0	None
2018	None	0	None
2019	0.79	48	10/07/2018
2020	None	0	None
2021	0.66	42	8/13/2021
2022	0.99	131	7/30/2022

There are also some precipitation gages within the watershed and in the vicinity. The location at Dynamite Road also has a precipitation gage (ID: 61000). There are two other precipitation gages within the watershed: 1) Pinnacle Peak Powerline (ID: 60700); and 2) Rawhide Wash at Pinnacle Peak Road (ID: 60600). However, neither of these gages have significant records since they were recently installed in 2017. Five others in the surrounding area have a much longer record. These are:

1) Fraesfield Mountain (ID: 76200),
2) Stagecoach Wash (ID: 20000),

[1] http://alert.fcd.maricopa.gov/alert/Flow/61007.htm

3) Reata Pass Dam (61200),
4) Jomax Road at 70th Street (ID: 60800), and
5) Reata Pass Wash (ID: 60000).

Table 4-3 shows the largest 6-hour and 24-hour rainfall totals recorded at each of these five stations (and the Rawhide Wash rain gage) along with its period of record. Note that the data in this table are based on the statistics generated by the FCDMC and only include up to water year 2018.

The largest events at any station in terms of return period occurred on July 31, 2007 and September 8, 2014 with 6-hour return periods of 37 and 31 years, respectively. The next largest storm occurred on November 30, 2007. None of these events produced significant runoff at the Rawhide Wash stream gage at Dynamite Boulevard.

Table 4-3. Maximum Precipitation Events near Rawhide Wash Watershed

Gage ID	Period of Record (years)	6-hr Max (inches)	Date	Return Period (years)	24-hr Max (inches)	Date	Return Period (years)
61000	22	2.44	9/8/2014	31	2.80	10/2/2018	9
76200	31	2.60	8/5/2016	28	4.09	11/30/2007	27
20000	20	2.68	7/31/2007	37	3.07	11/30/2007	11
61200	28	2.20	9/8/2014	14	2.76	10/2/2018	6
60800	28	2.32	9/8/2014	25	3.11	10/2/2018	21
60000	20	2.24	9/8/2014	19	2.95	10/2/2018	11

4.2.4 Statistical Parameters

As indicated in the previous section, one stream gage exists on Rawhide Wash with a 22-year period of record. However, during 13 of those years, the gage did not record any flows above the detection limit of 40 cfs. Hence, the effective gage record length is insufficient to compute statistically significant runoff statistics. Therefore, these statistics were not used to derive any peak flow information and are presented for information only. Thus, the NOAA Atlas 14 rainfall statistics for this study were derived from FCDMC's DDMSW software, as described below.

4.2.5 Precipitation (RAIN.DAT)

Based on the results of the hydrology CLOMR (JEF, 2018), the 100-year, 24-hour storm is the controlling event for this watershed. Therefore, this duration was the only one modeled. NOAA Atlas 14 as coded into DDMSW software was used to obtain the 100-year 24-hour rainfall depths.

Since there is variation of the rainfall across the model domain, spatially varied rainfall was modeled for the entire study area using the NOAA Atlas 14 rainfall statistics at each grid. This was accomplished by selecting the maximum point-precipitation depth for the model domain and assigning a reduction factor (RAINARF) to the remaining grid cells in the domain based on a percentage of the maximum. Rainfall depths and the associated reduction factors are provided in the FLO-2D RAIN.DAT input file. The selection of the maximum point-precipitation depth and subsequent rainfall reduction factor was conducted for the 100-year, 24-hour event.

This reduction should not be confused with traditional rainfall areal reduction. Rainfall areal reduction was not used for this study to ensure conservative flood hazard model results. The SCS Type II rainfall pattern was used for the 24-hour event per the DDM for Maricopa County, Arizona – Hydrology. Table 4-4 shows the maximum point 100-year rainfall depth for the 100-year 24-hour event.

The DDMSW rainfall data is in a gridded format and along the edges of each rainfall depth grid, the 20-foot FLO-2D grids were area-weighted to compute the point rainfall depth on those FLO-2D grids that intersect multiple DDMSW rainfall depth grids.

Table 4-4. Rawhide Wash Watershed Maximum 100-year Point Rainfall

Storm Event	NOAA Atlas 14 Maximum Point Rainfall Depth (inches)	Storm Pattern
100-year, 24-hour	5.501	SCS Type II

4.2.6 Physical Parameters

4.2.6.1 Rainfall Losses (INFIL.DAT)

Rainfall losses were computed using the Green-Ampt method as implemented in FLO-2D. Infiltration parameters were computed using the DDM methods. The loss parameters are a function of NRCS soil type and land uses. Values for initial abstraction (IA), soil moisture deficit (DTHETA), soil suction head (PSIF), saturated conductivity (XKSAT), and percent effective impervious area (RTIMP) were derived from FCDMC-provided GIS shapefiles processed for the model domain.

<u>Soils Data</u>

The soil data used for most of the study area is from the USDA Natural Resource Conservation Service (NRCS) soil survey data as provided by the District. The data is dated April 2010, and the watershed is mostly covered by the Aguila-Carefree Area survey #AZ645 (NRCS Soils Data). The northeastern corner of the watershed falls outside of the Aguila-Carefree survey. Therefore, the statewide STASGO database was used to determine the classification of the soil outside of the limits of the detailed NRCS soil data. The spatial extents of the areas covered by the detailed NRCS and STATSGO soil survey are shown in Figure 4-2. The detailed NRCS soils parameters are linked by the Map Unit Soil ID (MUID) to the DDMSW software program, and the soil parameters are extracted from the DDMSW database. The extracted parameters include XKSAT, PSIF, Rock Outcrop (RTIMP), and DTHETA. The DTHETA value is dependent on the initial condition based on the land use data (dry, normal, or saturated). The PSIF value and DTHETA value are assigned based on the soil XKSAT value. The assignment is based on the relationship between XKSAT and PSIF and DTHETA (dry and normal) built in to DDMSW.

Only one STATSGO soil type lies within the study area outside of the Aguila-Carefree survey – map unit S316 – Rock Outcrop-Gran-Lehmans soil complex. The Gran soil component is characterized as “very gravelly clay" or "very gravelly sandy clay". Per the District manual, an XKSAT of 0.02 in/hr is used for sandy clay. The Lehmans soil component is characterized as a "clay" or "gravelly clay". The District manual gives an XKSAT of 0.01 in/hr for clay. Since this clay soil type contains some sand but is also considered to be gravelly, the higher XKSAT of 0.02 in/hr was selected for this soil type. The rock outcrop was ignored for this soil type. The NRCS soil types and key hydrologic characteristics are presented in Table 4-5, while the spatial distribution of the soils is shown in Figure 4-3.

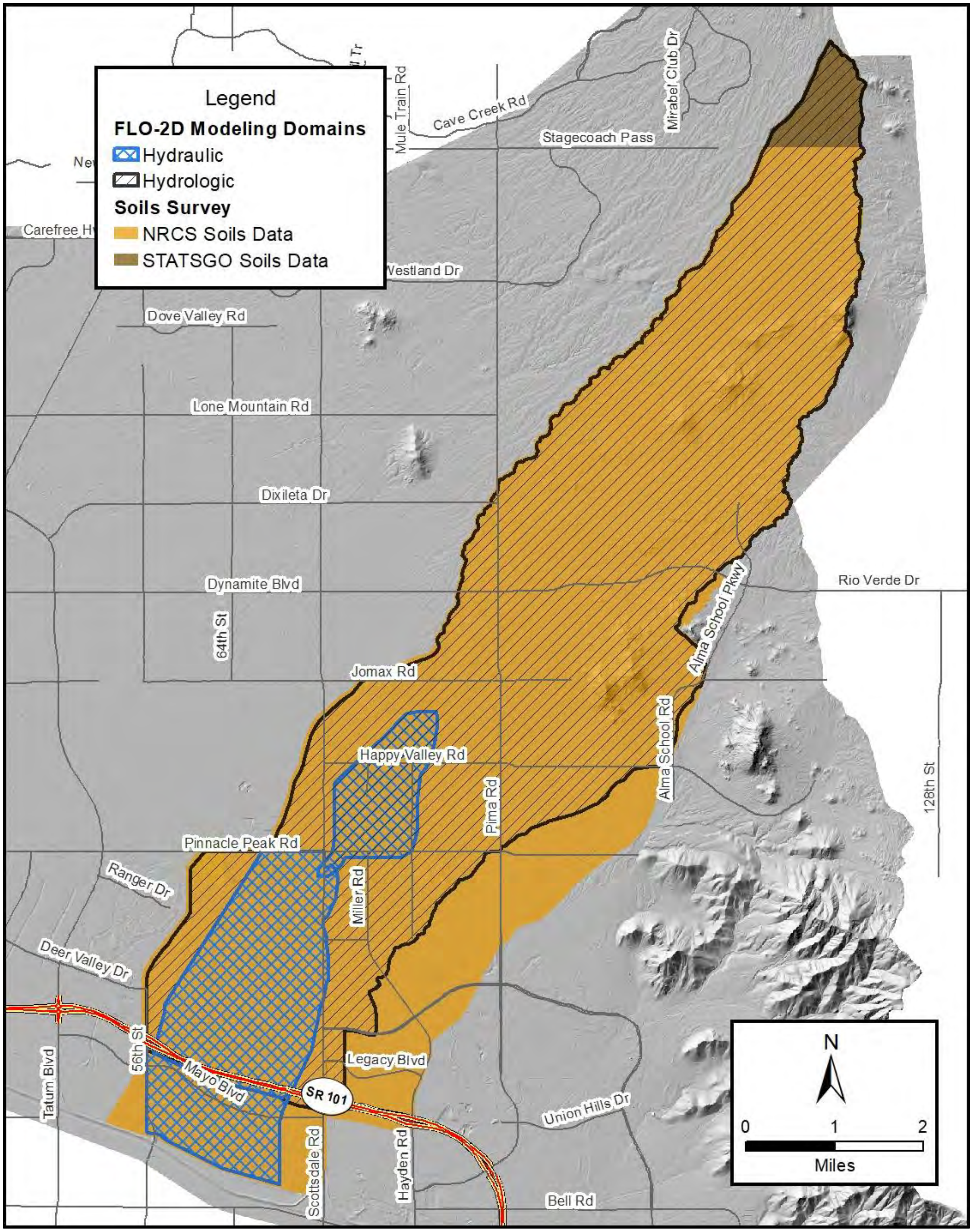


Figure 4-2. Soil Survey Coverage over the FLO-2D Model Domain

Table 4-5. Soils Data Parameters

Soil ID	Description	XKSAT (in/hr)	Rock Outcrop (%)
6451	Antho sandy loams	0.41	0
6452	Antho gravelly sandy loams	0.41	0
6453	Antho-Carrizo-Maripo complex	0.58	0
6456	Anthony-Arizo complex	0.62	0
6458	Arizo cobbly sandy loam	0.96	0
64512	Carefree cobbly clay loam, 1 to 8 percent slopes	0.01	0
64518	Cheriono-Rock outcrop complex, 5 to 60 percent slopes	0.33	15
64521	Cipriano very gravelly loam	0.38	0
64526	Continental cobbly clay loam, 1 to 8 percent slopes	0.01	0
64531	Dixaleta-Rock outcrop complex, 25 to 65 percent slopes	0.33	35
64533	Eba very gravelly loam, 1 to 8 percent slopes	0.23	0
64534	Eba very gravelly loam, 8 to 20 percent slopes	0.23	0
64540	Eba-Pinaleno complex, 3 to 20 percent slopes	0.17	0
64544	Ebon very gravelly loam, 1 to 8 percent slopes	0.03	0
64550	Estrella loams	0.26	0
64552	Gachado-Lomitas-Rock outcrop complex, 7 to 55 percent slopes	0.16	20
64554	Gila fine sandy loams	0.29	0
64555	Gilman loams	0.27	0
64560	Glenbar loams	0.26	0
64561	Gran-Wickenburg complex, 1 to 10 percent slopes	0.15	0
64563	Gran-Wickenburg-Rock outcrop complex, 1 to 7 percent slopes	0.14	25
64572	Lehmans-Rock outcrop complex, 8 to 65 percent slopes	0.09	30
64575	Mohall loam	0.23	0
64576	Mohall loam, calcareous solum	0.23	0
64577	Mohall clay loam	0.05	0
64578	Mohall clay loam, calcareous solum	0.05	0

Soil ID	Description	XKSAT (in/hr)	Rock Outcrop (%)
64590	Momoli gravelly sandy loam, 1 to 5 percent slopes	0.39	0
64591	Momoli-Carrizo complex	0.93	0
64593	Nickel-Cave complex, 8 to 30 percent slopes	0.33	0
64596	Pinaleno-Tres Hermanos complex, 1 to 10 percent slopes	0.07	0
64598	Pinamt-Tremant complex, 1 to 10 percent slopes	0.37	0
645101	Rillito loam, 0 to 3 percent slopes	0.28	0
645103	Rock outcrop-Gachado complex, 5 to 55 percent slopes	0.1	65
645110	Suncity-Cipriano complex, 1 to 7 percent slopes	0.13	0
645112	Tremant gravelly sandy loams	0.39	0
645113	Tremant gravelly loams	0.39	0
645118	Tremant-Rillito complex	0.42	0
645120	Tres Hermanos gravelly sandy loams	0.06	0
645121	Tres Hermanos-Anthony complex, 1 to 5 percent slopes	0.12	0
645122	Vado gravelly sandy loam, 1 to 5 percent slopes	0.33	0
645124	Valencia sandy loams	0.39	0
s316	Gran and Lehmans soils	0.02	0

The method of assigning the soil parameters for each grid cell was done by first area-weighting the bare ground XKSAT value of each 20-foot FLO-2D grid. The XKSAT values were not adjusted for vegetation cover to remain conservative in the infiltration estimates. Once a bare ground XKSAT value was assigned to each grid, the values of PSIF and DTHETA were selected from Figure 4.3 of the Hydrology Manual (FCDMC, 2018a), which relates the PSIF and DTHETA values to a given XKSAT value (see Figure 4-4). The DTHETA initial moisture condition (based on land use) is discussed in the following Land Use sub-section. The Percent Rock Outcrop was area-weighted for each 20-foot FLO-2D cell. However, the effective percent impervious assigned to each FLO-2D cell is also dependent on the coverage of percent impervious as it related to land use type. The Rock Outcrop percentage listed in the soils data summary (Table 4-5) is only the percent impervious as it relates to the rock outcrop per the soil unit. It is independent of the land use impervious percentage.

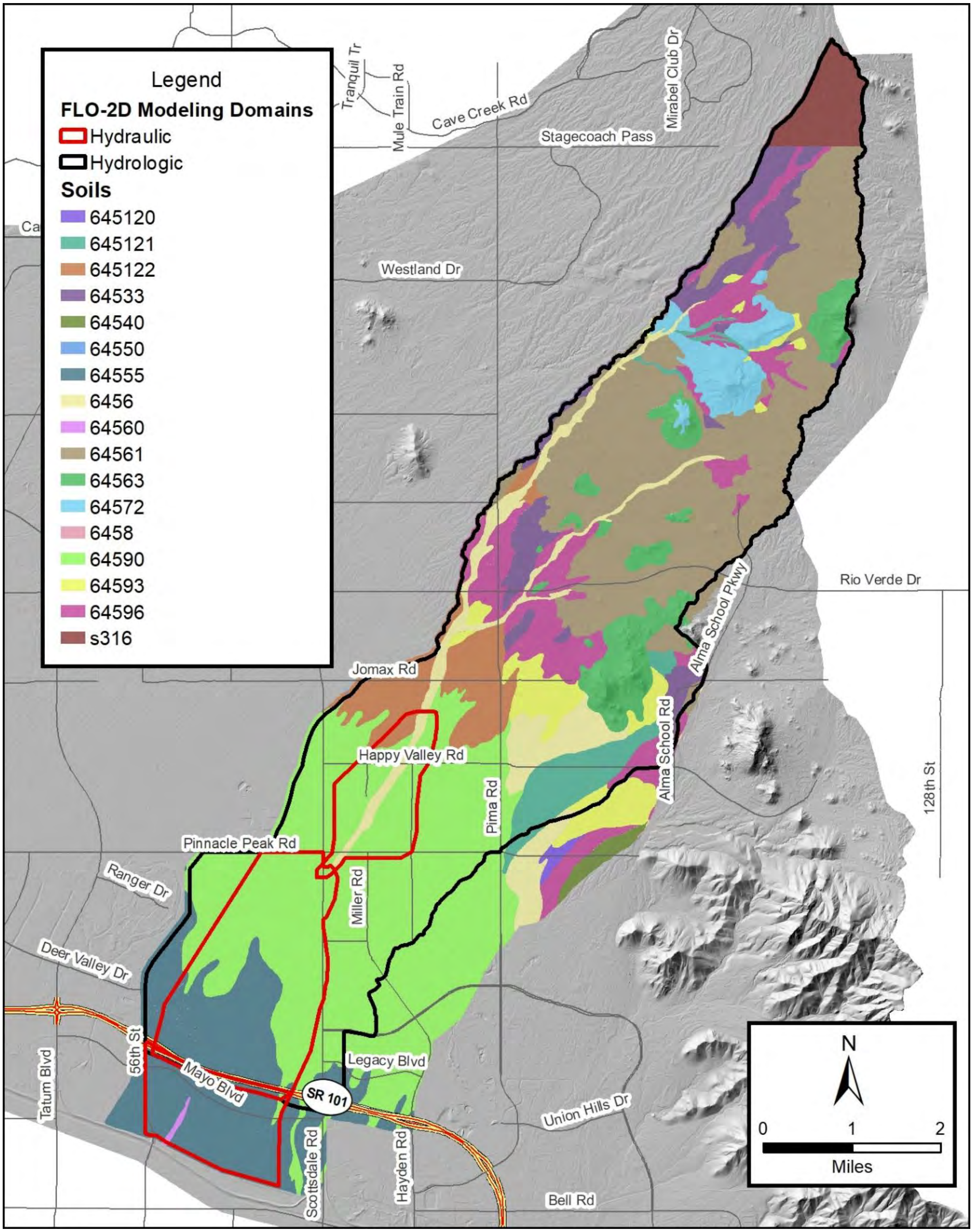


Figure 4-3. Spatial Distribution of Soils

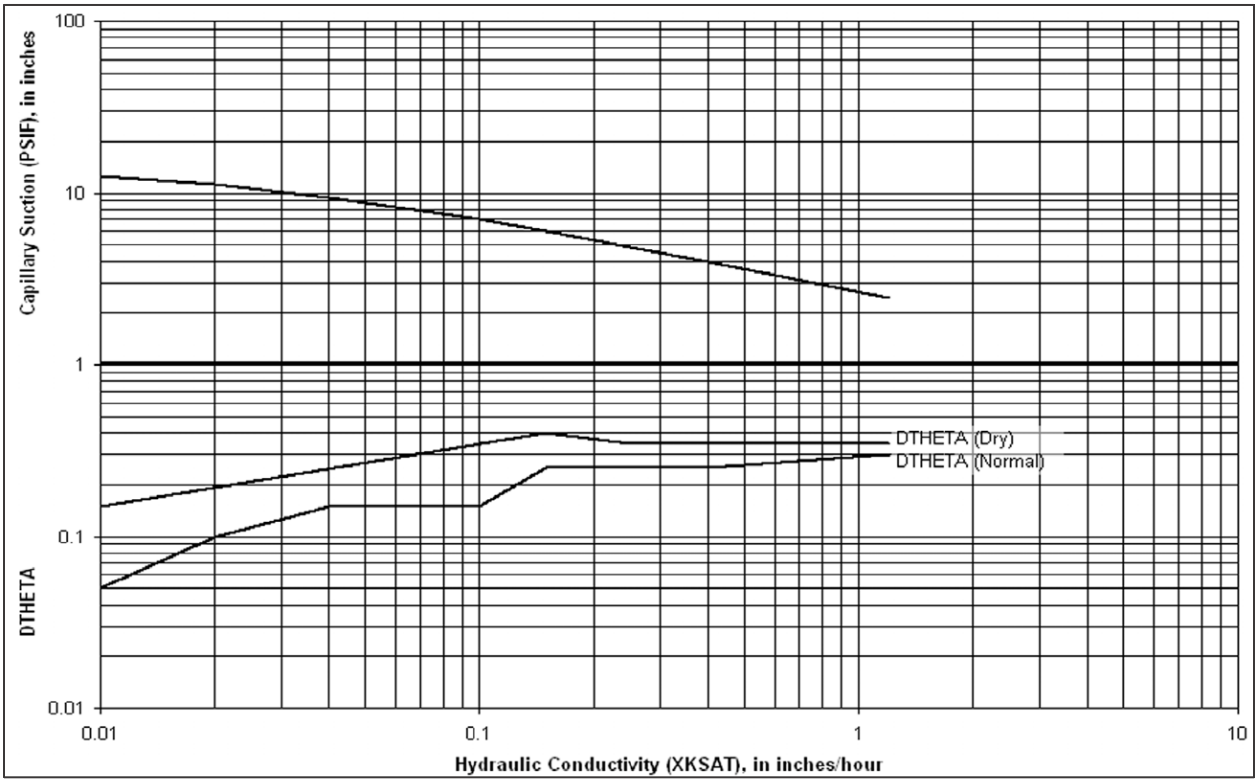


Figure 4-4. Values of PSIF and DTHETA as a Function of XKSAT, Reproduced from FCDMC (2018a).

<u>Land Use (Surface Feature Characterization)</u>

A comprehensive existing conditions land use (surface feature characterization) coverage was developed by the FCDMC based on planimetric features digitized with photogrammetry for the Pinnacle Peak West Area Drainage Master Study (ADMS) (JE Fuller, 2014) and the Pinnacle Peak South ADMS (TY Lin, 2013). Since this coverage was from earlier studies, it was reviewed and updated to match current existing conditions (as of early to mid-2020). During the time between the studies, the significant changes were limited to two areas – one near Rawhide Wash southeast of the intersection of Scottsdale and Pinnacle Peak and the other south of the Loop 101 freeway between Scottsdale Road and 56th Street. It should be noted that this second area contains ongoing construction at the time of this report, and the surface features will be updated to reflect the built condition once the final project has been built.

This coverage was used to develop the infiltration parameters related to land use, including initial abstraction (IA), Percent Impervious as it relates to land use coverage (RTIMP), and the DTHETA initial moisture condition. The GIS data consisted of surface features based on classifications of surface type (concrete, building, asphalt, etc.). This shapefile was used to assign infiltration parameters, see Table 4-6, and Manning's roughness coefficients (used for n value development in FPLAIN.DAT). Figure 4-5 shows a spatial distribution of the land use types within the FLO-2D model domains. The values assigned to each FLO-2D grid are area-weighted, except for the initial moisture condition.

The IA values listed in Table 4-6 are the full initial abstraction for each land use type. The IA values in the INFIL.DAT files are the area-weighted IA values from this Table minus the surface detention value

(TOL) from the TOLER.DAT file. The FLO-2D model control parameters, including TOL, are discussed in Section 4.2.8.

The total effective percent impervious (RTIMP) value in the FLO-2D INFIL.DAT files are based on both the percent impervious from rock outcrop from the soils data and the percent impervious from the land use coverage. Each percent impervious is independent of the other. The INFIL.DAT RTIMP is the summation of the percent impervious from the soils and land use data with a maximum value of 1.0 or 100%. For example, if a cell was located on a soils type with a rock outcrop of 20% and was also on a cell with a land use percent impervious of 30%, then the RTIMP reported to the INFIL.DAT file for that specific cell is 50%.

The initial moisture condition is related to the DTHETA value. For this study, the initial moisture condition was defined per guidance in the Hydrology Manual (FCDMC, 2018a). The “Dry” condition was applied to areas that were undeveloped and would be in a usual state of low soil moisture content, such as would occur in the desert and rangelands of Maricopa County. The “Normal” condition was assigned to developed areas where the soil would usually be in a state of moderate soil moisture, such as would occur in irrigated lawns, golf courses, parks, and irrigated pastures. The “Saturated” condition was only applied to areas that had permanent water, such as golf course lakes.

If a FLO-2D grid cell is within the “Dry” or “Normal” category, the DTHETA value is assigned to that cell based on the relationship between XKSAT and DTHETA (see Figure 4-4). If a grid is located within the saturated condition, then the DTHETA value is 0.0. The initial moisture condition is assigned based on the location of the FLO-2D grid centroid.

Finally, the limiting depth of infiltration was varied in the model area based upon the wash bottom characterization from the Surface Feature Characterization of the watershed. See the Calibration section below for a more detailed discussion.

Table 4-6. Land Use (Surface Feature Characterization) Parameters

Classification	Description	IA (inches)	Percent Impervious	Initial Moisture Condition	Manning's n-value
Asphalt	Streets and parking lots	0.05	98	Normal	0.025
Buildings	Physical structures that are flow obstructions	0.05	98	Normal	0.035
Concrete	Sidewalks, curb, patios	0.05	98	Normal	0.020
Lower Undeveloped Desert	Undeveloped areas in the lower watershed	0.35	0	Dry	0.040
Shade Structures	Parking covers, canopies	0.05	98	Normal	0.035
Unpaved Disturbed Ground	Gravel and dirt roadways/shoulders, Rough graded areas	0.10	50	Normal	0.030
Upper Undeveloped Desert	Undeveloped areas in the upper watershed	0.40	0	Dry	0.055
Urban High Vegetation	Dense trees and shrubs	0.25	0	Normal	0.060
Urban Low Vegetation	Lawns, Golf Courses, Low shrubs	0.10	0	Normal	0.030
Wash Bottom	Natural wash and river bottoms	0.10	0	Dry	0.030
Water	Lakes, canals, ponds	0.00	100	Saturated	0.020

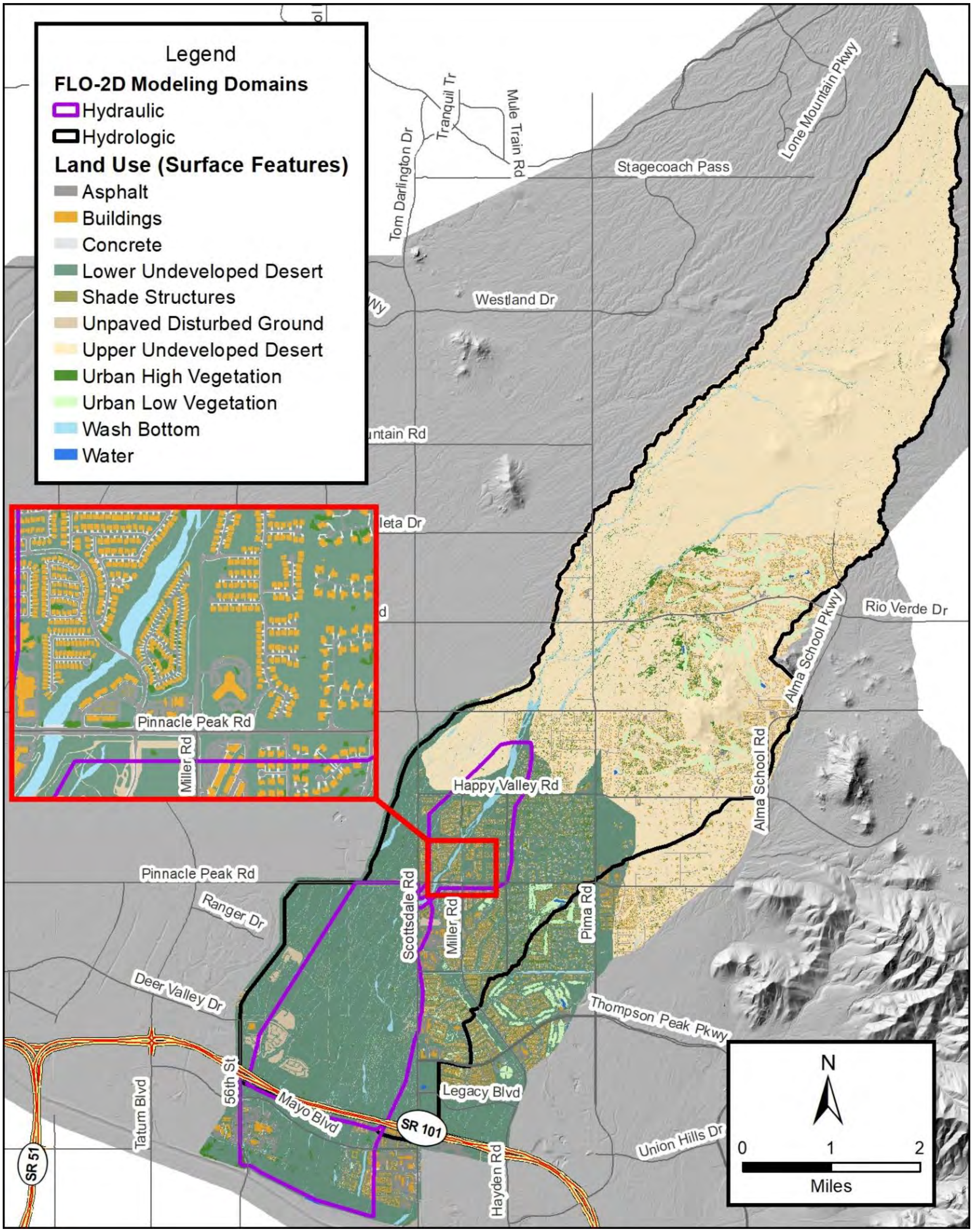


Figure 4-5. Spatial Distribution of Land Use Classifications

4.2.6.2 Unit Hydrographs
Unit hydrographs are not used in the modeling approach used for this study.

4.2.6.3 Channel Routings
Channel routings are performed explicitly across the grid as part of the FLO-2D program which accumulates and moves runoff across the surface within the model domain while conserving volume. The Manning's n values that were used in the model are listed in Table 4-6, and the values assigned to each FLO-2D grid cell were area-weighted.

4.2.6.4 Storage Routings
Storage routings are performed explicitly across the grid as part of the FLO-2D program which accumulates and moves runoff across the surface within the model domain while conserving volume.

4.2.6.5 Diversions
Flow diversions are performed explicitly across the grid as part of the FLO-2D program which accumulates and moves runoff across the surface within the model domain while conserving volume.

4.2.7 Floodplain Cross-sections (FPXSEC.DAT)
Floodplain cross-sections are locations where a flood hydrograph is written during the FLO-2D simulation. The cross-sections are placed at locations where the model stability and flood wave movement can be verified and at locations of specific hydrologic interest, such as the Rawhide Wash Apex and the outflow from the Rawhide Wash designed reach near Scottsdale Road.

4.2.8 Model Control Parameters (CONT.DAT and TOLER.DAT)
The control parameters and stability criteria that were used for the FLO-2D model are summarized in Table 4-7. The default SHALLOWN value of 0.10 was selected for the model. This global value is appropriate given the various land uses within the watershed and their associated roughness at a shallow depth. A higher SHALLOWN value has the effect of slowing down rainfall runoff producing lower discharges and runoff volumes by affecting the infiltration.

The global limiting Froude Number (FROUDL) was set to a value of 1.00 limiting the flow regime to critical to be consistent with FEMA modeling practices.

The surface detention parameter (TOL) was set to 0.048 inches (0.004 feet) to closely match the lowest initial abstraction value of 0.05 inches (0.0042 feet) shown in Table 4-6. The TOL value is subtracted from initial abstraction (IA) values in the actual INFIL.DAT file; therefore, initial IA values of 0.05 inches become 0.002 inches.

The 'Courant Only' stability criterion is used for this model. Thus, the depth tolerance criteria (DEPTOL) and dynamic wave flood routing criteria (WAVEMAX) are turned off with values of 0.00. The model files used a Courant number of 0.40 with an incremental timestep change coefficient (TIME_ACCEL) of 1.00 (default setting for this build of FLO-2D Pro). For this study, these values have produced good model stability and reasonable results.

Table 4-7. FLO-2D Model Control Parameters

CONT.DAT		
AMANN	Depth Varying Function of Roughness	0.00
SHALLOWN	Shallow Flow n-value	0.10
FROUDL	Limiting Froude Number	1.00
TOLER.DAT		
TOL	Surface Detention (ft)	0.004
DEPTOL	Depth Tolerance Stability	0.00
WAVEMAX	Dynamic Wave Flood Routing Stability	0.00
COURANTFP	Floodplain Courant values	0.40
COURANTC	Channel Courant values (not used)	0.00
TIME_ACCEL	Time Acceleration	1.00

4.2.9 Culverts (HYSTRUC.DAT)

Culverts that impacted major flow patterns were included in the FLO-2D model for Rawhide Wash as actual structures in the FLO-2D HYSTRUC.DAT file or by lowering the cells to approximate the slope of the culvert.

When entered in the FLO-2D HYSTRUC.DAT file, rating tables or the generalized culvert routine were used. Rating tables were developed from the two recent studies within the watershed – the Pinnacle Peak West and the Pinnacle Peak South Area Drainage Master studies. Tables were computed with HY-8 as part of the Pinnacle Peak West ADMS (JEF, 2014), while CulvertMaster was used in the Pinnacle Peak South ADMS (TY Lin, 2013). The rating tables from these studies were applied in the current hydrology modeling effort. The primary goal of including the culverts in the model was to ensure proper collection and distribution of flows along the drainage network within the Rawhide Wash watershed.

For large-capacity culverts or culverts that are wider than a grid cell, the rating tables were sometimes split into multiple cells to allow for more realistic conveyance of flows through the roadway crossing. Grid elevations for the inlets and outlets of each culvert were adjusted based on field survey of each structure. The rating tables and supporting output are provided in Appendix D.

In the overall hydrology model, the proposed Miller Road Bridge, the Pinnacle Peak Road Bridge, and the Scottsdale Road bridge were modeled as open channels with the width adjusted to match the outer abutments. This was done to ensure a conservative estimate of discharge through Rawhide Wash. However, these bridges were considered in the Reach 1 hydraulic model (see Section 5).

4.3 Issues Encountered During the Study

4.3.1 Special Issues and Solutions

No special issues were encountered during this study.

4.3.2 Modeling Warnings and Error Messages

The following warnings and error messages are reported in FLO-2D output files, ERROR.CHK, HYDRAULIC STRUCTURE_RUNTIME WARNINGS.OUT, FLOODPLAIN_CONVERGENCE.OUT and

DEPRESSED_ELEMENTS.OUT. Some messages are repeated multiple times if the applicable situation occurs multiple times during a single simulation. For example, the warning message appears quite often that the downstream water surface is higher than the upstream water surface for a hydraulic structure. However, each warning is only discussed once in the list below. The warnings that were reported for the final hydrology models run include the following:

ERROR.CHK Messages

- **FLO-2D Message:** THE CROSS SECTION ELEMENT: CAN ONLY BE ASSIGNED ONCE IN THE FPXSEC.DAT FILE

  JEF Explanation: This warning occurs when a grid cell is specified in two (or more) floodplain cross-sections. Since a reasonable hydrograph was generated for the cross-sections and the cross-sections were located where a hydrograph was desired, no action was taken to eliminate this message.

- **FLO-2D Message:** WARNING: THE IMPERVIOUS AREA REPRESENTED BY THE RTIMP PERCENTAGE IS LESS THAN THE ARF VALUE FOR AT LEAST ONE GRID ELEMENT.THE IMPERVIOUS AREA ASSIGNED BY THE RTIMP VARIABLE MUST INCLUDE THE BUILDING AREA, STREET AND ALL OTHER IMPERVIOUS AREAS WITHIN THE GRID ELEMENT.IF THE RTIMP PARAMETER IS LESS THAN THE BUILDING ARF VALUE, YOU MAY HAVE GLOBALLYUNDERESTIMATED THE RTIMP PARAMETER.FOR THIS SIMULATION THE RTIMP IS RESET TO THE ARF VALUE, HOWEVER, YOU SHOULD REVIEW ALL THE RTIMP ASSIGNMENTS.

  JEF Explanation: This message occurs because the maximum RTIMP assigned to grid cells in the INFIL.DAT file is 98 percent for impervious surfaces (e.g., roof tops, concrete). However, FLO-2D assigns an RTIMP of 100 percent to grid cells that have an ARF value of 1.0 (completely blocked) at runtime and there is currently no control for this. Therefore, a slight increase in rainfall runoff will occur from roofs for example. This error is considered conservative but will likely be imperceptible in the model results.

- **FLO-2D Message:** THE INITIAL ABSTRACTION VALUE IS GREATER THAN THE TOL VALUE (DEPRESSION STORAGE) FOR AT LEAST ONE GRID ELEMENT. CONSIDER (NOT REQUIRED) LOWERING THE TOL VALUE OR ADJUSTING THE IA VALUE.

  JEF Explanation: This general warning occurs when the initial abstraction is greater than the TOL value. Since the TOL value is very low (0.004 feet) and assigned based on guidance and experience from other FLO-2D studies, it is expected that the initial abstraction will be larger than the TOL value. Since the infiltration results were calibrated to match generalized HEC-1 modeling infiltration volumes, this warning message was ignored. This message does not warrant the need to modify the model.

- **FLO-2D Message:** *** THERE ARE POTENTIAL DATA ERROR(S) IN FILE HYSTRUC.DAT ***

  JEF Explanation: This general warning occurs when other warning messages that are related to hydraulic structures are generated, such as the adverse slope message that is shown below. This warning does not warrant the need to modify the model. Nothing was specifically done to eliminate this message.

- **FLO-2D Message:** WARNING: THE HYDRAULIC STRUCTURE: NO. INLET ELEMENT: OUTLET ELEMENT: HAS AN ADVERSE BED SLOPE. THE OUTLET INVERT IS HIGHER THAN THE INLET INVERT. PLEASE CHECK TO ENSURE THIS IS CORRECT

  JEF Explanation: This warning indicates that the outlet grid elevation is higher than the inlet grid elevation. Each hydraulic structure was reviewed to ensure that its resulting hydrograph was reasonable. Since the hydraulic structure results appeared reasonable, no action was taken to eliminate this message.

HYDRAULIC STRUCTURE_RUNTIME WARNINGS.OUT Messages

- **FLO-2D Message:** WARNING: THE RATING TABLE FOR HYDRAULIC STRUCTURE: WAS ADJUSTED TO BETTER MATCH THE STREAM FLOW CONDITIONS.

  JEF Explanation: This warning informs the modeler that the rating curve has been adjusted to stabilize the model/structure and that a portion of the rating table has been written to a separate output file, REVISED_RATING_TABLES.OUT. The REVISED_RATING_TABLES.OUT file was reviewed. Since the suggested revisions were minor (e.g., most revisions were at low depths, there was only a single depth listed, or the modified discharge value was approximately equal to values in the table), the original rating tables were not adjusted.

  This warning does not warrant the need to modify the model. No action was taken to eliminate all instances of this message.

- **FLO-2D Message:** THE HYDRAULIC STRUCTURE: NO. RATING TABLE WAS NOT REVISED, BUT SUGGESTED POTENTIAL RATING TABLE REVISONS ARE PRESENTED IN REVISED_RATING_TABLE.OUT FILE.

  JEF Explanation: This warning is notifying the modeler that the rating curve has not been adjusted but a portion of the rating table has been revised and written to a separate output file, REVISED_RATING_TABLES.OUT. The REVISED_RATING_TABLES.OUT file was reviewed; and since the suggested revisions were minor (as compared to the HYSTRUC.DAT rating tables), the original rating tables were not adjusted.

  This warning does not warrant the need to modify the model. No action was taken to eliminate all instances of this message.

- **FLO-2D Message:** WARNING: THE DOWNSTREAM WATER SURFACE GETS HIGHER THAN THE UPSTREAM WATER SURFACE AT TIME: THERE IS POTENTIAL FOR UPSTREAM FLOW THROUGH THE STRUCTURE: CONSIDER SETTING THE UPSTREAM FLOW SWITCH INOUTCONT = 1

  JEF Explanation: This warning indicates that the water surface elevation is higher at the outlet than the inlet. For this hydrology model, the assumption that flow will only go downstream through a structure and that runoff will not back up through the structure was used. However, structures that have this warning were investigated to determine if it would be prudent to allow the flow to back up through the structure. In all locations it was determined that it would be unnecessary to change the INOUTCONT switch to allow the upstream flow since the current modeling technique would result in a slightly higher downstream peak flow.

This warning does not warrant the need to modify the model. No action was taken to eliminate all instances of this message.

- **FLO-2D Message:** WARNING: AT TIME (HR) HYDRAULIC STRUCTURE NO. AND NAME DISCHARGE (CFS OR CMS) EXCEEDS THE INFLOW DISCHARGE (CFS OR CMS) TO THE INLET NODE BY 50% (1.5 X).

  JEF Explanation: This warning informs that modeler that inflowing stream conditions at the identified hydraulic structures may cause excessive timestep decrements (increasing total run time) due to grid cells at structure inlets having rapid drawdown in depths at certain timesteps. The structures that were listed in the warning messages were reviewed to ensure that they matched physical conditions in the field. Since the structures 1) seemed to represent physical conditions, 2) their resulting hydrographs and local depth results appeared reasonable, 3) total model run time was not excessive, and 4) the flow rates mentioned in the warning were all at low discharges (i.e., < 10 cfs), the final warnings were considered reasonable.

  This warning does not warrant the need to modify the model. No action was taken to eliminate all instances of this message.

FLOODPLAIN_CONVERGENCE.OUT Messages

- **FLO-2D Message:** THE FOLLOWING FLODPLAIN [SIC] ELEMENTS FAILED TO NUMERICALLY CONVERGE FOR THE ROUTING ALGORITHM FOR DEPTHS AND VELOCITY GREATER THAN 1 FT (0.3 M) AND 1 FPS (0.3 MPS). THIS IS NOT NECESSARILY A ISSUE THAT NEEDS TO BE RESOLVED.

  JEF Explanation: The grid cell (number 800899) where this warning occurred was reviewed, and it is located on the upstream side of a house where an ARF cell is used. Since: 1) there were only seven timesteps when this warning occurred; 2) they all occurred between hours 6-9 which well before the peak flow was determined (~12-13 hours); and 3) the floodplain hydrographs were all stable in this time range, no action was taken to eliminate this message. This warning does not indicate a need to modify the model.

EVACUATEDFP.OUT Messages

- **FLO-2D Message:** THE FOLLOWING FLOODPLAIN ELEMENTS HAVE BEEN EVACUATED OF VOLUME AND THE DISCHARGES ADJUSTED TO ELIMINATE ANY NEGATIVE VOLUME RESULTS

  JEF Explanation: The four grid cells (numbers 800057, 800899, 1720177, and 1720174) where this warning occurred were reviewed, and all were located near a building (i.e., on a partially blocked cell) where an ARF cell is used. Since 1) there were only eight timesteps when this warning occurred, 2) they all occurred between hours 3-8 which is well before the peak flow occurs (~12-13 hours), and 3) the floodplain hydrographs were all stable in this time range, no action was taken to eliminate this message. This warning does not indicate a need to modify the model.

DEPRESSED_ELEMENTS.OUT Messages

- **FLO-2D Message:** THE FOLLOWING GRID ELEMENTS ARE DEPRESSED BY AT LEAST 4.0 (FT OR M) BELOW ALL CONTIGUOUS NEIGHBORS:

JEF Explanation: The grid cells were reviewed, and most cells are located at hydraulic structures where a surveyed elevation was used. The surveyed elevations are more accurate than the averaged grid cell elevations, and the warnings for these cells were ignored.

The other cells (numbers 1186644, 1275788, and 1657568) that are not located at culverts were reviewed in more detail, and all appear to be accurate features that are shown in the original topography.

No action was taken to eliminate this message.

4.4 Calibration

Total runoff volume was calibrated to results from a generalized HEC-1 because there are very few gages in the watershed and very little real-event data to provide any other data for calibration. Additionally, calibration with HEC-1 should result in conservative values since the generalized HEC-1 models do not account for transmission losses, which are potentially significant in this watershed. This methodology was the same that was used in the approved hydrology CLOMR (JEF, 2018).

A single basin HEC-1 model was developed based on the averaged infiltration and rainfall parameters for the hydrology FLO-2D model domain. The basin area is the same as the hydrology model size, and the rainfall and infiltration parameters were averaged based upon the FLO-2D input files for the hydrology model. For example, the rainfall area reduction factors for a FLO-2D sub-area were averaged to compute the average reduction factor. That factor was applied to the point rainfall depth specified in the FLO-2D RAIN.DAT file. This average point rainfall depth was applied to the corresponding sub-basin in the HEC-1. Likewise, the infiltration parameters were averaged from the INFIL.DAT file and applied to the Green and Ampt LG record in the HEC-1 model for each corresponding sub-basin. See Table 4-8 for a summary of the HEC-1 input parameters based upon the averages from the FLO-2D input files. A generic Clark Unit Hydrograph record was used in the HEC-1 with a value of 1.5 for the TC and R values, respectively. The percent runoff generated from the HEC-1 model for each individual sub-basin was designated as the desired calibration goal for each sub-area FLO-2D model. The HEC-1 model input and output are included in Appendix D.

Table 4-8. HEC-1 Model Parameters

Storm Event	100-Year Average Rainfall (in)	IA (in)	DTHETA	PSIF (in)	XKSAT (in/hr)	RTIMP (%)
100Y24H	4.631	0.285	0.339	5.378	0.254	12.5

The FLO-2D model had a single limiting depth value assigned to all grid cells in the model. The limiting depth values for each storm event are summarized in Table 4-9.

Table 4-9. FLO-2D Limiting Depth Value

Storm Event	Limiting Depth (ft)
100Y24H	0.80

The runoff volume of each FLO-2D model was computed using the summary values in the SUMMARY.OUT file with the equation:

$$1 - (W_{II} + TOL)/VOL_R \qquad (1)$$

where:

W_{II} = water lost to infiltration and interception (ac-ft),

TOL = TOL floodplain storage (ac-ft), and

VOL_R = total rainfall volume (ac-ft).

The final comparisons between the HEC-1 and FLO-2D runoff volumes are summarized in Table 4-10. These results were comparable to the previous CLOMR, so no additional calibration was done. The total infiltration between the existing (without levee) and proposed (with levee) hydrology models was comparable.

Table 4-10. FLO-2D Limiting Depth Calibration for the 100-year 24-hour Event

Model Condition	No. Grids	Area (sq. mi.)	HEC-1 Loss (in)	HEC-1 Excess (in)	HEC-1 % Runoff	FLO-2D % Runoff
Existing	2081647	29.87	2.74	1.89	**40.8**	37.8
Proposed	2081647	29.87	2.74	1.89	**40.8**	38.0

4.5 Final Results

4.5.1 Hydrologic Analysis Results

Table 4-11 shows the computed peak discharges at relevant locations throughout the Rawhide Wash study area for the 100-year 24-hour storm event for the proposed (i.e., after levee project completion) conditions. Complete results for the entire FLO-2D model domain are provided electronically in Appendix D.

Table 4-11. Summary of Hydrologic Discharges

Location	FLO-2D FPXSEC ID	Area (sq. mi.)	100-year 24-hour		
			Peak Q (cfs)	Time (hrs)	Volume (ac-ft)
Rawhide Wash Upstream of East Happy Valley Road (Rawhide Wash Apex)	1	14.1	9,105	13.36	1,343
Rawhide Wash at Miller Road alignment	26	14.2	9,033	13.52	1,346
Rawhide Wash upstream of Scottsdale Road	24	14.3	9,026	13.59	1,357
Deer Valley Road Channel at Scottsdale Road	27	7.1	1,486	14.25	544

4.5.2 Verification of Results

The results from the Rawhide Wash hydrology modeling were used to make an indirect methods verification assessment with three methods from the FCDMC Hydrology Manual. The indirect methods are as follows:

- FCDMC Method 1 – Extreme Event Peak Discharge Curves, shown in Figure 4-6
- FCDMC Method 2 – USGS Flood Frequency Data for Arizona, shown in Figure 4-7
- FCDMC Method 3 – 2014 USGS Flood Frequency Data for Arizona (Paretti et al., 2014), shown in Figure 4-8 and Figure 4-9

The peak flow at the Rawhide Wash Apex is the focus of these comparisons because 1) it is the largest watercourse within the study area and 2) it has a tributary watershed with an easily calculable drainage area. Only the proposed conditions result is shown because the peak discharges were essentially the same. From the figures, the Rawhide Wash peak flows are contained under the applicable envelope curves for Method 1, within the 75% tolerance limits of Method 2 and within the cloud of data points trending along the curve for Method 3. Since the Rawhide Wash watershed is within Region 3 but right on the border of Regions 3 and 4, the results are shown for both Region 3 and the adjacent Region 4 for the Method 3 comparison. These results indicate that the calculated flow rate for the Rawhide Wash apex is reasonable and acceptable for design. For reference, the Deer Valley Road Channel flow is also plotted on these figures.

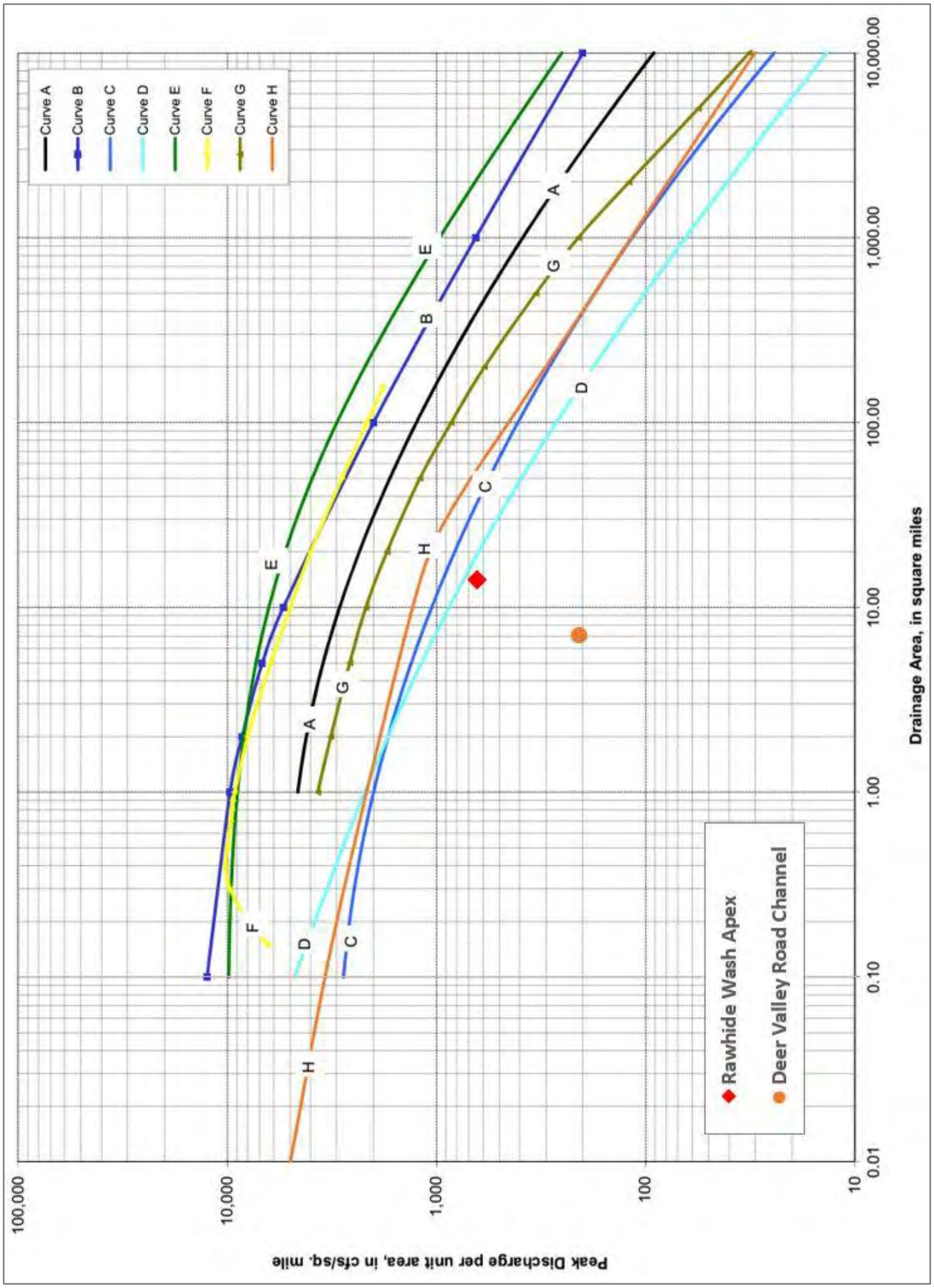


Figure 4-6. FCDMC Method 1 – Extreme Event Peak Discharge Curves

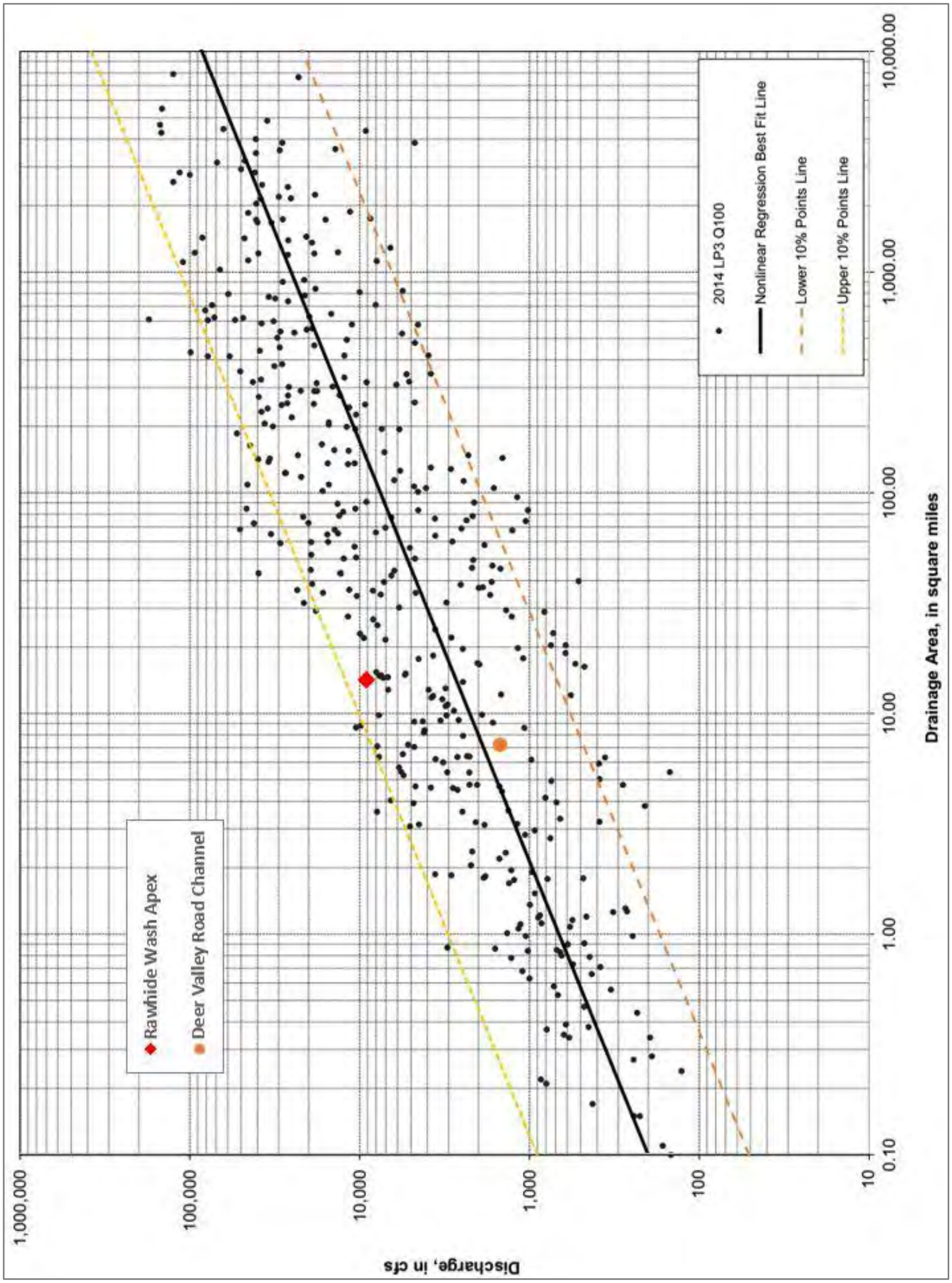


Figure 4-7. FCDMC Method 2 – USGS Flood Frequency Data for Arizona

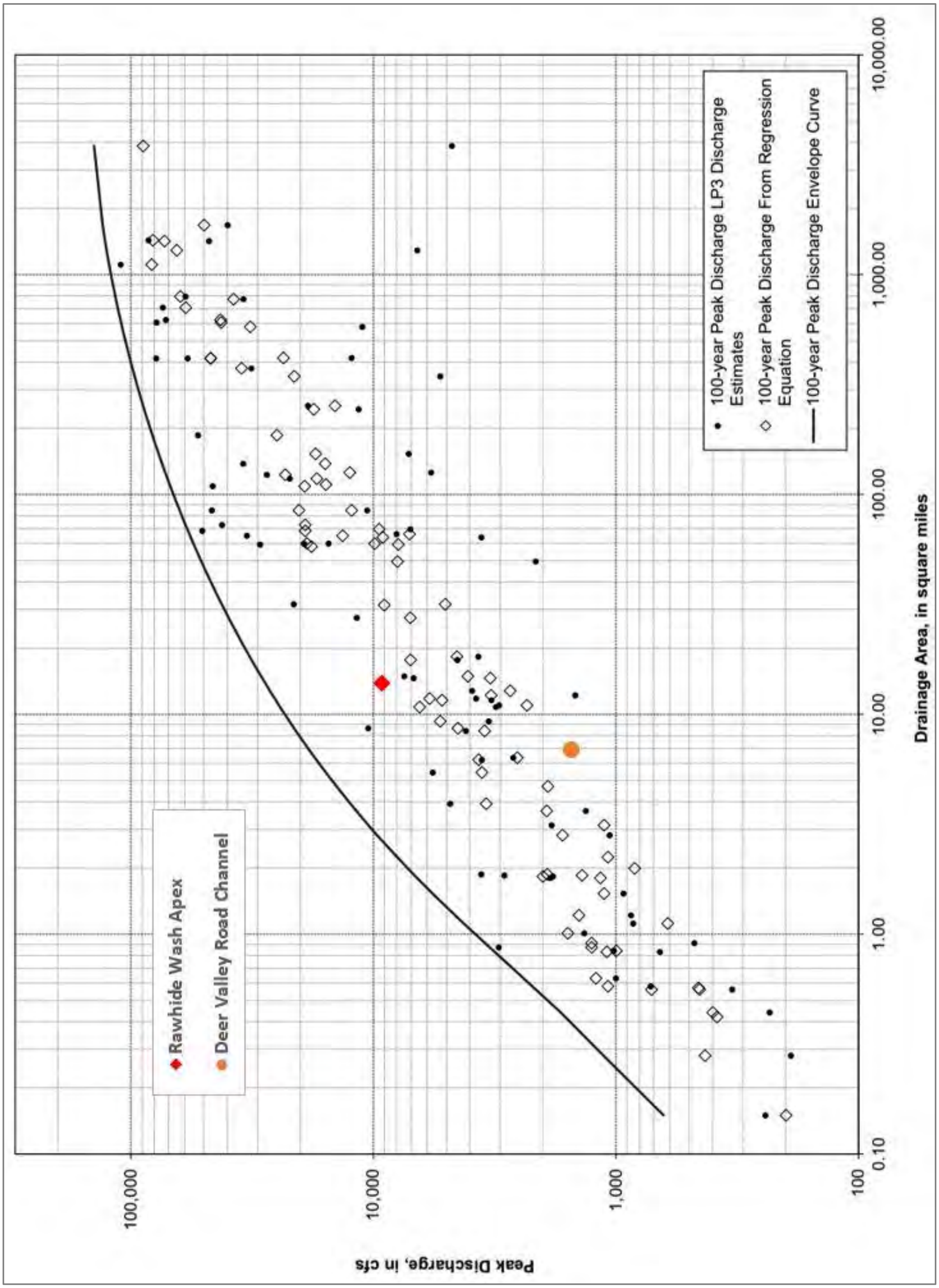


Figure 4-8. FCDMC Method 3, Region 3 100-year Regression Equation

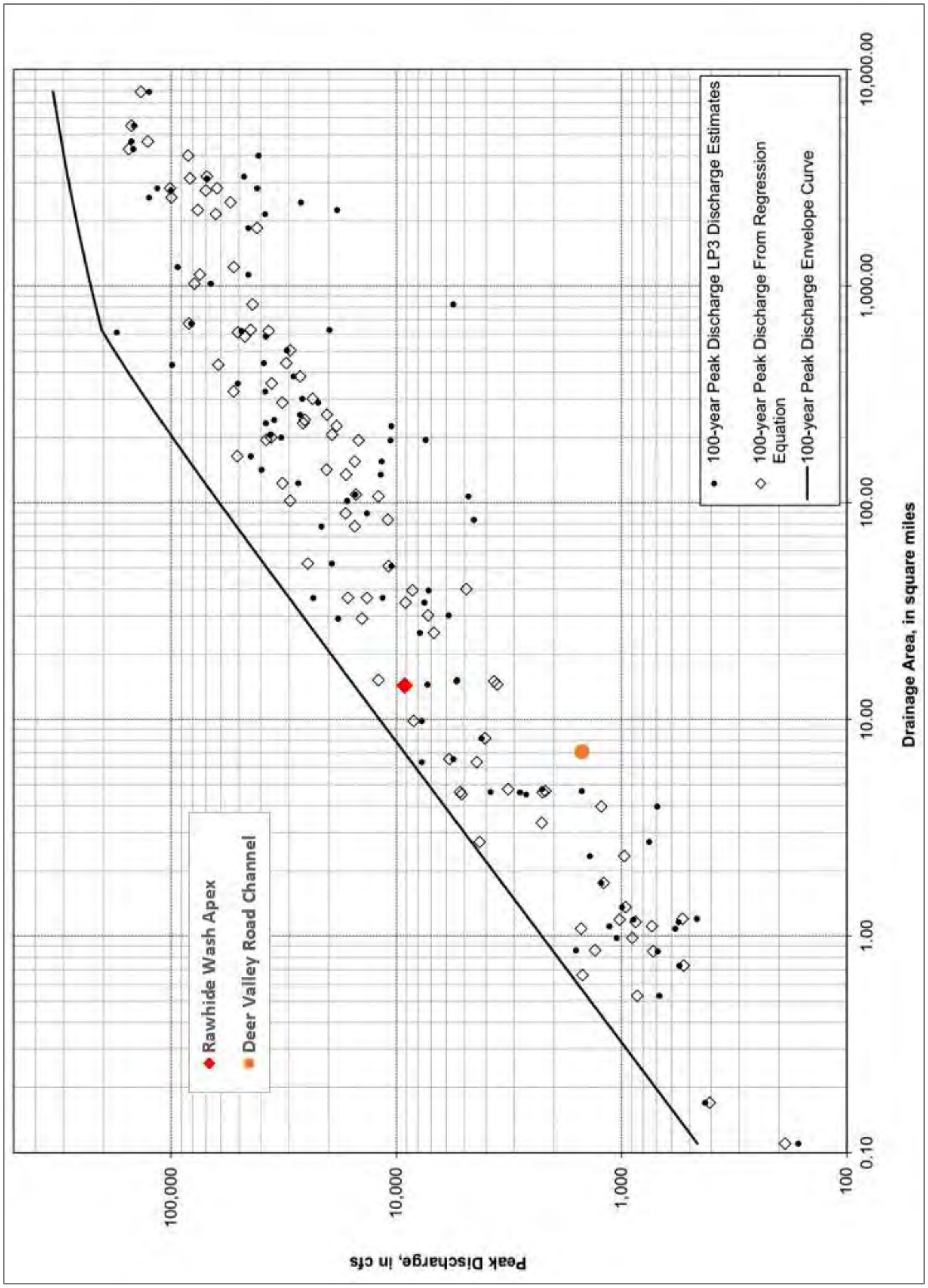


Figure 4-9. FCDMC Method 3, Region 4 100-year Regression Equation

4.5.1 Other Recurrence Intervals

Since there are peak flow estimates for the 10- and 50-year recurrence intervals in the current effective FIS, the calibrated 100-year proposed condition model was run with new RAIN.DAT files for those recurrence intervals for the 24-hour duration. The results from these new models are shown in Table 4-12.

Table 4-12. Summary of 10- and 50-year Hydrologic Results for Rawhide Wash (Proposed Conditions)

Location	Recurrence Interval	24-hour			
		Maximum Precipitation (inches)	Peak Q (cfs)	Time (hrs)	Volume (ac-ft)
Rawhide Wash Upstream of East Happy Valley Road (Rawhide Wash Apex)	50-Year	3.85	6,683	13.56	1,044
Rawhide Wash at Miller Road alignment			6,615	13.73	1,042
Rawhide Wash upstream of Scottsdale Road			6,609	13.82	1,049
Rawhide Wash Upstream of East Happy Valley Road (Rawhide Wash Apex)	10-Year	2.76	2,410	14.20	468
Rawhide Wash at Miller Road alignment			2,342	14.45	460
Rawhide Wash upstream of Scottsdale Road			2,329	14.58	461

5 Hydraulics

5.1 Method Description

The 100-year floodplains for Rawhide Wash were calculated using two different software packages – HEC-1 and FLO-2D. The software was used where each was most applicable to the physical conditions of the modeled reach. For this CLOMR, Rawhide Wash was separated into four distinct reaches. These reaches are discussed below, and the reach limits are shown in Figure 5-1.

1) Reach 1 – This reach is the confined (with levees or an actual channel) reach that extends from the Rawhide Wash alluvial fan apex to Scottsdale Road. The portion of this reach from the apex to Pinnacle Peak Road is the proposed leveed section that is the focus of the final design project (see Design Data Report for details), while the lower portion from Pinnacle Peak Road to Scottsdale Road is an existing channelized section. This reach, currently delineated as a Zone AE on the effective FIRM, was delineated with an AE flood zone. This entire reach was delineated using a high resolution (10-foot cell size) FLO-2D model, which used the hydrograph results from the FLO-2D hydrology model (see Section 4) as input.
2) Reach 2 – This reach extends from Scottsdale Road to the upstream faces of the Loop 101 freeway (L101) culverts. Since the levee project is designed to control the Rawhide Wash alluvial fan apex (located upstream of Happy Valley Road), there is the potential that this apex is shifted to the downstream limits of the leveed/channelized reach (at Scottsdale Road). Therefore, this reach was modeled with FLO-2D to capture the two-dimensional (2D) aspects of distributary flow. Additionally, multiple avulsion scenarios were developed and run in FLO-2D to assess: 1) the activity and severity of the distributary flow; and 2) if this flow rises to the level of an active alluvial fan (see Section 6). This model uses a 10-foot cell size and was developed using the same input features as the hydrology model (see Section 4). This reach was delineated as an AO zone with depth and velocity components (i.e., an active alluvial fan floodplain). This model used inflows from the overall hydrology model. Finally, the FLO-2D grid-based results (velocity, flow depth, cross-section-based discharges, and water surface elevation) are not intended to be used for floodplain permitting purposes or for the design of any drainage structures for this reach.
3) Reach 3 – This reach extends from the downstream ends of the L101 culverts to the ponding limits of the CAP Reach 11 Dike 2. Because avulsions are unlikely in Reach 3 for multiple reasons, a fixed bed model was applied to this reach. These reasons include:

    a. Flow to Reach 3 is physically controlled by L101 and its culverts.
    b. There are multiple hard structures within this area, such as Mayo Blvd and other associated streets and culverts.
    c. This reach is located at the very downstream area of the alluvial fan where the predominant slope is about 1% (lower than on a typical active alluvial fan).
    d. In general, the FEMA alluvial fan delineation methodology is applicable to areas that are not subject to human disturbance (e.g., infrastructure such as L101) because the methodology relies on existing geomorphology data which may be already changed.

    Additionally, FLO-2D was chosen to model this reach because of the number of streams, culverts, and storm drains within this reach. This enables the entire reach to be simulated with one model and with one model run. The FLO-2D model domain extended into the Dike 2 flood pool to identify and delineate which water surface controlled (i.e., the ponded elevation or the channel peak flow elevation). This model also used a 10-foot cell size and was developed using the same input features as the hydrology model (see Section 4). Because Reach 3 is an extension of the Rawhide Wash alluvial fan, this reach was delineated an AO zone with depth and velocity components (i.e., an active alluvial fan floodplain). The lateral extents of the revised AO zone were determined from the FLO-2D results. Inflows for the Reach 3 model were determined from the maximum L101 culvert hydrographs from the Reach 2 modeling scenarios, and these same hydrographs scaled to the original design flows for each L101 culvert within the revised AO zone of Reach 2. However, the FLO-2D grid-based results (velocity, flow depth, cross-section-based discharges, water surface elevation) are not intended to be used for floodplain permitting purposes or for the design of any drainage structures for this reach.

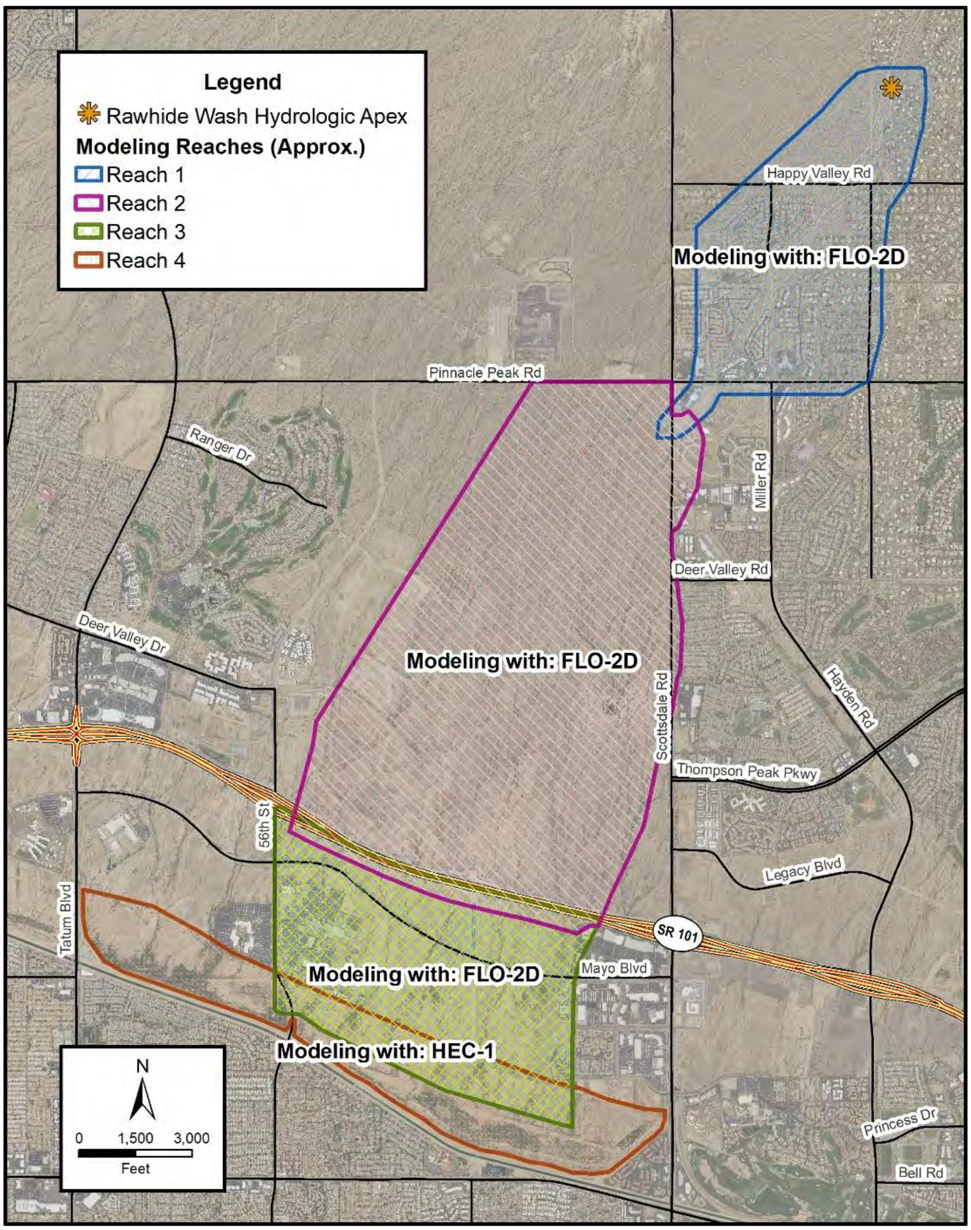


Figure 5-1. Hydraulic Modeling Reaches

4) Reach 4 – This reach is the flood pool of the CAP Reach 11 Dike 2. This Dike is part of the system that protects Reach 11 of the Hayden-Rhodes Aqueduct that is a part of the Central Arizona Project (CAP). The Reach 11 Dikes are federally owned, maintained, and operated by United States Bureau of Reclamation (USBR) in cooperation with the CAP (see Figure 5-2 for an excerpt from the National Inventory of Dams[2]).

   Since this is a ponded area, the modeling was done in HEC-1 to calculate the runoff volume to the Dike and the peak stage of its flood pool. The Dike does have floodgates that can allow stormwater to leave the flood pool. However, these floodgates were kept in a closed position until 2014. Since historically the gates have been closed, no outflow from the flood pool was considered when developing the ponded water surface elevation. This reach was delineated as an AE zone without a floodway. The entire floodplain for the flood pool is federally owned by either the USBR or the Bureau of Land Management. The current floodplain is a mix of Zone A and Zone AO on the effective FIRM.

Details of Dam

Dam Name:	REACH 11 DETENTION DIKE 2
Other Dam Name:	-
NID ID:	AZ82914
River:	NONE
County:	MARICOPA
State:	AZ
Owner Type:	Federal
Owner Name:	RECLAMATION
NID Height:	40 feet
NID Storage:	14,600 acre-feet
Primary Dam Type:	Earth
Primary Purpose:	Flood Control
Year Completed:	1977
Hazard Potential:	High
EAP:	Yes
Last Inspection Date:	6/13/2017
State Regulated Dam	N
Federal Regulatory Agency	DOI BR
Date Data Provided to NID:	07/02/2018

Figure 5-2. Excerpt from the NID for Reach 11 Dike 2

[2] https://nid.sec.usace.army.mil/ords/f?p=105:1::::::

5.2 Work Study Maps

Work study maps were prepared using a scale of 1 inch = 200 feet to be consistent with FEMA standards. The maps delineate the proposed floodplain boundaries and the base flood elevations. The base flood elevations were developed from the Reach 1 FLO-2D maximum water surface elevation raster and smoothed to appropriately tie into the floodplain boundaries. The maps are included with the TSDN as Exhibit F.

5.3 Parameter Estimation

The hydraulic modeling used the same detailed land use classification that was used for the hydrology modeling. Please refer to Section 4.2.6.

5.4 Cross Section Description

Not applicable. The model is based on a two-dimensional grid surface. No one-dimensional cross-sections are used.

5.5 Modeling Considerations

5.5.1 Hydraulic Jump and Drop Analysis

Not applicable, any hydraulics jumps that occur within the watershed are handled implicitly by the two-dimensional grid surface and numerical scheme. However, the flow was limited to subcritical by using a limiting Froude number of 1 (see Table 4-7).

5.5.2 Bridges and Culverts

Culverts and bridges (with one exception) within the detailed delineation area were modeled in FLO-2D with the hydraulic structure rating table routine. The rating tables were developed with HY-8 and are included in Appendix E. The one exception was the proposed Miller Road Bridge. Since this bridge is on a severe skew, it was simulated with levees where the abutments are located and area reduction factors (ARF) to model the piers by reducing the available area in the cell. ARF values were applied at locations where the piers intersected the cell area. Some pier locations roughly intersected two cells equally, and ARFs were applied to both cells at these locations. The ARF values and their location in relation to the pier locations are shown in Figure 5-3. Since the low chord of this bridge is designed to be 2 feet above the 100-year water surface elevation and hydraulics were compared to a HEC-RAS 2D model for this bridge (included in Appendix E), this approach was considered reasonable.

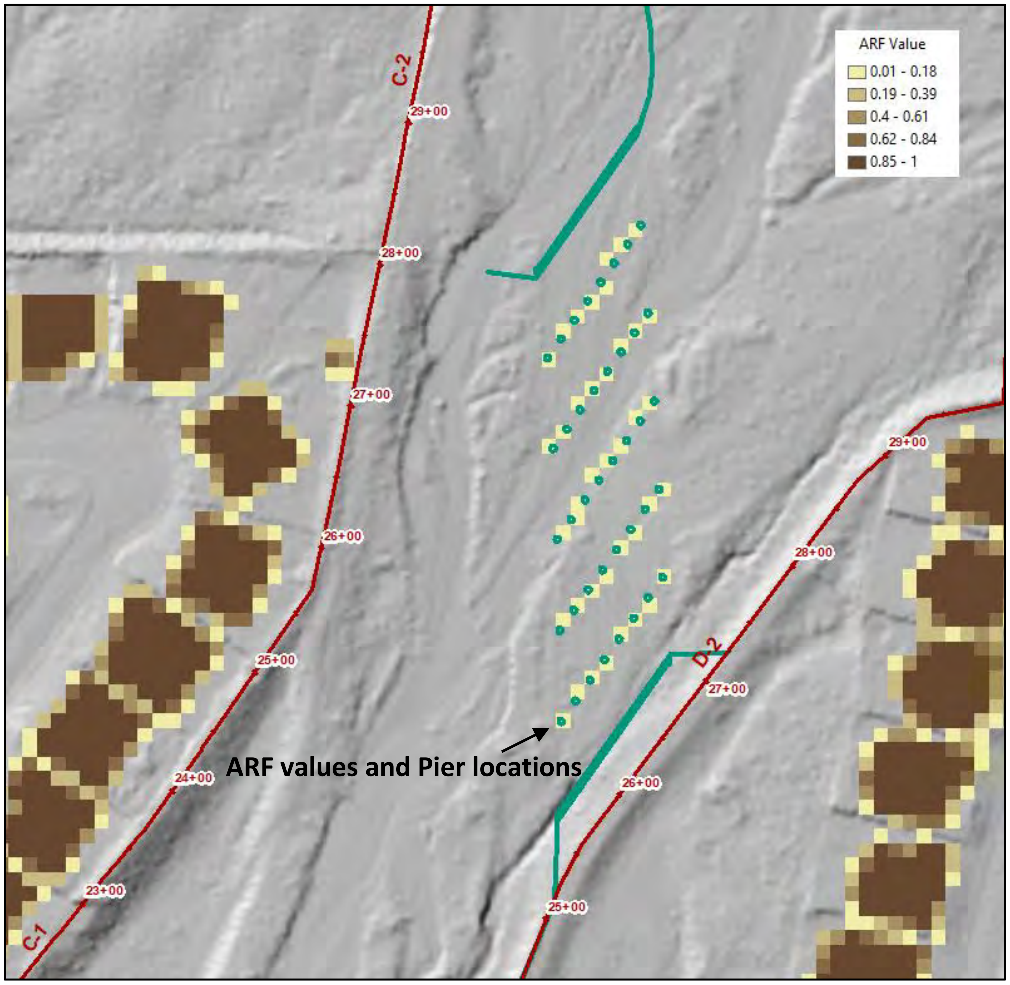


Figure 5-3. Schematic of Miller Road Piers and ARF Values

5.5.3 Levees and Dikes

The levees along Reach 1 of Rawhide Wash are the focus of the design project. The certification documents (i.e., MT-2 Forms and supporting technical reports) are included with this document, while the design details are documented in the Design Data Report.

The CAP Reach 11 Dike 2 is a dam that forms the southern limit of Reach 4. It is federally owned, maintained, and operated by United States Bureau of Reclamation (USBR) in cooperation with the CAP. This dam was considered in the HEC-1 modeling.

5.5.4 Levee Interior Drainage

In general, all areas propose drainage facilities to pass interior drainage from the dry side of the levee system to the Rawhide Wash side. There are five interior drainage locations, shown in Figure 5-4, along the project reach that were evaluated. Two were modeled directly within the Reach 1 FLO-2D model and the remaining three were designed using Rational Method and culvert/storm drain analysis.

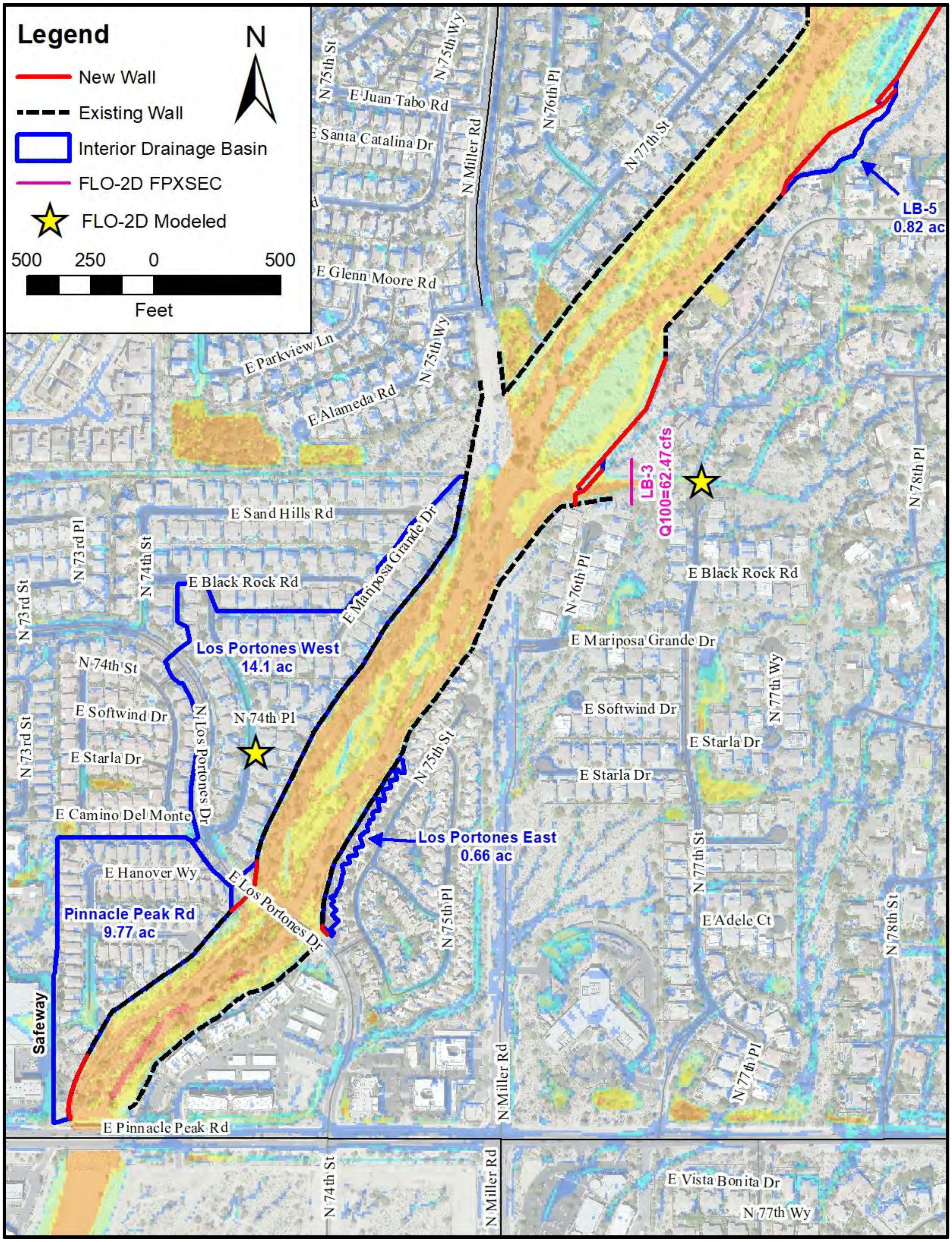


Figure 5-4. Interior Drainage Locations

The design of the areas not modeled in FLO-2D were analyzed assuming a 100-year flow event on the dry side that is coincident with a 10-year flood in Rawhide Wash. This approach is considered conservative for these small areas and is verified by comparing the FLO-2D generated, coincident hydrographs for the inflow to the LB-3 and Rawhide Wash for a 100-year event, shown in Figure 5-4. The LB-3 watershed is the largest of the all the interior drainage area watersheds and has the highest potential for generating a peak discharge that is close to a peak in Rawhide Wash for the same event. As can be seen in Figure 5-4, the times to peak are significantly offset and show that a 100-year peak from the dry side has only minimal flow in Rawhide Wash. Alternately, the inflow from the LB-3 watershed is minimal when compared to peak in Rawhide Wash.

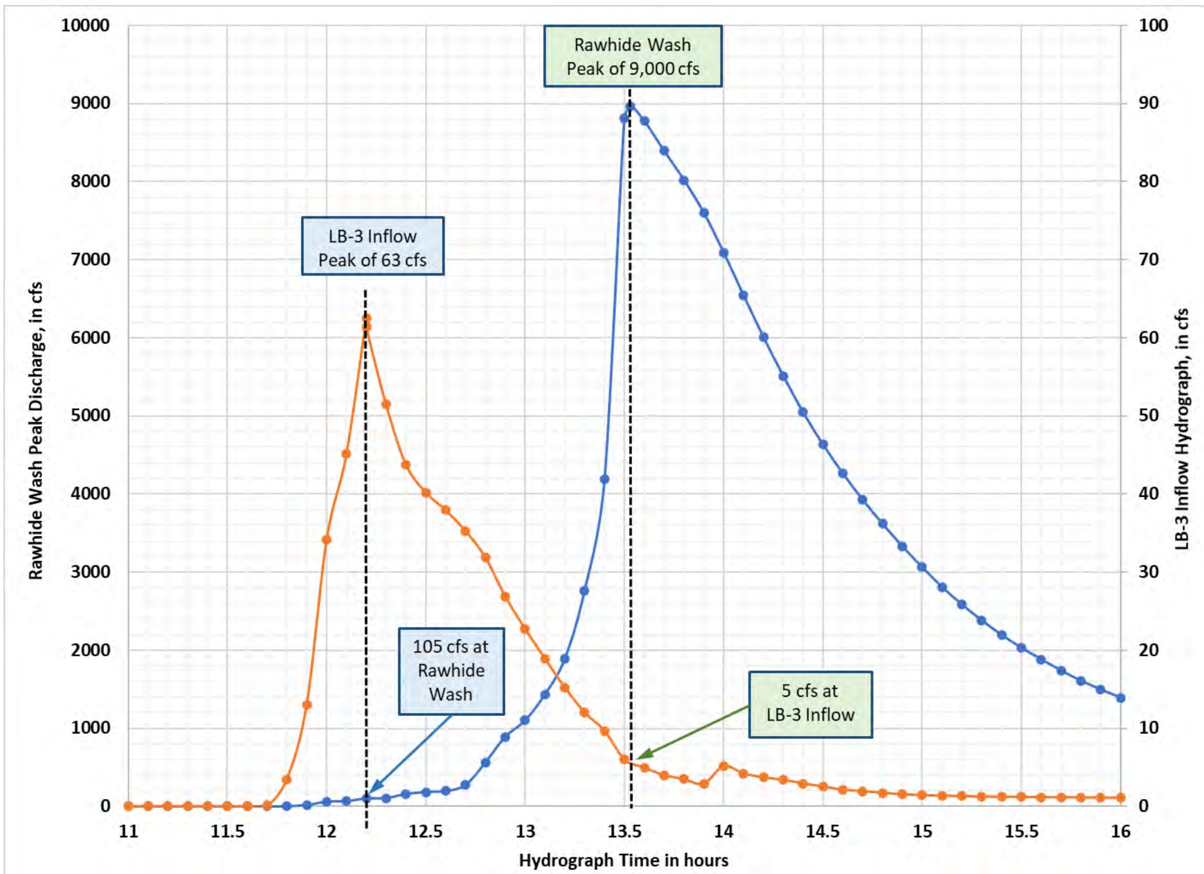


Figure 5-5. Comparison of Hydrographs at LB-3 Inflow Location

Rational Method calculations and simple culvert analyses are used to size the interior drainage infrastructure for the areas not modeled in FLO-2D. See the Design Data Report for more information on the designs and hydraulics.

Two interior drainage locations, indicated by the yellow star on Figure 5-4, were analyzed with the Reach 1 FLO-2D model because they required a more detailed analysis and were anticipated to be shown on the workmaps as a delineated floodplain. These two locations are discussed further in the following subsections.

5.5.4.1 West Side of Los Portones

Currently, flows accumulate in a sump located in Los Portones Drive, west of Rawhide Wash. The sump is drained by two separate culvert and storm drain systems that discharge to Rawhide Wash on the downstream side of Los Portones Drive. Major flow events in Rawhide Wash can spill to the sump and cause flooding to the surrounding area. The proposed construction of the west floodgate will require intercepting and rerouting the runoff to avoid profile conflicts with the floodgate's concrete base.

The Reach 1 FLO-2D model is used to evaluate the proposed condition hydrology and hydraulics of the interior drainage system at this location. The new proposed rerouted storm drain configuration is modeled using the SWMM module of FLO-2D. In general, all conduits were modeled with the standard entrance and exit loss coefficients of 0.5, but the exit loss coefficient at the outlet to Rawhide Wash was modeled with a higher exit loss coefficient of 2.0 to simulate the backflow prevention device. The value of 2.0 was chosen through an iterative process to match the standard increase in head caused by the backflow preventer as shown in the manufacturer's charts. Finally, the outfall for this system was modeled with a tide gate to prevent water from Rawhide Wash entering the system.

A comparison of the existing and proposed conditions results indicates that the 100-year conditions west of the floodgate at Los Portones are improved with the addition of the proposed floodgate and rerouted storm drain, and that 100-year flow depths are essentially contained within the Los Portones Drive Right of Way (see Figure 5-6). The proposed rerouted storm drain intercepts and conveys a peak discharge of approximately 31 cfs from a tributary area of 14.1 acres. A review of the detailed hydraulic output determined that most of the interior drainage runoff volume is discharged to Rawhide Wash through the rerouted pipe system before flow depths in Rawhide Wash rise substantially. On average, the proposed conditions depths are less than 1 foot, and a floodplain delineation is not warranted.

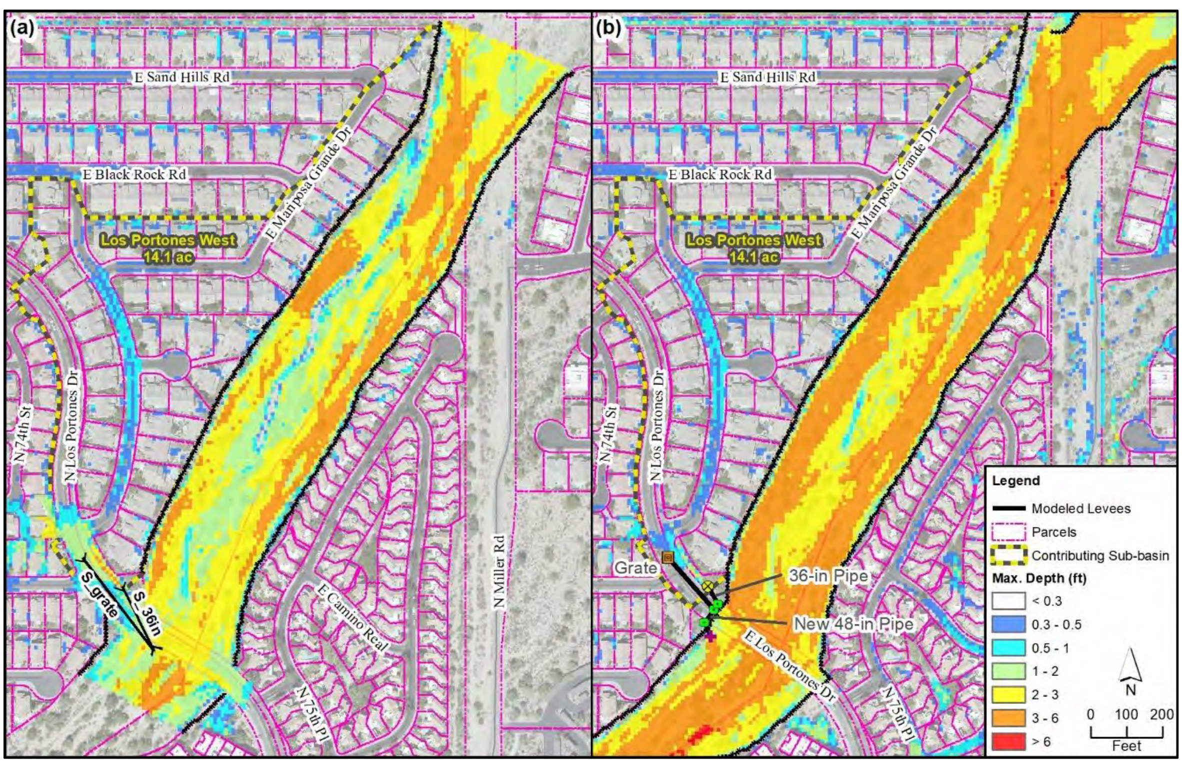


Figure 5-6. Comparison of (a) Existing and (b) Proposed Conditions on the West Side of Los Portones Drive

5.5.4.2 Floodwall LB-3 Impoundment

The runoff for this interior drainage area was best estimated using the Reach 1 FLO-2D model given the distributary flow within the tributary area and the detention of flood flows passing to Rawhide Wash through Floodwall LB-3. The 100-year peak inflow to the land side of Wall LB-3 is estimated from the Design FLO-2D model to be approximately 63 cfs with a volume around 6 ac-ft. This flow will be routed to the wash side of Floodwall LB-3 through 2-24" pipes equipped with backflow prevention devices and will create a detained flood elevation of 1936.3 feet in the interior impoundment area. The detention affect will not negatively impact the adjacent properties and fill will be placed on the land side of Floodwall LB-3 (see the construction plans) to move the ponded flows away from the wall subgrade. On average, this area has depths greater than 1 foot. Therefore, an AE Zone floodplain was delineated and is shown on the workmaps.

5.5.5 Non-Levee Embankments

One non-levee embankment was removed from the modeling surface as a part of the sediment depositional analyses (see Section 6). Other non-levee embankments, such as roadway embankments like the Loop 101 freeway, were not removed from the model since the upstream ponded elevation only occurs for a few hours (i.e., approximately 5 hours based on the hydrographs from the modeling) and I less. The Loop 101 freeway embankment is over 180-feet wide and varies between 5 to 15 feet above the prevailing grade just north (upstream) of the freeway. Concrete lined channels intercept flows from the north and direct them to existing culverts. The freeway culverts are designed to pass the flood flows to the south and have inverts typically constructed several feet below the natural prevailing grade. Accordingly, ponding depths on the roadway embankment itself are very limited.

5.5.6 Islands and Flow Splits

Islands and flow splits are automatically calculated in the FLO-2D model. Small islands generally less than 0.1 acres were included in the revised floodplains during the delineation process.

5.5.7 Ineffective Flow Areas

Not applicable; the model is a two-dimensional, and ineffective flow areas are not necessary.

5.5.8 Supercritical Flow

While the washes run very close to critical due to the high slope of the watershed, the model was run with a limiting Froude number of 1 to be consistent with FEMA standards.

Both the hydrology (20-ft grid) and detailed hydraulic (10-ft grid) FLO-2D models show supercritical elements. All of these occur at depths less than 1-ft where flow is not controlled by the limiting Froude number. Generally, these elements occur in the mountainous areas in the hydrology models and along the Pinnacle Peak Road drop structure in the hydraulics models. Since these supercritical elements generally occurred in high slope areas and at depths that do not coincide with peak flows, the results were considered acceptable.

5.5.9 Reach 3 Inflow

Since Reach 2 is delineated as an active alluvial fan, the inflows to the Reach 3 model are not directly determined from the Hydrology model. Therefore, multiple sediment deposition scenarios were run with the Reach 2 FLO-2D model (see Section 6) to estimate the flow limits of the active fan. The combined worst-case maximum discharges (developed by adding the instantaneous peaks from each FLO-2D hydraulic structure shown in the model output files) through the L101 culverts in the Reach 2 modeling were used to develop inflow hydrographs to the Reach 3 modeling. Additionally, these worst-case hydrographs were scaled to match the original design flows of each culvert within the revised AO zone to provide a second modeling scenario for the Reach 3 model. The development of these two sets of inflow hydrographs is documented in an Excel file in Appendix E, and more details about the culvert locations and inflows are provided in Section 6.

5.6 Floodway Modeling

No floodway analyses were performed for this study.

5.7 Issues Encountered During the Study

5.7.1 Natural Valley Procedure

FEMA's Natural Valley Procedure (NVP) was used to define the revised flood zone "X" with subtype "Area With Reduced Flood Risk Due To Levee" for the landward areas where the levees provide protection. The "natural valley" term of the procedure refers to natural floodplain of the river system that existed for the construction of the levee. For this project, the overall hydrology FLO-2D model was used for the NVP modeling since this domain is large enough to capture flow patterns outside the leveed corridor. A total of six scenarios were run to define the proposed zone "X" with subtype "Area With Reduced Flood Risk Due To Levee", and these scenarios are outlined in Table 5-1 with locations of the East and West Levee Systems shown in Figure 5-7. The proposed zone "X" derived from these scenarios is shown on the Exhibit F floodplain work maps. Note that in each scenario the Miller Road Bridge abutments were left in the model since these are designed to the 500-year event, which is a larger storm than the regulatory 100-year event.

Table 5-1. Scenarios used in the Natural Valley Procedure FLO-2D Modeling

SCENARIO	Assumed Status of Levee System (With Levee or Natural)		
	East System North	East System South	West System
1	Natural	Natural	With Levee
2	Natural	With Levee	With Levee
3	With Levee	Natural	With Levee
4	With Levee	Natural	Natural
5	Natural	With Levee	Natural
6	With Levee	With Levee	Natural

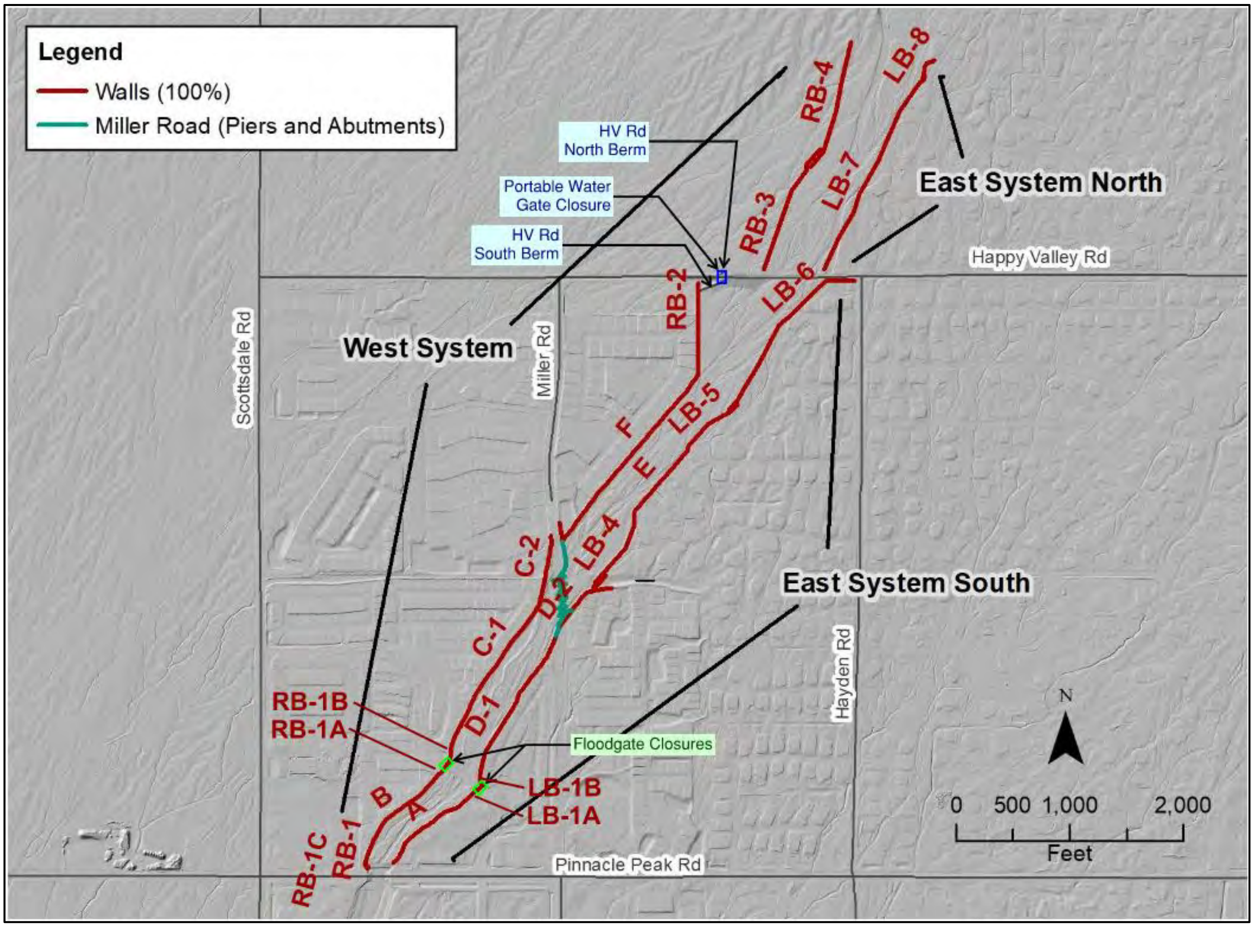


Figure 5-7. Levee Systems in the Natural Valley Procedure FLO-2D Modeling

5.8 Calibration

No additional calibration was done for the hydraulics portion of the study except what was done for the hydrology portion of the study (see Section 4).

5.9 Final Results

5.9.1 Hydraulic Analysis Results

The hydraulic results for Rawhide Wash are summarized in Appendix E and shown on the Exhibit F floodplain work maps.

5.9.2 Verification or Comparison of Results

No other verification was performed except the indirect methods that are documented in Section 4.5.2.

6 Erosion, Sediment Transport, and Geomorphic Analysis

6.1 Methodology

Since the project is designed to control the Rawhide Wash alluvial fan apex, various sedimentation analyses were performed as a part of this study. These analyses include:

1) HEC-6T sediment transport modeling for Reach 1; and
2) Alluvial fan flow path uncertainty and avulsion modeling for Reach 2.

The HEC-6T modeling was a part of the detailed design for the levees; and, as such, those analyses are presented in detail in the Design Data Report (JEF, 2021). Please see that report for detailed information about the levee design. For this CLOMR document, the Reach 2 analyses are documented herein because they are a guideline for the floodplain delineation within Reach 2.

Since this levee project confines Rawhide Wash (an active alluvial fan per the effective FEMA FIRM panels) to a single channel from the apex to Scottsdale Road, an analysis of flow path uncertainty was performed downstream of the culvert at Scottsdale Road using the procedures outlined in *Re-analysis of Alluvial Fans No. 5 and No. 6 in Scottsdale and Phoenix, Maricopa County, Arizona based on 2003 FEMA Alluvial Fan Guidelines* (FCDMC, 2014).

Since this is a design project (i.e., an alluvial fan apex does not currently exist downstream of Scottsdale Road), many of the steps, outlined in Figure 6-1, were skipped, and an active engineering analysis was performed (see circled area in the figure). This is equivalent to proceeding to Step 3 of the three-step process that is outlined in *Guidelines and Specifications for Flood Hazard Mapping Partners-Appendix G: Guidance for Alluvial Fan Flooding Analyses and Mapping* (FEMA, 2003). Therefore, the area downstream of Scottsdale Road is being considered an active alluvial fan, and the floodplains were developed using an engineering analysis and a comparison with the effective fan delineation in the Reach 2 area.

The engineering analysis consisted of developing estimates of potential deposition at the new apex and applying these estimates as fill areas to the geometry of the Reach 2 FLO-2D model. Of primary concern with active alluvial fan flooding is flowpath uncertainty (i.e., flowpaths that are subject to lateral changes over time or during a single flood event). The intent of this analysis is to address potential flowpath uncertainty downstream of the fan apex by simulating avulsion scenarios. The Scenarios are described below:

A. 2016/2019 LiDAR without modification,
B. 2016/2019 LiDAR with stock tank embankment removed,
C. 2016/2019 LiDAR minor channels downstream of Scottsdale filled with 3 feet of sediment (an amount that was assumed to approximate deposition during a single significant event),
D. 2016/2019 LiDAR minor channels downstream of Scottsdale filled with 3 feet of sediment and stock tank embankment removed,
E. 2016/2019 LiDAR with five times the 10-year sediment yield (based on FCDMC, 2014) distributed across the area downstream of Scottsdale Road, and

F. 2016/2019 LiDAR with five times the 10-year sediment yield (based on FCDMC, 2014) distributed across the area downstream of Scottsdale Road and stock tank embankment removed.

The smaller depositional volume (Scenarios C and D) was chosen to approximate shorter duration impacts from sediment deposition, while the larger volume was chosen to be consistent with FCDMC methodology and to approximate longer term sediment impacts (without sediment removal or maintenance). In addition, the berm that formed the major storage area of a stock tank and a diversion dike was also removed in Scenarios B, D, and F since this berm performs as a levee-like structure but is not a FEMA certifiable levee. The spatial locations of the sediment and berm modifications are shown in Figure 6-2. More detail about how the 10-year sediment volumes were determined is provided in the next subsection.

6.1.1 Development of 10-year Sediment Volumes and Spatial Distribution

Sediment yield was computed as part of Work Assignment #6 (WA6) of the Pinnacle Peak West (PPW) Area Drainage Master Plan (JEF, 2016) using the FCDMS's DDMSW software (Version 4.8.2). The Rawhide Wash watershed was delineated at Jomax Road to obtain a watershed area of 13.88 square miles. With this area, the results from the WA6 study indicated an annualized yield of 0.18 ac-ft/mi^2/year, which is comparable to values reported in the FCDMC Hydraulics Manual (2018b) for watersheds of similar sizes. The total sediment yield reported in Table 6-1 includes both the wash and bed loads.

Two concentration points are specified in this CLOMR study where the long-term sediment yield volumes estimates will be calculated. A linear scaling based upon watershed size was used to compute sediment yield at both these locations. The first point corresponds to the fan apex, which is just downstream of the Jomax location from the PPW Study, while the second point is at the outlet of the Deer Valley Channel. The Rawhide Wash fan apex was used because sediment transport modeling indicated that sediment will be transported to the relocated fan apex at Scottsdale Road.

Additionally, HEC-6T modeling that was performed as part of the final design was modified to include a single 10-year event, and a detailed description of the model approach can be found in the Design Data Report (JEF, 2021). Hydrology for this event was extracted from the 10-year FLO-2D model performed as a part of the CLOMR modeling (see Section 4.5.1). The results of this modeling indicate approximately 5.8 ac-ft of sediment was delivered to Scottsdale Road during the 10-year event.

Therefore, the final sediment volumes that were used for the long-term sediment deposition scenarios (E and F) are 29.2 ac-ft at the Rawhide Wash Apex and 11.8 ac-ft at Deer Valley Road Channel, which is five times the 10-year sediment volume. The higher HEC-6T sediment volume was used for Rawhide Wash, while the sediment yield volume was used at Deer Valley Road. The location of the two deposition locations and the sediment yield watersheds are shown in Figure 6-3.

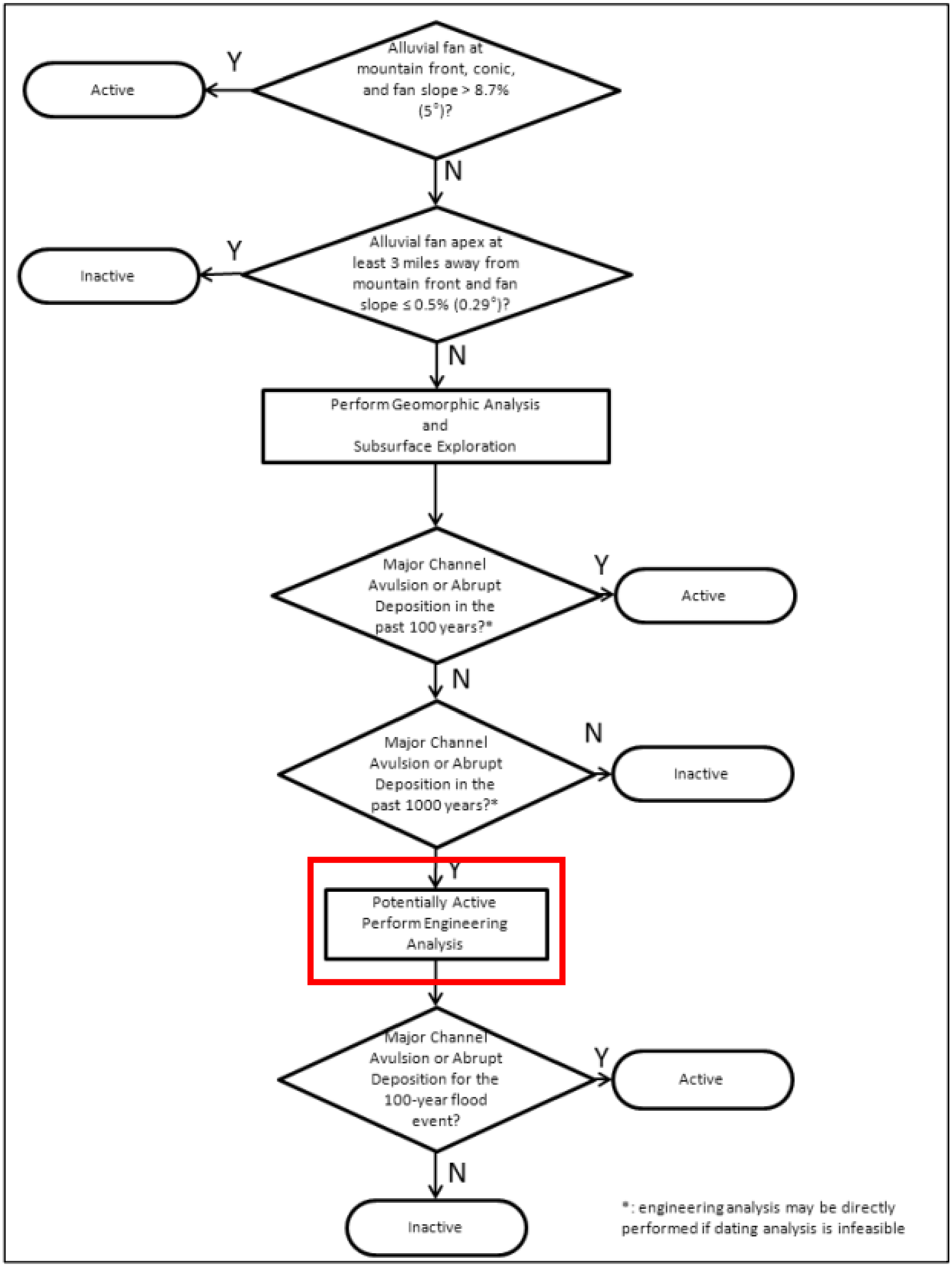


Figure 6-1. Flowchart for Stage 2 Determination of Active Alluvial Fan, from (FCDMC, 2014)

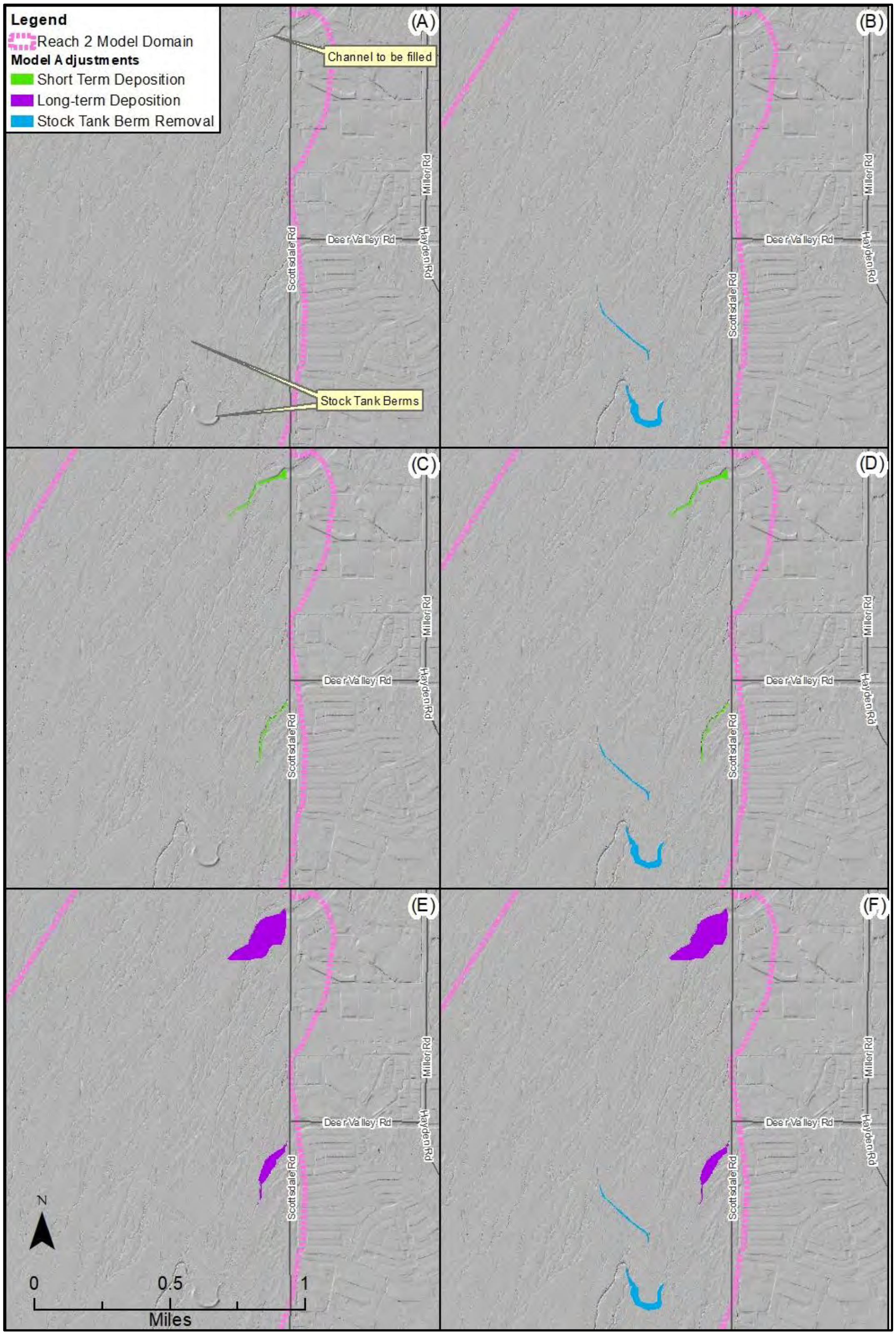


Figure 6-2. Spatial Locations of Model Adjustments for Avulsion Models: (A) 2016/2019 Topography, (B) 2016/2019 Topography with Stock Tank Removed, (C) 2016/2019 Topography with Short-term Deposition, (D) 2016/2019 Topography with Short-term Deposition and Stock Tank Removed, (E) 2016/2019 Topography with Long-term Deposition, and (F) 2016/2019 Topography with Long-term Deposition and Stock Tank Removed

Table 6-1. Sediment Yield Results

	PPW Study (JEF, 2016)	to Apex	Deer Valley Channel
Watershed Area (sq. mi.)	13.88	14.12	7.08
Recurrence Interval	**Total Yield (ac-ft)**		
2-year	1.32	1.34	0.67
5-year	3.46	3.52	1.77
10-year	4.64	4.72	2.36
25-year	9.57	9.74	4.88
50-year	14.03	14.28	7.16
100-year	20.15	20.49	10.28

These volumes were applied to the FLO-2D surface by filling the main channels first. Then decreasing the applied sediment depth at each cell as the location moved further away from the apex and the main channels in both the downstream and lateral directions. The applied sediment depth locations are included as a shapefile in the GIS folder of Appendix E.

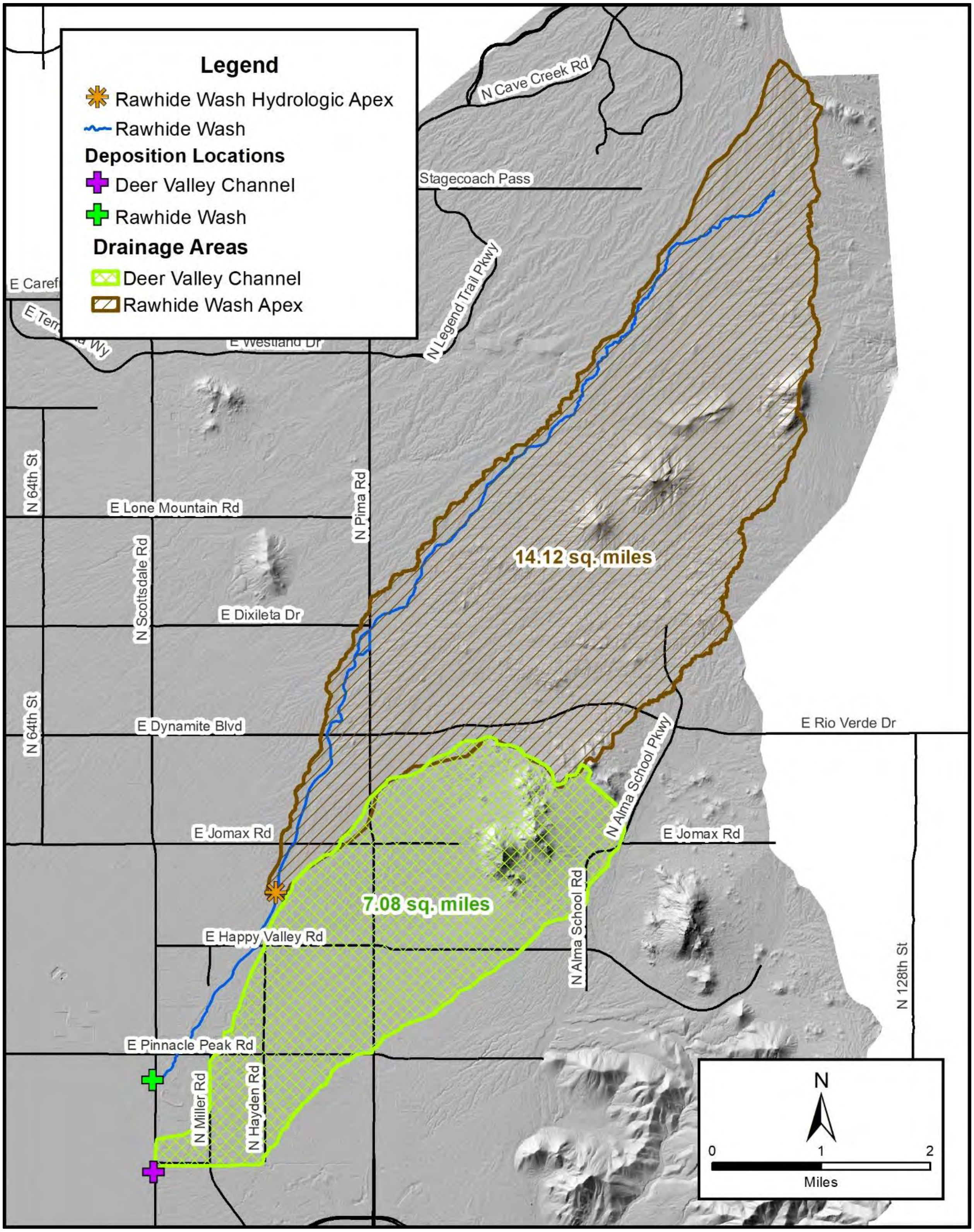


Figure 6-3. Spatial Locations of Watershed Areas and Deposition Locations for the Six Deposition Scenarios

6.2 Results

The maximum depth results from each scenario (Scenarios B-F) and the existing topography (Scenario A) from the Reach 2 FLO-2D model were combined into worst-case maximum depth and worst-case maximum velocity rasters using the mosaic tool in ArcGIS. These worst-case rasters were used to delineate the areal extent (or maximum width) of the flow that leaves the leveed section of Reach 1 at the moved apex location. The worst-case depth and velocity rasters are shown in Figure 6-4 and Figure 6-5, respectively. However, it should be noted that the FLO-2D results were only used as a guide for the revised floodplain. The results are not intended to be used for floodplain permitting purposes or for the design of any drainage structures. The detailed steps on how the delineation was revised are enumerated in the next subsection.

6.2.1 Steps Used for Redelineation of Reach 2

The following steps list the procedure that was used to delineate the floodplain for the Reach 2 area.

1) Delineate a floodplain area based on combined maximum depth from the six sediment deposition scenarios.
2) Separate this floodplain area into multiple zones based on smoothed 10-ft contours.
3) Apply zonal statistics with these zones to the combined maximum depths and maximum velocities to obtain the average values for each of these zones.
4) Intersect with the Effective FEMA floodplain shapefile.
5) Compare with average depths from step 3 with existing FEMA depth and velocity and use the higher of the two.
6) Combine zones into regions with the same velocity and depths.
7) Calculate new average values from the combined FLO-2D with these new zones for reference.
8) Delineate a separate AE zone upstream of the Scottsdale road culvert and immediately downstream of the culvert to show higher risk from depths greater than 5 feet.

The resulting AO zones (with depth and velocity components) are shown on the work maps as Exhibit F.

6.2.1 Redelineation of Reach 3

As was mentioned in Section 5.5.9, two sets of hydrographs were developed as inflows to Reach 3. First, since Reach 3 is an extension of the Rawhide Wash alluvial fan, the worst-case maximum discharges through the L101 culverts in the Reach 2 modeling were used to develop inflow hydrographs to the Reach 3 modeling. Second, these worst-case hydrographs were scaled to match the original design flows of each culvert within the revised AO zone to provide a second modeling scenario for the Reach 3 model. A comparison of the peak inflows for the two scenarios, the culvert size, and stationing from the design as-builts are shown in Table 6-2, while the spatial locations of the culverts in relation to Reaches 2 and 3 are shown in Figure 6-6. Only the highlighted culverts were used as inflows to Reach 3 since the majority of flow reached these culverts based on the results of the Reach 2 modeling scenarios.

The results from these two scenarios were combined to provide worst-case rasters of these two scenarios. These worst-case results were used to define the lateral extents (i.e., width) of the revised AO zone within Reach 3. The resulting AO zones (with depth and velocity components) are shown on the work maps as Exhibit F.

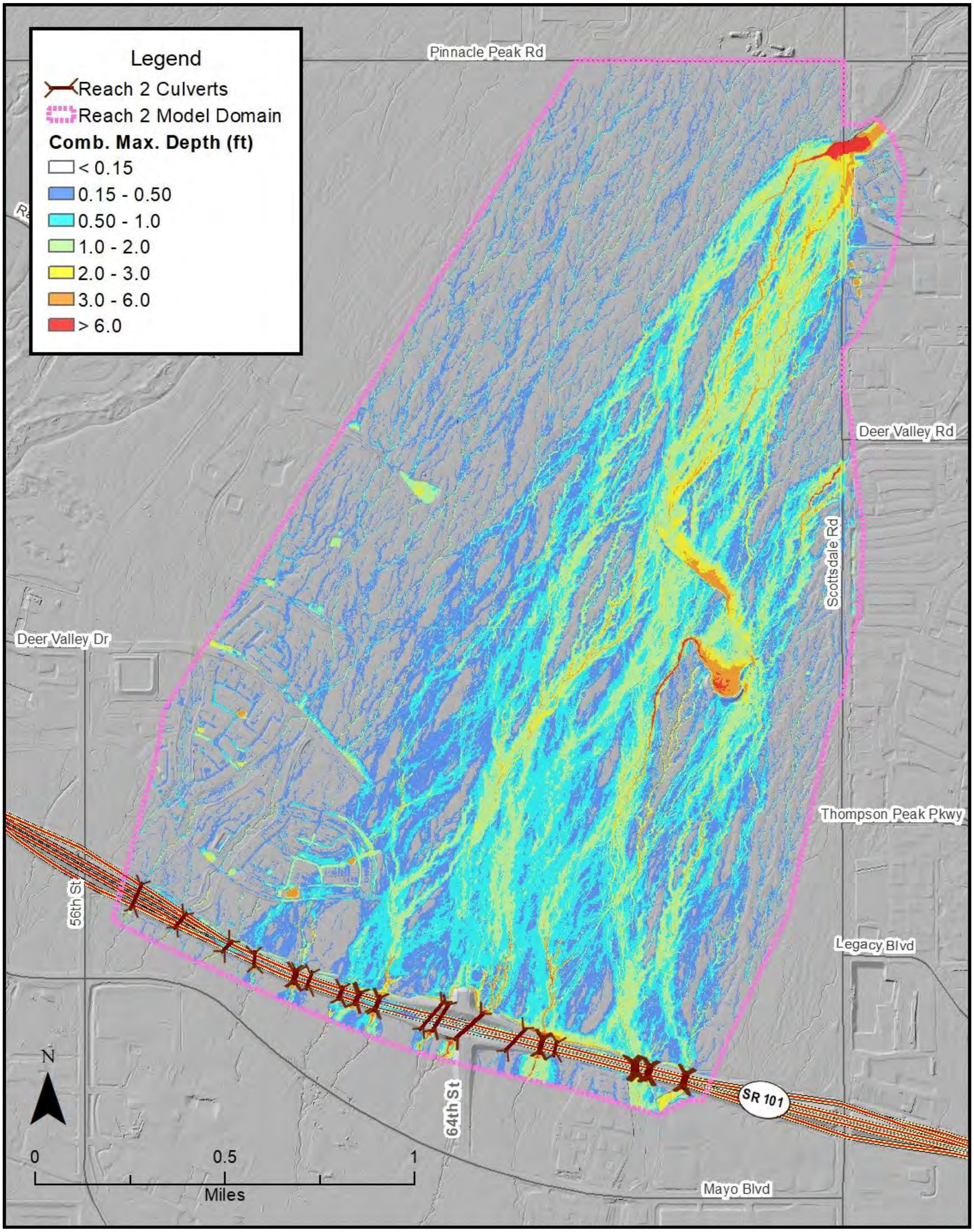


Figure 6-4. Combined Maximum Depths from All Six Scenarios

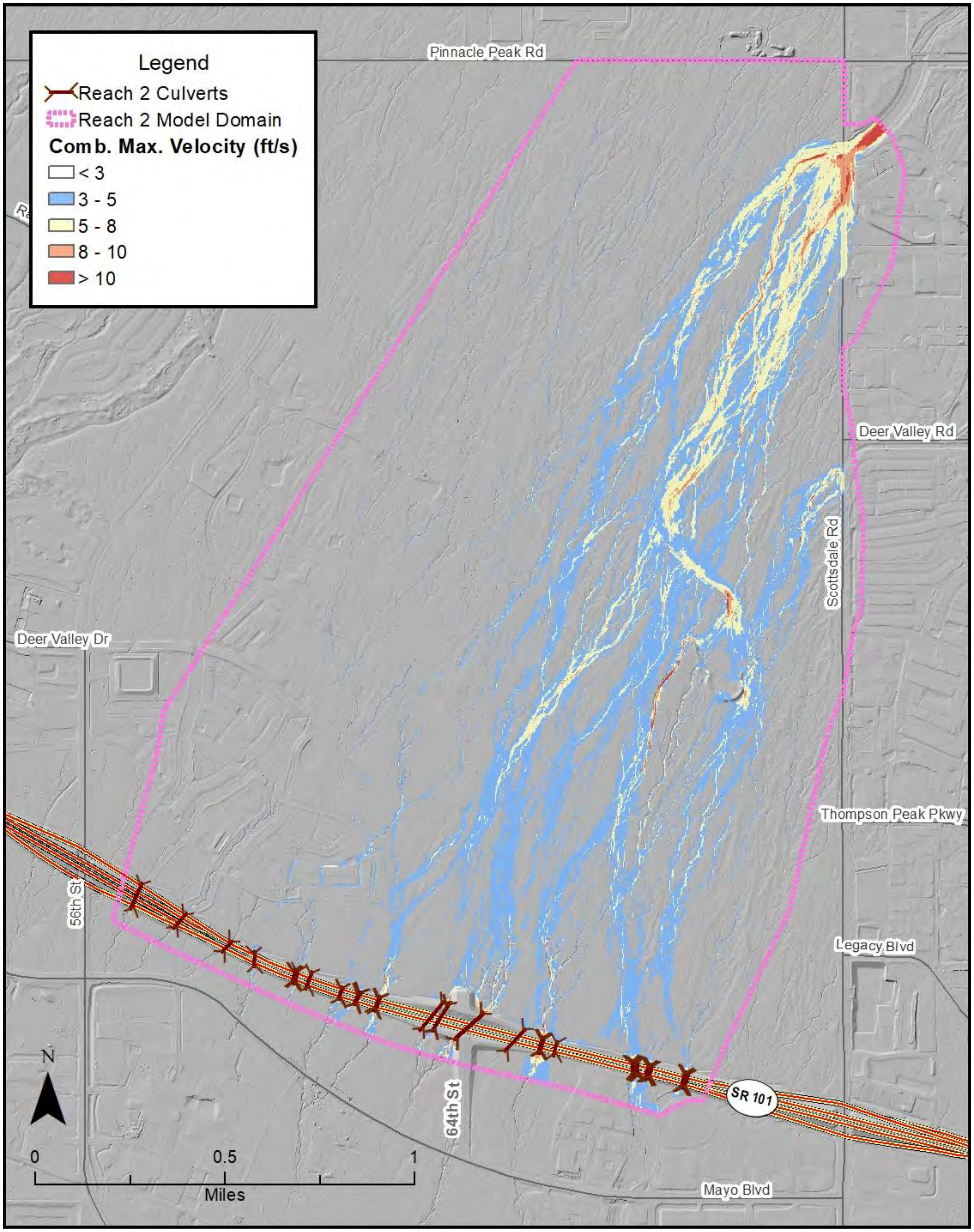


Figure 6-5. Combined Maximum Velocities from All Six Scenarios

Table 6-2. Culvert Inflows for the Two Reach 3 Scenarios

Number	Station	Name	Size	Maximum FLO2D Results (cfs)	Design Flow Rate (cfs)
1	53+244	S-2.1	3-6x6 RCBC	-	565
2	53+390	S-2.2	2-6x6 RCBC	26	428
3	53+604	S-2.3	3-6x6 RCBC	37	701
4	53+831	S-2.4	1-6x6 RCBC	36	137
5	53+955	S-2.5	2-6x6 RCBC	237	393
6	54+143	S-2.6	6-8x7 RCBC	268	2321
7	54+350	S-2.7	3-6x6 RCBC	295	619
8	54+424	S-2.8	5-6x6 RCBC	468	1026
9	54+511	S-2.9	3-8x8 RCBC	1130	1403
10	54+793	S-2.10	3-8x8 RCBC	779	1451
11	54+910	S-2.11	4-8x8 RCBC	1992	1741
12	55+230	S-2.12	6-8x6 RCBC	2693	1886
13	55+640	S-2.13	8-10x6 RCBC	1049	3542
14	55+700	S-2.14	6-10x6 RCBC	555	2498
15	55+856	S-2.15	6-10x6 RCBC	573	2377
16	56+045	S-2.16	1-6x6 RCBC	-	250
17	56+162	S-2.17	5-6x6 RCBC	-	1004
18	56+297	S-2.18	1-6x6 RCBC	-	250
19	56+415	S-2.19	1-6x6 RCBC	-	189
20	-	D-2.55.1	1-36" RCP	10	-
21	-	D-2.55.2	1-36" RCP	33	-
22	-	D-2.55.3	1-36" RCP	47	-
23	-	D-2.56.1	1-36" RCP	49	-
Note: Only blue highlighted culverts were included as Reach 3 inflows					

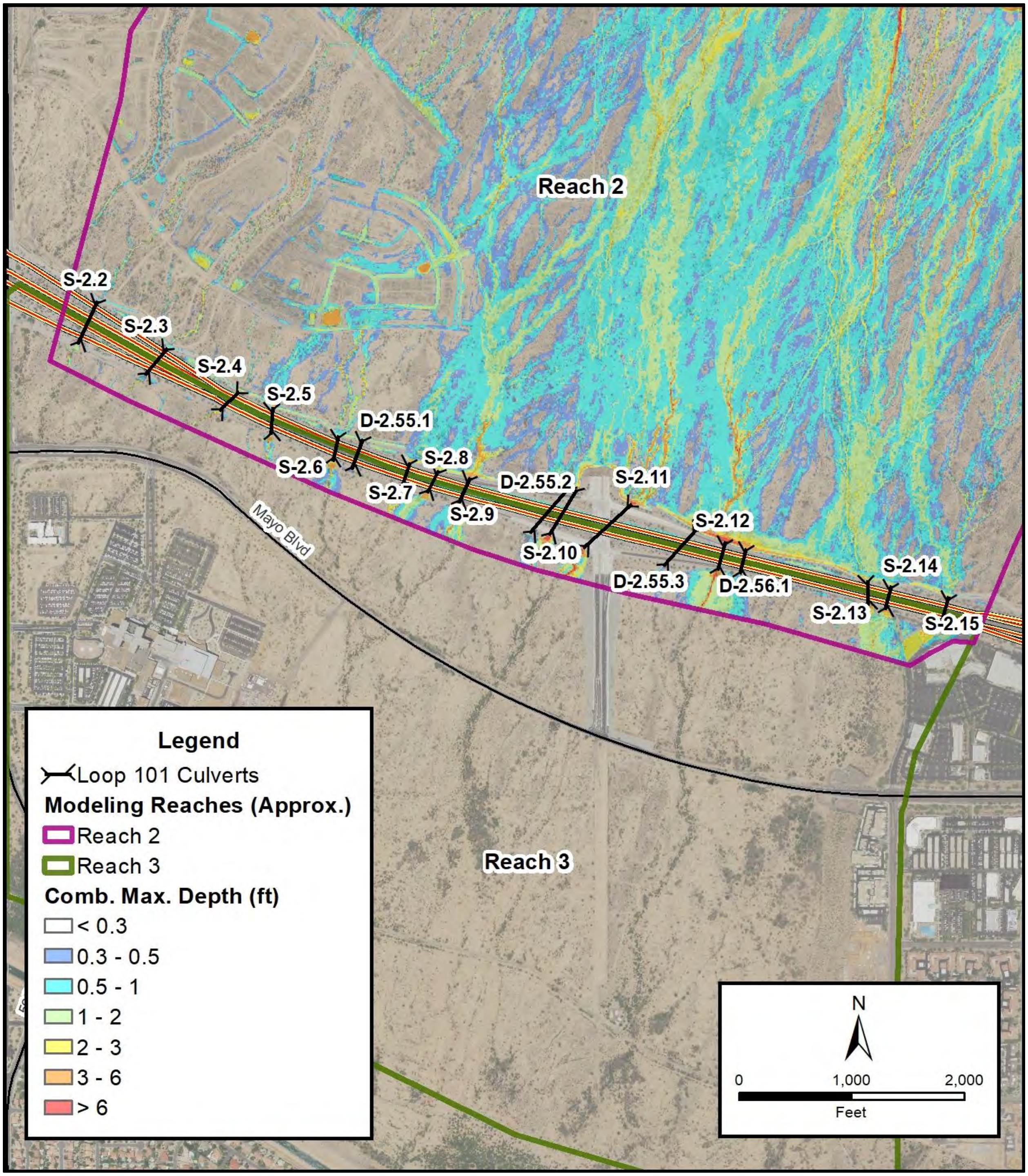


Figure 6-6. Spatial Locations of L101 Culverts Used as Inflows for Reach 3

7 Draft FIS Data

7.1 Summary of Discharges

Table 7-1 presents the recommended regulatory discharges within the floodplain revision area.

Table 7-1. Summary of Discharges for Rawhide Wash

Flooding Source and Location	Drainage Area (sq. mi.)	Peak Discharges (cfs)			
		10-year	50-year	100-year	500-year[1]
Rawhide Wash Upstream of East Happy Valley Road	14.1	2,410	6,683	9,105	-
Rawhide Wash at Miller Road alignment	14.2	2,342	6,615	9,033	-
Rawhide Wash upstream of Scottsdale Road	14.3	2,329	6,609	9,026	-
Deer Valley Road Channel at Scottsdale Road	7.1	-[1]	-[1]	1,486	-

[1] Data not available.

7.2 Floodway Data

No floodway analyses were performed as a part of this study.

7.3 Annotated Flood Insurance Rate Maps (FIRMs)

Annotated FIRMs based on proposed conditions are included as Exhibit G.

7.4 Flood Profiles

Flood profiles were not developed as a part of the CLOMR submittal but can be developed as a part of the LOMR.

Appendix A: References

Civil & Environmental Consultants, Inc. (CEC), 2021a, Rawhide Wash Flood Hazard Mitigation Project, Analysis of Existing Floodwalls, Technical & Supporting Calculations.

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Paretti, N. V., Kennedy, J. R., Turney, L. A., & Veilleux, A. G., 2014, Methods for Estimating Magnitude and Frequency of Floods in Arizona, Developed with Unregulated and Rural Peak Flow Data through Water Year 2010, Scientific Investigations Report 2014-5211. US Geological Survey.

TY Lin International (TY Lin), 2013, Pinnacle Peak South Area Drainage Master Study, Draft Hydrology and Hydraulics Report, Volumes 1 and 2.

Appendix B: General Documentation and Correspondence (Digital)

ESA Documentation

Pinnacle Peak South ADMS Reports

Pinnacle Peak West ADMS Reports

FEMA Correspondence

Appendix C: Survey Field Notes (Digital)

LiDAR Data

Reach 1 As-builts and Design Plans

Reach 2 As-builts, Construction Plans, and Drainage Reports

Reach 3 As-builts and Construction Plans

Appendix D: Hydrologic Analysis Supporting Documentation (Digital)

FLO-2D Models

Generalized HEC-1 Models (for calibration)

FCDMC Gage Data

GIS Data

Culvert Supporting Data (HY-8, etc.)

Appendix E: Hydraulic Analysis Supporting Documentation (Digital)

FLO-2D Models (Reach 1, Reach 2, and Reach 3)

HEC-1 Model (Reach 4)

GIS Data (Floodplain Shapefiles, Misc. Supporting Shapefiles)

Relevant Culvert Supporting Data (HY-8, Excel, etc.)

Appendix F: Erosion, Sediment Transport, and Geomorphic Analysis Supporting Documentation (Digital)

HEC-6T

GIS Data

Appendix G: Supporting Design Documents, Report, and GIS Files (Digital)

Rawhide Wash Flood Hazard Mitigation Construction Plans

Rawhide Wash Flood Hazard Mitigation Design Data Report and Appendices

Miller Road Bridge Construction Plans

GIS Files (e.g., levee stationing, etc.)

Appendix H: MT-2, Section 6 Supporting Index Maps, Plans and Profiles (Digital)

Rawhide Wash Flood Hazard Mitigation Project Levee System Index Map

Rawhide Wash Flood Hazard Mitigation Project Levee System Plan and Profile Maps

Exhibit Maps (Digital)

Exhibit A – General Watershed Map

Exhibit B – Soils Map

Exhibit C – Land Use (Surface Features) Map

Exhibit D.1 – 100-year 24-hour Maximum Discharge (Existing Conditions) Map – Hydrology Model

Exhibit D.2 – 100-year 24-hour Maximum Discharge (Proposed Conditions) Map – Hydrology Model

Exhibit D.3 – 100-year 24-hour Maximum Discharge (Proposed Conditions) Map – Hydraulics Models

Exhibit E.1 – 100-year 24-hour Maximum Depth (Existing Conditions) Map – Hydrology Model

Exhibit E.2 – 100-year 24-hour Maximum Depth (Proposed Conditions) Map – Hydrology Model

Exhibit E.3 – 100-year 24-hour Maximum Depth (Proposed Conditions) Map – Hydraulics Models

Exhibit F – Floodplain Work Maps

Exhibit G – Annotated FIRM Panels