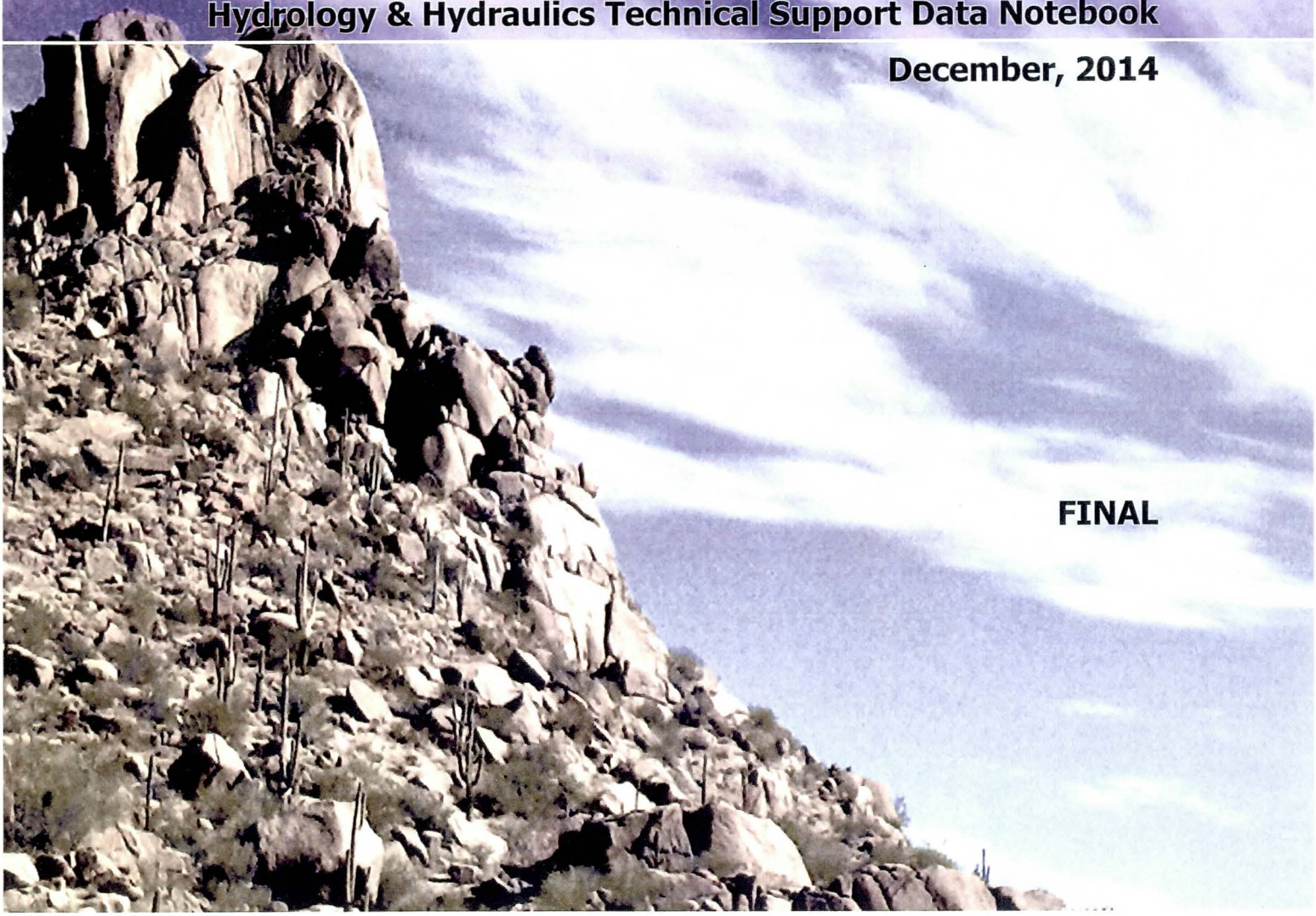
Flood Control District of Maricopa County

Pinnacle Peak West Area Drainage Master Study
PCN 122.01.20 FCD 2011C024 WA #3
Hydrology & Hydraulics Technical Support Data Notebook

December, 2014



FINAL

Pinnacle Peak West Area Drainage Master Study

PCN 122.01.20 FCD 2011C024 WA#3

Hydrology and Hydraulics Technical Support Data Notebook

Prepared For:

Flood Control District of Maricopa County

FINAL SUBMITTAL

Registered Professional Engineer (Civil) CERTIFICATE NO. 14720 PATRICIA K. QUINN Date Signed 12/17/2014 ARIZONA, U. S. A. Expires 09/30/2017

Registered Professional Engineer (Civil) CERTIFICATE NO. 49827 NATHAN B. LOGAN Date Signed 12/17/2014 ARIZONA, U. S. A. Expires 06/30/2015

Prepared By:

JE FULLER HYDROLOGY & GEOMORPHOLOGY, INC.

In Association with:

Stantec

December 17, 2014

Flood Control District
of Maricopa County

INTEROFFICE MEMORANDUM

Date: April 2, 2015

To: William D. Wiley, P.E., Chief Engineer and General Manager

From: Theresa Pinto

Subject: Pinnacle Peak West Area Drainage Master Study – Hydrology and Hydraulic Models and Results

The hydrology and hydraulic modeling for the Pinnacle Peak West Area Drainage Master Study (PPW ADMS) is complete and available to be used for this study area. The model results are based on the best available data at the time the model was developed, and standard modeling practices, assumptions, and engineering judgment. The models and results were thoroughly reviewed and approved by staff within the District's Engineering Division and Planning and Project Management Divisions.

The hydrology and hydraulic models were developed to identify flood hazards and risks in the PPW ADMS area. If the model results are used for other purposes, it is the user's responsibility to check the results for accuracy and applicability to their purpose. Furthermore, the results do not supersede or negate FEMA effective floodplains or any local, state, or federal floodplain or drainage regulatory requirements.

The results, models, and associated reports are available in the District's library. The report is titled "Pinnacle Peak West Area Drainage Master Study Hydrology and Hydraulics Technical Support Data Notebook December, 2014". The results and reports will also be available online in Spring/Summer 2015.

By signing below, you accept and approve the use of the PPW ADMS model and results as described herein.

Date: 4-2-15 Theresa Pinto, AICP, PMP Project Manager	Date: 4/6/15 William D. Wiley, P.E., Chief Engineer and General Manager
Date: 4/3/15 Catherine Regester, P.E. Hydrology/Hydraulics Branch Manager	Date: 4/6/15 Kelli Sertich, AICP Floodplain Management & Services Division Manager
Date: 4/6/15 Scott Vogel, P.E. Engineering Division Manager	Date: 4/3/15 Don Rerick, P.E. Planning and Project Management Division Manager

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TABLE OF CONTENTS

1. INTRODUCTION 7
   1.1. Purpose of Study 7
   1.2. Authority for Study 7
   1.3. Location of Study 8
   1.4. Methodology 11
   1.5. Acknowledgments 14
   1.6. Project Description 14
2. LOCAL GOVERNMENT ABSTRACT 16
3. SURVEY & MAPPING INFORMATION 19
   3.1. Digital Projection Information 19
   3.2. Field Survey Information 19
   3.3. Mapping 19
4. HYDROLOGY 22
   4.1. Method Description 22
   4.2. FLO-2D Model Development and Parameter Estimation 22
      4.2.1. Grid Element Size and Elevations (CADPTS.DAT and FPLAIN.DAT elevations) 23
      4.2.2. Watershed Workmaps 25
      4.2.3. Gage Data 25
      4.2.4. Grid Element Elevations Adjustments 28
      4.2.5. Rainfall/Precipitation Data (RAIN.DAT) 34
      4.2.6. Infiltration (INFIL.DAT) 40
      4.2.7. Outflow nodes (OUTFLOW.DAT) 51
   4.3. Off-Site Inflows 52
      4.3.1. Carefree DMP Inflow Hydrographs 52
      4.3.2. Pinnacle Peak South (PPS) ADMS Inflow Hydrographs 55
   4.4. Floodplain Cross-sections (FPXSEC.DAT) 58
   4.5. Model Control Parameters (CONT.DAT and TOLER.DAT) 58
   4.6. Issues Encountered During the Study 60
   4.7. Special Issues and Solutions 60
      4.7.1. Central Arizona Project Canal 60
      4.7.2. Sub-Area R-11 (Reach-11 Dikes) Model 61
   4.8. Modeling Warning and Error Messages 61

Registered Professional Engineer (Civil) CERTIFICATE NO. 14720 PATRICIA K. QUINN Date Signed 12/17/2014 ARIZONA, U. S. A. Expires 09/30/2017

Registered Professional Engineer (Civil) CERTIFICATE NO. 49827 NATHAN B. LOGAN Date Signed 12/17/2014 ARIZONA, U. S. A. Expires 06/30/2015

TABLE OF CONTENTS

4.9. Calibration 63

4.10. Final Results 64

4.11. Verification of Results 64

4.11.1. FEMA FIS DATA 64

4.11.2. Comparison of PPW ADMS Results to Other Studies 68

4.11.3. Indirect Methods Verification 73

4.11.4. Gage Data and recorded Storm Event Verification 77

5. HYDRAULICS 78

5.1. Method Description 78

5.2. Work Study Maps 78

5.3. Parameter Estimation 78

5.3.1. Roughness Coefficients 78

5.3.2. Grid Element Roughness Adjustments 81

5.3.3. Expansion and Contraction Coefficients 81

5.3.4. Grid Element Width and Area Reduction Factor (ARF.DAT) 81

5.4. Cross Section Description 81

5.5. Modeling Considerations 82

5.5.1. Hydraulic Jump and Drop Analysis 82

5.5.2. Bridges and Culverts (HYSTRUC.DAT) 82

5.5.3. Property Walls 93

5.5.4. One-Dimensional Channels (CHAN.DAT) 98

5.6. Floodway Modeling 101

5.7. Issues Encountered During the Study 101

5.7.1. Special Issues and Solutions 101

5.7.2. Modeling Warning and Error Messages 107

5.8. Calibration 107

5.9. Final Results 107

5.9.1. Hydraulic Analysis Results 107

5.9.2. Verification or Comparison of Results 108

6. EROSION, SEDIMENT TRANSPORT, & GEOMORPHIC ANALYSIS 109

6.1. Method Description 109

TABLE OF CONTENTS

List of Tables

Table 1. Flood Control District of Maricopa County Contact and Contract Information. 7
Table 2. Consulting Firm Contact Information. 8
Table 3. Townships, Ranges, and Sections Defining the Project Area[1]. 8
Table 4. PPW ADMS FLO-2D Model Sub-area Nomenclature Legend 11
Table 5. PPW ADMS Internal FLO-2D Model Sub-area Outflow-to-Inflow Relationship 14
Table 6. Mapping Data Information 19
Table 7. FLO-2D Sub-Area Grid Elements. 23
Table 8. Gages located within the PPW ADMS Study Area 26
Table 9. PPW FLO-2D Sub-Area Maximum Point Rainfall Depths 35
Table 10. HEC-1 Model Parameters. 42
Table 11. FLO-2D Limiting Depth Values per Sub-Area. 43
Table 12. FLO-2D Limiting Depth Calibration. 43
Table 13. PPW Soils Data Parameters. 44
Table 14. PPW Surface Feature Characterization Parameters. 47
Table 15. PPW Outflow-to-Inflow Relationships 51
Table 16. Comparison of Point Rainfall Depths for the Carefree ADMP HEC-1 Model 53
Table 17. HEC-1 Inflow Peak Discharges 54
Table 18. District Ratios to 100-Year Flood Hydrographs 56
Table 19. Comparison of Peak Discharges and Time to Peaks 59
Table 20. Summary of FLO-2D Control Parameters 60
Table 21. Summary of FEMA FIS discharges compared to PPW Results 64
Table 22. Summary of Existing Studies used to Verify Results 68
Table 23. PPW Model Results compared to Cave Creek FDS Results 69
Table 24. PPW Model Results compared to Rawhide Wash FDS Results 71
Table 25. PPW Model Results compared to North Scottsdale FDS Results 72
Table 26. FCDMC Stream Gage Data Summary 77
Table 27. Assigned n-values Based on Depth of Ponding. 81
Table 28. Number of Hydraulic Structures by Model Area 84
Table 29. Additional Clogging Factor Applied to Severely Clogged Culverts 90
Table 30. Number of Storm Drains by Model Area 91
Table 31. 1-Dimensional Channel Mileage by Model Area 99

TABLE OF CONTENTS

List of Figures

Figure 1. PPW ADMS Vicinity Map 10
Figure 2. PPW ADMS FLO-2D Sub-Area Interaction 13
Figure 3. PPW ADMS Mapping Data Sets 21
Figure 4. Typical Vegetation of the Upper Watershed 24
Figure 5. Typical Vegetation of the Lower Watershed 25
Figure 6. PPW ADMS Gage Locations 27
Figure 7. Rawhide Wash Looking Upstream at Pinnacle Peak Road Bridge – Photograph Dated June 2013 29
Figure 8. Rawhide Wash Existing Interim Channel Looking Downstream From Pinnacle Peak Road Bridge – Photograph Dated June 2013 30
Figure 9. Rawhide Wash Looking Upstream at the Scottsdale Road Bridge Construction – Photograph Dated June 2013 30
Figure 10. Rawhide Wash Proposed Channel Contours Between Pinnacle Peak Road and Scottsdale Road – Note the Existing Contours Show the Existing Interim Channel Downstream of the Pinnacle Peak Road Bridge 31
Figure 11. Rawhide Wash Interim Channel downstream of Scottsdale Road – Photograph Dated February 2013 32
Figure 12. Sonoran Boulevard Vertcon Adjustment Location 33
Figure 13. 100-year 24-hour Rainfall Depths 36
Figure 14. 25-year 24-hour Rainfall Depths 37
Figure 15. 10-year 24-hour Rainfall Depths 38
Figure 16. SCS Type II 24-Hour Temporal Distribution 39
Figure 17. Soils Survey Coverage Over the PPW Watershed 46
Figure 18. Surface Feature Dataset Areas Merged and Digitized to Stitch Datasets Together 49
Figure 19. Existing FEMA Floodplains 50
Figure 20. PPW Internal Boundary Inflow/Outflow 52
Figure 21. Carefree DMP HEC-I Sub-basins and Inflow Locations 53
Figure 22. Carefree DMP HEC-I Model Area-NOAA Atlas 14 54
Figure 23. 100-year 24-hour inflow hydrographs from Unnamed Central Tributary 55
Figure 24. PPS/PPW Boundary with Additional Outflow Nodes 57
Figure 25. PPS/PPW CADPTS Alignment 58
Figure 26. Sub-Critical and Super-Critical Hydrograph Comparison 59
Figure 27. Cave Creek FDS Sub basins with FLO-2D Depth Results 70
Figure 28. Indirect Verification Method 1 74
Figure 29. Indirect Verification Method 2 75
Figure 30. Indirect Verification Method 3 76
Figure 31. Upper and Lower Undeveloped Desert Roughness Divide 80
Figure 32. Model Hydraulic Structure Locations 83
Figure 33. Blended Rating Curve 89
Figure 34 Model Storm Drains Locations 92
Figure 35. Modeled Walls Not Part of the 3D Data 95
Figure 36. Initial Coverage of 3D Lines Including Insignificant Walls 96
Figure 37. Modeled (Green) and Non-Modeled (Red) Walls 97
Figure 38. Method of Determining Property Wall Failure Elevation 98
Figure 39. Location of 1D Channels 100

Figure 40. Just North of Loop 101 102
Figure 41. Just North of Loop 101 103
Figure 42. Just North of Loop 101 104
Figure 43. Hydrograph Channel Segment 353 105
Figure 44. Channel Segment near Tatum Blvd and L101 106
Figure 45. Hydrograph Channel Segment 519 106
Figure 46. Hydrograph Channel Segment 570 107

List of Appendices:

Appendix A: References

A.1 Data Collection Summary

- Design Plans for Topographic Adjustments
- Model Verification Data
- Work Assignment #1 Task 5 Hydrology and Hydraulics Report
- Work Assignment #1 Task 6.1 Alluvial Fan Landform Verification Memorandum

A.2 Referenced Documents

Appendix B: General Documentation and Correspondence

B.1 General Project Documentation and Correspondence

- FCDMC Review Comments and JEF Responses

B.2 Contract Documents

Appendix C: Survey Field Notes

C.1 Survey Field Notes For Hydraulic Modeling

- Wall Openings, Culvert, and Storm Drain Survey Information

Appendix D: Hydrologic Analysis Supporting Documentation

D.1 HEC-1 Model Input/Output

- Offsite Inflow Carefree DMP Models
- Calibration Model

D.2 Floodplain Cross Section Summary Tables and Hydrograph Plots

Appendix E: Hydraulic Analysis Supporting Documentation

E.1 Analysis of Structures

- HY-8 Analysis, Wall Opening Analysis, Other

E.2 Hydraulic Structure Rating Curves, Summaries, and Hydrograph Plots

- Culvert Performance Plots

E.3 One-Dimensional Channel Output Profiles and Hydrographs

- Profiles
- Cross-Section Plots

Exhibit Maps

- General Watershed Terrain
- Infiltration Exhibits (XKSAT, PSIF, DTHETA, IA, RTIMP, and Limiting Infiltration Depths)

- Surface Feature Characterization (Existing Land Use Data)
- Soils Data
- ARF Coverage
- Maximum Flow Depths
    - With-Wall and Failure
    - With-Wall and No Failure
    - Without Wall
- Maximum Discharge and Floodplain Cross Sections
    - With-Wall and Failure
    - With-Wall and No Failure
    - Without Wall
- Maximum Velocities
    - With-Wall and Failure
    - With-Wall and No Failure
    - Without Wall

Digital Data (USB Drive)

Technical data Notebook Volumes:

Volume 1 **TDN Text**

Volume 2 **Appendix A through Appendix D**

Volume 3 **Appendix E.1 through Appendix E.2**

Volume 4 **Appendix E.3 (Without Property Wall Model)**

Volume 5 **Appendix E.3 (With Property Wall Model and No Failure Model)**

Volume 6 **Exhibit Maps**

1. INTRODUCTION

1.1. Purpose of Study

The Pinnacle Peak West (PPW) Area Drainage Master Study (ADMS) will identify and evaluate flood hazards in the study area by implementing a work plan which includes data collection; review of previous planning and engineering studies; information gathering and sharing from/to project partners, stakeholders, and the public; hydrologic and hydraulic modeling; geomorphologic assessments; field surveys; landscape architecture; and environmental overview.

This report covers the hydrologic and hydraulic modeling of the watershed, the primary goal of the modeling component of the PPW ADMS is to update and characterize the flood hazard using current detailed topography, updated precipitation data, and two-dimensional modeling methodologies. Based on this updated understanding of the flooding hazard, this project may include formulation of a flood hazard mitigation strategy to address the identified flooding hazards. The results can also be used as input to the planning and design of drainage infrastructure and flood mitigation measures that are appropriate for the physical environment for both existing and future development.

The results of this hydrologic and hydraulic analyses will be used to:

- More accurately characterize the location and extent of the existing flood hazards in the study area;
- Determine the adequacy of current and proposed drainage infrastructure;
- Plan and design future drainage infrastructure;
- Determine if there are practicable mitigation solutions that can reduce all or part of the flood hazard risk; and
- Compare to the effective FEMA floodplains and determine if additional floodplains should be delineated or if the existing floodplains should be redelineated.

1.2. Authority for Study

The Flood Control District of Maricopa County (District) has retained JE Fuller Hydrology and Geomorphology, Inc. (JEF) for completion of the PPW ADMS project. The District's contact and contract information is provided below in Table 1. The primary and sub-consulting firm contact information is provided below in Table 2.

Table 1. Flood Control District of Maricopa County Contact and Contract Information.

Authorizing Agency	***Flood Control District of Maricopa County (District)***
Contact Information	**Theresa Pinto**, AICP, CFM, PMP; Project Manager 2801 W Durango St., Phoenix, AZ 85009 602-506-8127 tmp@mail.maricopa.gov
Contract	Contract FCD 2011C024
Study Duration	Start Date: March 19, 2012; End Date: September 30, 2015

Table 2. Consulting Firm Contact Information.

Primary Consulting Firm	***JE Fuller Hydrology & Geomorphology, Inc. (JEF)***
Contact Information	**Patricia K. Quinn**, PE, RLS, AVS; Project Manager 8400 S. Kyrene Rd, Ste. 201,Tempe, AZ 85284 480-222-5708 pat@jefuller.com
Sub-Consulting Firm	***Stantec (Culvert and 1-D Channel Inventory and Modeling)***
Contact Information	**Mike Gerlach**, PE, Associate-Water Resources 8211 S 48th St., Phoenix, AZ 85044 602-707-4695 mike.gerlach@stantec.com

1.3. Location of Study

The PPW ADMS project study area is 97 square miles in size and is located in the northeastern portion of Maricopa County and encompasses land within the jurisdiction of the City of Phoenix, City of Scottsdale, Town of Cave Creek, Town of Carefree, and unincorporated Maricopa County. The primary stakeholders affected by the project are the City of Phoenix, City of Scottsdale, Maricopa County, and Arizona State Land Department (ASLD). The project is bound by approximately the Carefree Highway and Cave Creek Road to the north, the Pinnacle Peak South (PPS) ADMS study area and drainage divide to the east, the Central Arizona Project (CAP) Reach 11 Dikes to the south, and Cave Creek Road and the eastern Cave Creek floodplain limits to the west. More specifically, the study area is located within the Townships, Ranges, and Sections listed below in Table 3. The study area location and limits are shown on Figure 1. It should be noted that the 97 square miles of the PPW study area includes a portion (approximately 9.7 square miles) of the PPS ADMS study area. The purpose of including the roughly 9.7 square miles of the PPS watershed in the PPW modeling area is to address the inflows from PPS into the PPW watershed, see Section 4.3.2.

Table 3. Townships, Ranges, and Sections Defining the Project Area[1].

Township	Range	Section
T03N	R04E	2, 3
T04N	R03E	1-3, 11-15, 23-25
T04N	R04E	1-30, 32-35
T04N	R05E	6, 7
T05N	R03E	12-14, 23-27, 34-36
T05N	R04E	1-36
T05N	R05E	2-11, 15-22, 27-33
T06N	R04E	36
T06N	R05E	21-22, 27-29, 31-35

[1]Located within the Gila and Salt River Meridian

The following effective FIRM Panels will be affected by this study. With the exception of panel 04013C0827F, all other panels are effective dated September 2005.

04013C1245H
04013C1230H
04013C1240H
04013C1210H
04013C1220J
04013C0795H
04013C0815J
04013C0820G
04013C0839F
04013C0838F
04013C1255G
04013C1235G
04013C0829F
04013C0837F
04013C0836F
04013C0827F*
04013C0828F
04013C0809J

* Panel not printed-Effective date April 1988

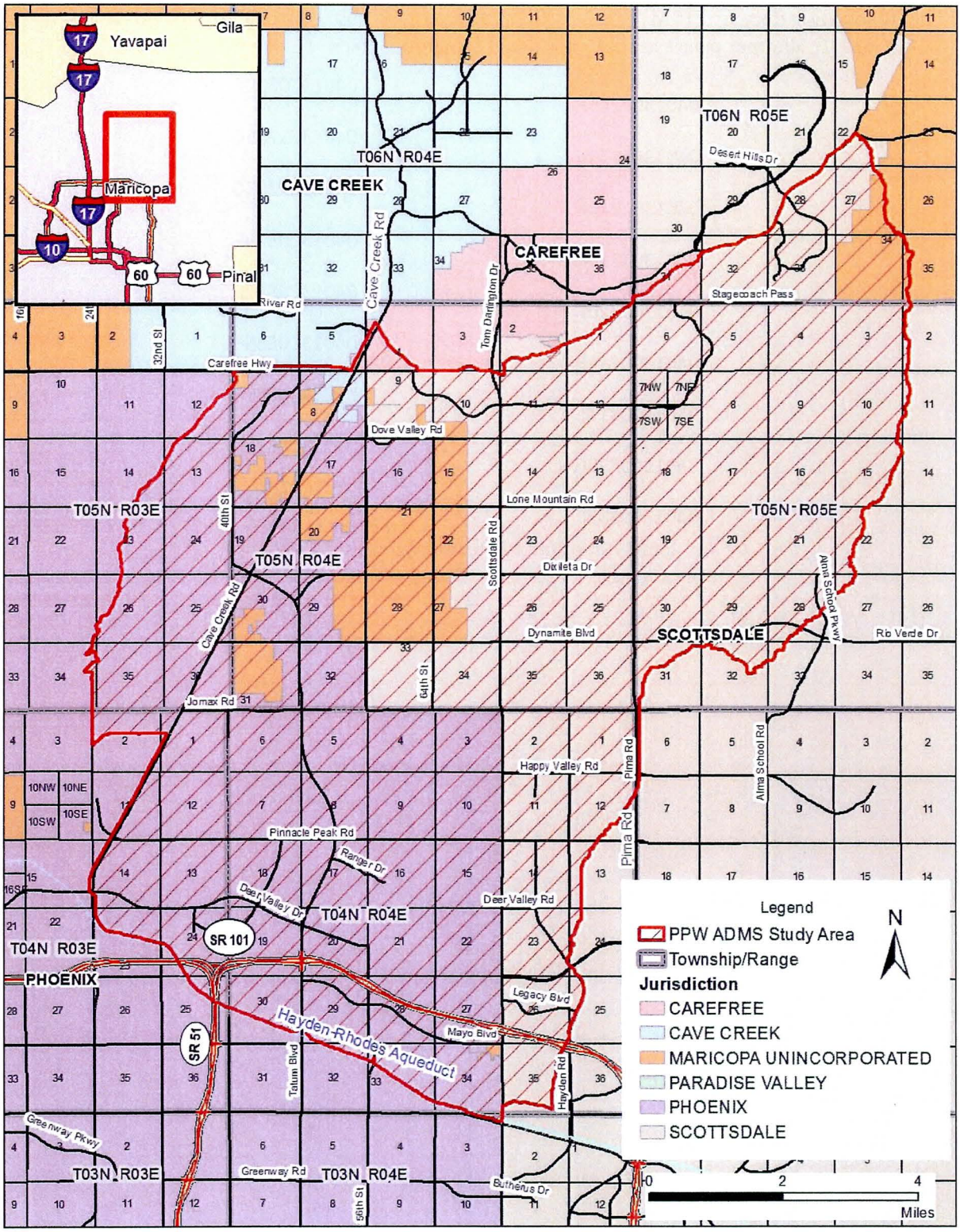

Figure 1. PPW ADMS Vicinity Map

1.4. Methodology

Hydrologic and hydraulic (H&H) modeling for the PPW ADMS Project has been completed with the use of FLO-2D (FLO-2D Software, Inc., Professional Version), a volume conserving, two-dimensional (2-D), flood routing model. The model routes flow (rainfall runoff and inflow hydrographs) over a grid comprised of square elements based on topography (defined by grid element elevations) and watershed roughness (Manning's n-value assigned to each grid element). This 2-D modeling approach is highly suited for simulating the shallow, distributary flow prevalent within the watershed as flow travels from northeast to southwest through shallow braided channels in the undeveloped areas and through streets and around building structures in the developed areas. The FLO-2D model also incorporates hydraulically significant culverts, walls, and channels within the model area. The models are developed using the existing land use conditions at the time of this report preparation.

The organization of this Technical Study Data Notebook (TSDN) generally follows the State Standard outline. However, given the integral nature of the hydrologic and hydraulic aspects of FLO-2D, the TSDN presents some of the information in Section 4 – Hydrology and some in Section 5 – Hydraulics.

The PPW watershed, which is 97.4 square miles, was subdivided into multiple model domains to represent seven primary sub-areas due to the large watershed size and the target grid cell size of 20 feet. Given the detail of the sub-area models, significant drainage features such as channels and basins (natural and man-made) are topographically well reflected. Additionally, the land use surface characteristics (streets, buildings, vegetation types etc.) are well defined through the use of smaller grid element sizes. Further detailed discussions regarding the FLO-2D grid development, the off-site inflows, and the interaction between the PPW internal sub-area models is provided in Section 4.2.

For this study, the FLO-2D models were simulated using the professional version of FLO-2D (FLO-2D PRO), Build No. 13.07.05 and an executable dated 9-10-2013 and have been developed with grids comprised of elements measuring 20 feet by 20 feet for each of the seven sub-areas. Flow is passed from upstream sub-area model to the downstream sub-area(s) on a cell-to-cell basis along the overlapping sub-area boundaries. The nomenclature for the sub-area naming is based off of prominent geographic features (e.g. Rawhide Wash) or master-planned communities (e.g. Desert Ridge) that lie within the vicinity of the sub-area domain. The prominent feature name and approximate model area for each sub-area model is listed in Table 4.

Table 4. PPW ADMS FLO-2D Model Sub-area Nomenclature Legend

Sub-Area ID	Prominent Feature Name	Area (mi^2)
Area LT	Legend Trail	12.0
Area UR	Upper Rawhide Wash	13.3
Area WR	Whisper Rock	15.1
Area TR	Tatum Ranch	15.7
Area LR	Lower Rawhide Wash	15.7
Area CB	Cave Buttes	9.9
Area DR	Desert Ridge	15.7
Area R-11*	Reach-11 Dikes	2.6**

*R-11 Model was developed to model the ponding of the Reach-11 Dikes upstream of the CAP canal.
**The area of the R-11 Model is included in the overlapping areas of LR and DR. The 2.6 square-mile area is not in addition to the total area.

Areas DR and LR overlap with an 8th model developed to simulate the ponding against the CAP canal and the Reach-11 dikes at the downstream end. The 8th model is named R-11 after the Reach-11 dikes and, as previously noted, the R-11 domain overlaps with the DR and LR model domains. The R-11 model was developed after the DR and LR models had been built. As a result, it was decided to leave the DR and LR models intact and set up outflow nodes along the intersecting boundary of the R-11, LR and DR domains and generate inflows from LR and DR into the R-11 model. The full model area of DR and LR account for the overlap between these modes and the R-11 domain. The modeling of the overlap area is discussed in more detail in the Special Issues and Solutions section of this report, Section 4.7.

Off-site flows enter the PPW ADMS watershed from two sources, the Unnamed Central Tributary to Cave Creek (Carefree Drainage Master Plan (DMP)) from the north, and distributary flows studied during the Pinnacle Peak South (PPS) ADMS from the southeast. The Carefree DMP was completed using HEC-1 and contributes flows to PPW sub-areas WR and TR. The PPS ADMS was completed using FLO-2D and contributes flows to PPW areas UR and LR. The off-site flows are described in more detail in Section 4.3.

The interaction of the FLO-2D models internal to PPW ADMS is addressed by using the FLO-2D outflow-to-inflow routine. This routine takes the outflow hydrographs from upstream models and generates inflow hydrographs to the downstream model. The upstream FLO-2D grid systems that flow into downstream grid systems are listed in Table 5 and shown graphically over the watershed with the Carefree DMP and PPS ADMS inflow locations in Figure 2. The inflow locations shown in Figure 2 are intended to show which models transfer flow to the others and not the specific locations where flow is transferred.

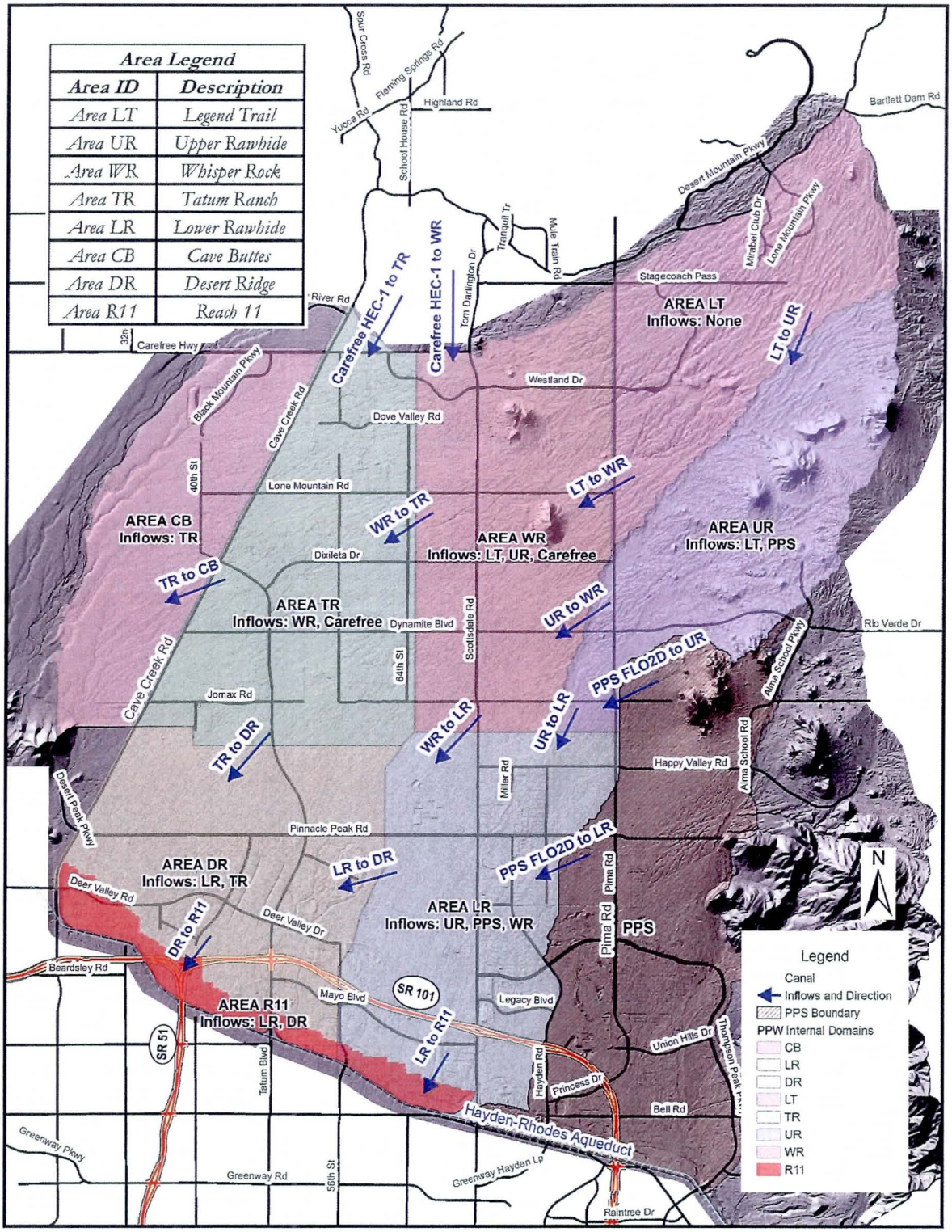

Figure 2. PPW ADMS FLO-2D Sub-Area Interaction

Table 5. PPW ADMS Internal FLO-2D Model Sub-area Outflow-to-Inflow Relationship

Upstream Grid	Downstream Grid
AREA LT	AREA UR
AREA LT	AREA WR
AREA UR	AREA WR
AREA UR	AREA LR
AREA WR	AREA TR
AREA WR	AREA LR
AREA TR	AREA CB
AREA TR	AREA DR
AREA LR	AREA DR
AREA LR	AREA R-11
AREA-DR	AREA R-11

Details on the FLO-2D modeling methodology are presented in Section 4.2. Additional information regarding the FLO-2D software can be found at https://www.flo-2d.com/.

1.5. Acknowledgments

The PPW ADMS was prepared for the Flood Control District of Maricopa County (FCDMC) by JE Fuller/Hydrology & Geomorphology, Inc. (JEF) under Contract FCD 2011C024. The contract was managed by Theresa Pinto, AICP, CFM, PMP, FCDMC Project Manager, with support from Doug Williams, AICP, Planning Branch Manager; Felicia Terry, PE, CFM, Assistant Project Manager; Tom Loomis, PE, RLS, CFM, Special Projects Branch Manager; Julie Cox; Richard Waskowsky, PE; and Amir Motamedi, PE. The project benefited greatly from the collective expertise of the entire FCDMC team.

The Project Manager for JEF was Pat Quinn, PE, RLS, AVS with support from Rob Lyons, PE, CFM, Project Engineer; Ted Lehman, PE, Project Hydrologist; Nathan Logan, PE, CFM; Hari Raghavan, PhD, PE, CFM; and Mike Gerlach, PE, Stantec Consulting, Inc.

The project partners include Ashley Couch, PE, CFM and Richard Anderson, PE, CFM with the City of Scottsdale, Hasan Mushtaq, PE, PhD, CFM with the City of Phoenix, and Mark Edelman, AICP and Scott Molstad, PE with the Arizona State Land Department (ASLD). The project partners provided valuable data and input throughout the duration of the project.

1.6. Project Description

The PPW ADMS will identify and evaluate flood hazards in the study area based on data collection and initial assessments using various methods and data/ information sources. The work plan includes data collection and review of previous planning and engineering studies; information gathering and sharing from/to project partners, stakeholders, and the public; hydrologic and hydraulic modeling; geomorphologic assessments; field surveys; landscape architecture; and environmental overview.

Using relatively recent topographic mapping data (Section 3.3) and updated precipitation data along with two-dimensional hydrologic and hydraulic modeling techniques, the PPW ADMS will provide an updated depiction of the flooding problems and hazards in the study area. Based on this updated understanding of the flooding hazard, this project may include formulation of a flood hazard mitigation strategy to address the identified hazards. Due to existing and pending development in the project area, this project may also include alternatives development and evaluation to mitigate the existing or potential flooding hazards in certain areas. The results can also be used to guide future development and as input to the planning and design of drainage infrastructure and flood mitigation measures that are appropriate for the physical environment for both existing and future development.

The PPW ADMS will use the Context Sensitive Flood Hazard Mitigation (CSFHM) planning and design approach to assist in the development of flood hazard mitigation plan alternatives. The purpose and intent of the application of the CSFHM Approach is to identify the range of possible flood hazard mitigation structural and non-structural solutions that have the potential ability to simultaneously perform the three functions of being Acceptable to local communities, Compatible with the land and resources and Effective in reducing flood hazards within the project study area.

The primary goals of the PPW ADMS are as follows:

- Identify and characterize the existing and potential flooding hazards in the study area based on current conditions;
- Assess the risk of the flooding hazards and categorize the flood hazards for mitigation consideration; and
- Develop consensus and support from the public, stakeholders, and project partners on the project's modeling results and subsequent hazard identification, classification, and mitigation where applicable.

2. LOCAL GOVERNMENT ABSTRACT

Study Documentation Abstract for Local Governmental Submittals		
Section 1: Project Contact Information		
1.1	Owner Contact Information - Mailing Address - Phone Number - E-mail Address	Theresa Pinto, AICP, CFM, PMP; Project Manager Flood Control District of Maricopa County 2801 W Durango St., Phoenix, AZ 85009 602-506-8127 tmp@mail.maricopa.gov
1.2	Study Contractor Contact Information - Mailing Address - Phone Number - E-mail Address	Patricia K. Quinn, PE, RLS, AVS; Project Manager JE Fuller Hydrology & Geomorphology, Inc. 8400 S. Kyrene Rd, Ste. 201,Tempe, AZ 85284 480-222-5708 pat@jefuller.com
1.3	Local Technical Reviewer - Mailing Address - Phone Number - E-mail Address	Julie Cox; Engineering Division, Technical Review Flood Control District of Maricopa County 2801 W Durango St., Phoenix, AZ 85009 602-506-8401 jrc@mail.maricopa.gov
1.4	Date Study Submitted	November 2014
1.5	Date Review Comments Returned	Review comment responses were returned throughout the duration of the project as they were received.
1.6	Date Study Approved by Local Reviewing Agencies FCDMC/COS/COP/ASLD	
Section 2: General Information		
Section 2.1: Project Location		
2.1.1	Community	Scottsdale, Phoenix, Carefree, Cave Creek, Unincorporated Maricopa County
2.1.2	County	Maricopa County
2.1.3	River or Stream Name	Rawhide Wash, Fan 5, Fan 6, Stagecoach Pass Wash, Eastern Pima Wash, Unnamed Central Tributary to Cave Creek, Cave Creek
2.1.4	Reach Description	
2.1.5	Study Type	Riverine, Distributary, Mountain, Ponding, Sheet Flooding, Alluvial Fan
Section 2.2: Project Purpose and Summary of Findings		
2.2.1	Purpose of the Study	To identify flood hazards within the project area
2.2.2	Summary of Hydrology and Hydraulic Methodologies Utilized	2-Dimensional combined hydrologic and hydraulic modeling with FLO-2D
2.2.3	Brief Summary Description of the Study Results	
2.2.4	Acknowledgements	

Section 3: Survey and Mapping Information		
3.1	Digital Projection Information Type/Source Coordinate System Date	All data was provided in or projected to North American Datum of 1983 (NAD 1983) in State Plane Central Coordinates, International Feet. All data was provided in or adjusted to the North American Vertical Datum of 1988 (NAVD88).
3.2	USGS Quad Sheet(s)	Not used for modeling
3.3	Mapping for Hydrologic Study Type/Source Scale Date	See Table 6
3.4	Mapping for Hydraulic Study Type/Source Scale Date Subcontractor Date of Aerial Mapping	See Table 6
Section 4: Hydrology		
4.1	Model or Method Used (including vendor and version description)	FLO-2D PRO (executable dated 09-10-2013) Build No.13.07.05
4.2	Storm Duration	24-Hour
4.3	Hydrograph Type	SCS Type II
4.4	Frequencies Determined	10-Year, 25-Year, & 100-Year
4.5	List of Gages Used in Frequency Analysis or Calibration (Location, Years of Record, Gage Ownership)	See Figure 6
4.6	Rainfall Amounts and Reference	Variable, see Table 9, Figure 13, and Figure 15
4.7	Unique Conditions and Issues	
4.8	Coordination of Discharges (Agency, Date, Comments)	
Section 5: Hydraulics		
5.1	Model or Method Used (including vendor and version description)	FLO-2D PRO (executable dated 09-10-2013)
5.2	Regime	Critical, limiting Froude (FROUDL) set to 1.0 (Section 4.5)
5.3	Frequencies for which profiles were computed	No profiles generated
5.4	Method of Floodway Calculation	No floodway calculated

5.5	Unique Conditions and Issues	
Section 6: Erosion, Sediment Transport, and Geomorphic Analysis		
6.1	Summary of Method	No Erosion, Sediment Transport, or Geomorphic analysis modeled.
6.2	Issues Encountered During Study	
6.3	Summary of Findings	
Section 7: Additional Study Information		
Item		**Description/Discussion**

3. SURVEY & MAPPING INFORMATION

3.1. Digital Projection Information

A Terrain Data Set (TDS) was developed using ArcGIS version 10.1 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)
- Projected Coordinate System:
  North American Datum of 1983 (NAD 1983) in State Plane Central Coordinates, International Feet. (NAD_1983_HARN_StatePlane_Arizona_Central_FIPS_0202_FEET_INTL)

3.2. Field Survey Information

Field survey conducted for this project was limited to hydraulic structures that can significantly affect flow characteristics such as culverts, engineered channels, wall openings, and limited storm drain infrastructure. The horizontal position of structures surveyed was obtained from a Trimble Juno 3D Hand Held 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, project mapping data products were used instead.

3.3. Mapping

Aerial mapping data covering the PPW ADMS watershed came from multiple mapping sources (Figure 3). The data for each mapping source was provided by the District in the form of mass-point and break-line data. The mapping data was used to develop the TDS which was in turn used to develop the FLO-2D grid element 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 6 below.

Table 6. Mapping Data Information

Project	Mapping ID	FCDMC Contract No.	Mapping Contour Interval	Flight Date	Vertical Datum
Pinnacle Peak ADMS	1311	09-44	2-foot	6/28/2010	NAVD88
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
Cave Creek Mapping	1268	FCD 05-30	2-foot	10/20/2004	NAVD88
Cave Creek Mapping	1254	FCD 03-48	2-foot	10/20/2004	NAVD88
Carefree Additional Mapping	1239	FCD 02-30	2-foot	8/20/2001	NGVD29
Carefree DMP	1184	FCD 00-37	2-foot	8/20/2001	NGVD29

Mapping area IDs 1239 and 1184 are on the National Geodetic Vertical Datum of 1929 (NGVD 29) whereas the rest of the mapping data is on North American Vertical Datum of 1988 (NAVD 88). An adjustment of the elevation data for mapping areas 1239 and 1184 was performed using the National Geodetic Survey (NGS) VERTCON data. A point near the center of the mapping areas in the northeast corner of Carefree Highway and Cave Creek Road intersection was chosen for the VERTCON adjustment location. The VERTCON adjustment factor for this area was 2.014 feet. This value was added to the base shapefile data (mass-point and break-line data) for mapping areas 1239 and 1184 to build the TDS. The point location used to compute the VERTCON adjustment factor is also shown on Figure 3.

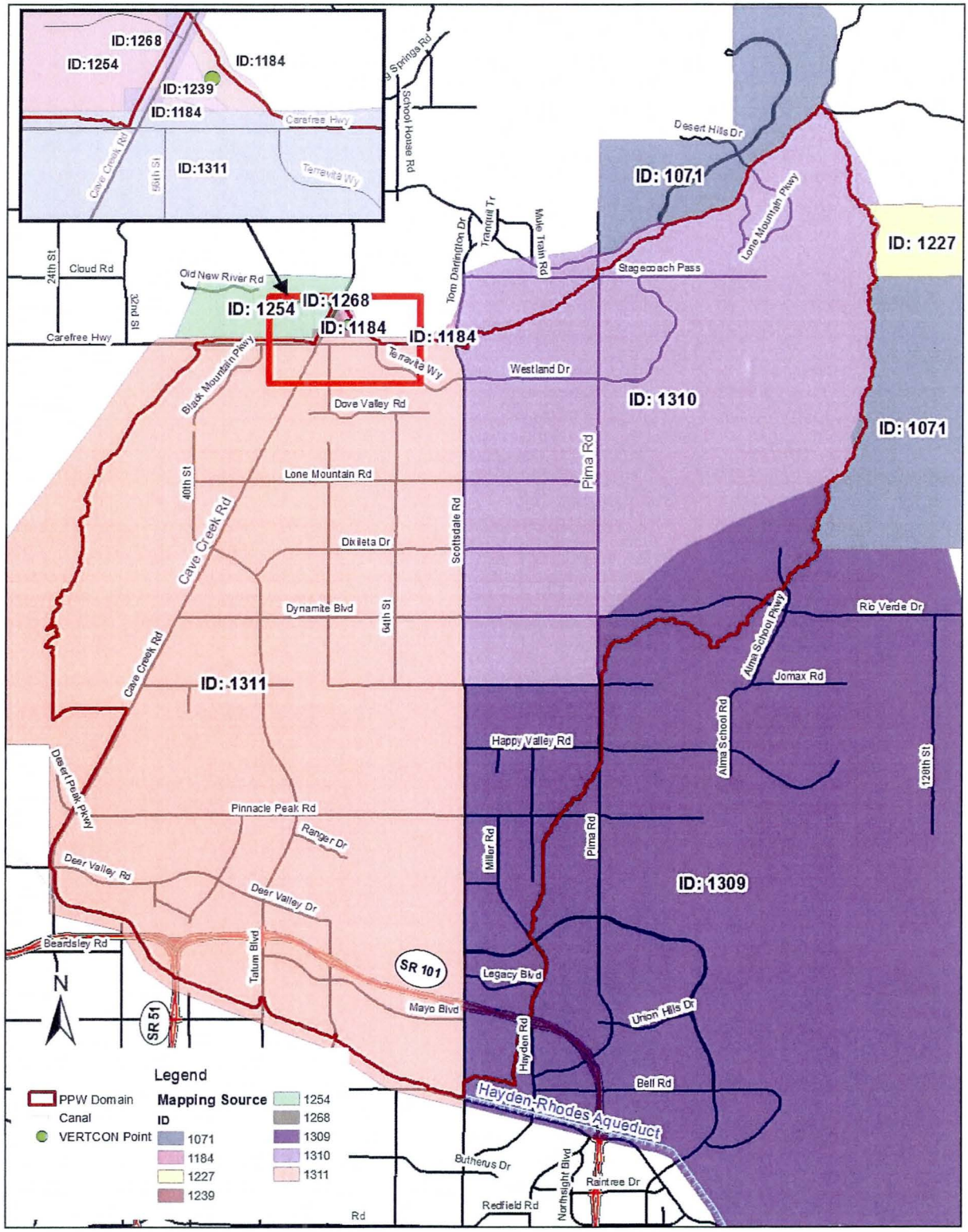

Figure 3. PPW ADMS Mapping Data Sets

4. HYDROLOGY

4.1. Method Description

The hydrology and hydraulics for the PPW ADMS project have been modeled using the two-dimensional software package FLO-2D. The analysis was completed using District guidance and recommendations for model parameter estimation and development as well as two-dimensional modeling techniques appropriate for the area. Where applicable, JEF made changes to base input parameters such as Manning's n-values and infiltration parameters (initial abstraction, percent impervious, etc.) in areas where the watershed warranted specific unique values different from the District's default recommended values. FLO-2D PRO Build No. 13.07.05, dated 9-10-2013 was used for the analysis. The input files for the models were developed using ArcGIS version 10.1 software.

4.2. FLO-2D Model Development and Parameter Estimation

Preliminary methodologies and approaches to the hydrologic and hydraulic modeling for the PPW ADMS watershed was investigated and summarized in Work Assignment 1 (WA#1) of the FCD 2001C024 contract. The summary report documenting the decisions and findings from that task (Work Assignment #1 Task 5 Hydrology and Hydraulics Report) is included in the appendix. The purpose of the report was to develop many of the base assumptions and methodologies values for the FLO-2D modeling discussed in the following sections, such as selection of grid cell size, duration and frequencies for analysis, topographic interpolation method etc.

The ADMS study area was divided into multiple sub-areas due to the size of the watershed, totaling 97.4 square miles, with a grid size of 20 feet. The sub-areas are shown on Figure 2 and the sub-area names, sizes, and naming nomenclature are listed in Table 4 in Section 1.4. The following sections describe the development of the FLO-2D parameters and input files. Hydraulic related parameters are presented in Section 5 as noted in the listing below. The models are developed to simulate rainfall-runoff over the 2D grid and utilize the following components of FLO-2D:

- Rainfall – RAIN.DAT
- Infiltration – INFIL.DAT
- Area Reduction Factors – ARF.DAT (Section 5.3.4)
- Hydraulic Structures (Culverts, Storm Drains & Wall Openings) – HYSTRUC.DAT (Section 5.5.2)
- Property Walls – LEVEE.DAT (Section 5.5.3)
- One-Dimensional Channels – CHAN.DAT (Section 5.5.4)

The following input files are included with the FLO-2D models but are not components to be switched on in the FLO-2D control file CONT.DAT:

- Inflows – INFLOW.DAT
- Floodplain Cross-Sections – FPXSEC.DAT
- Outflows – OUTFLOW.DAT

Three rainfall events were modeled for the study including the 10-year 24-hour, 25-year 24-hour, and 100-year 24-hour events; the 10- and 25-year events are only used for risk assessment purposes. A report documenting the risk assessment is submitted under a separate cover, *Work Assignment #3 Task 12-15 Hazards*

Prioritization and Alternatives and is not included with this TSDN. The 10- and 25-year FLO-2D models and select digital output (depth, velocity, and discharge raster datasets) is located with the digital data (USB drive).

The PPW ADMS watershed receives off-site flow into the ADMS study area from two sources. In the north, inflows are from the Unnamed Central Tributary to Cave Creek HEC-1 model (Carefree DMP, CH2M Hill, 2003). In the southeast, the inflows are from the Pinnacle Peak South (PPS) ADMS FLO-2D models (TYLin, 2012). The inflows are discussed in more detail in Section 4.3.

Due to the large number of property walls in the watershed and their impact on flow patterns, the PPW ADMS is modeled with three scenarios related to the property walls: without property walls, with property walls and no failure (walls overtop when the runoff ponds to the top of the wall), and with property walls allowing for failure (walls fail when flow ponds to a specified depth against the wall). The modeling of the walls and the development of the LEVEE.DAT file used to model them is discussed in more detail in Section 5.5.3.

4.2.1. Grid Element Size and Elevations (CADPTS.DAT and FPLAIN.DAT elevations)

The FLO-2D surface is represented as a grid comprised of square elements that route the flood wave over the watershed's topographic surface. The grid element size selected for the PPW ADMS project measured 20 feet by 20 feet. The total PPW watershed is 97.4 square miles. With a grid size of 20 feet, the PPW ADMS watershed was split into multiple sub-areas to keep the number of grid elements per model around one million to provide for more manageable sub-area model run-times. The number of grid elements for each sub-area is listed in Table 7 and the sub-area domains are shown in Figure 2.

Table 7. FLO-2D Sub-Area Grid Elements.

Sub-Area ID	Area (mi^2)	Number of Grid Elements
Area LT	12.0	837,058
Area UR	13.3	927,138
Area WR	15.1	1,057,569
Area TR	15.7	1,098,242
Area LR	15.7	1,098,474
Area CB	9.9	694,837
Area DR	15.7	1,097,612
Area R-11*	2.6	186,381

*This area overlaps with Area DR and Area LR and is developed to model the ponding with the Reach-11 Dikes. It is not an additional area.

The internal watershed boundaries were delineated using preliminary modeling results and existing flow patterns as well as topographical ridgelines. The individual sub-areas interact with each other where the outflows from the upstream model(s) become inflows to the downstream model(s). The outflow nodes of the upstream model(s) are in the same location as the inflow grid elements in the downstream model(s). The internal boundaries of the sub-areas are delineated to overlap by exactly one grid cell to take advantage of the outflow-to-inflow functionality of the FLO-2D software. A more detailed discussion of the outflow-to-inflow routine and development is discussed in Section 4.2.7.

The elevation data for the FLO-2D grid was developed starting with a TDS generated from the aerial mapping data (photogrammetry) mass-point and break-line data supplied by the District (See Section 3.3). The TDS was converted to a 20-foot pixel raster using built-in ArcGIS v10.1 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 each of the FLO-2D model sub-area domains.

In general, the upper watershed, primarily sub-areas LT, UR, and WR, is characterized by relatively incised channels with rocky terrain. The vegetation is typical of the upper Sonoran desert scrub consisting of a mixture of cacti, yucca, small palo verde, and mesquite trees and a variety of brush (Figure 4). The drainage is generally from northeast to southwest.

The lower watershed, sub-areas LR, TR, and DR, is characterized by more flat, shallow, braided channels with sparse vegetation. The vegetation is generally creosote and other small shrubs with denser vegetation along the major wash corridors, see Figure 5. Sub-area CB, located west of Cave Creek Road, is vegetated similarly to the lower watershed but the drainage corridors are more incised similar to the upper watershed. The downstream limits of the study encompass the Reach-11 Dikes which pond water away from and to protect the CAP canal embankment. Area R-11 was developed to model the deep ponding in this area.

Figure 4. Typical Vegetation of the Upper Watershed

Figure 5. Typical Vegetation of the Lower Watershed

4.2.2. Watershed Workmaps

The following watershed workmaps have been prepared and are located in the Exhibit Maps section at the end of this document:

- General Watershed Map (Topographic Contours and Spatially Varied Elevations)
- Land Use (Surface Characterization)
- Soils
- Hydraulic Conductivity (XKSAT)
- Wetting Front Capillary Suction (PSIF)
- Percent Impervious (RTIMP)
- Initial Abstraction (IA)
- Volumetric Soil Moisture Deficit (DTHETA)
- Limiting Infiltration Depths

4.2.3. Gage Data

There are seven gages (stream and rain gages are separate gages at Stagecoach Wash and Rawhide Wash) located within the study area (Table 8); these gages are maintained by the District. There are no USGS gages within the study area. The locations of the gages, and those surrounding the watershed, are shown in Figure 6.

Table 8. Gages located within the PPW ADMS Study Area

Gage ID	Gage Name	Gage Type	Installation Date
4918	Cave Cr. nr Cave Cr.	Stream	5/27/1994
4915	Cave Creek Landfill	Rain	4/22/1993
4910/4913	Stagecoach Wash	Rain/Stream	6/13/2001
4860/4863	Rawhide Wash	Rain/Stream	7/22/1999
4670	Pima @ Jomax	Weather	5/6/1993

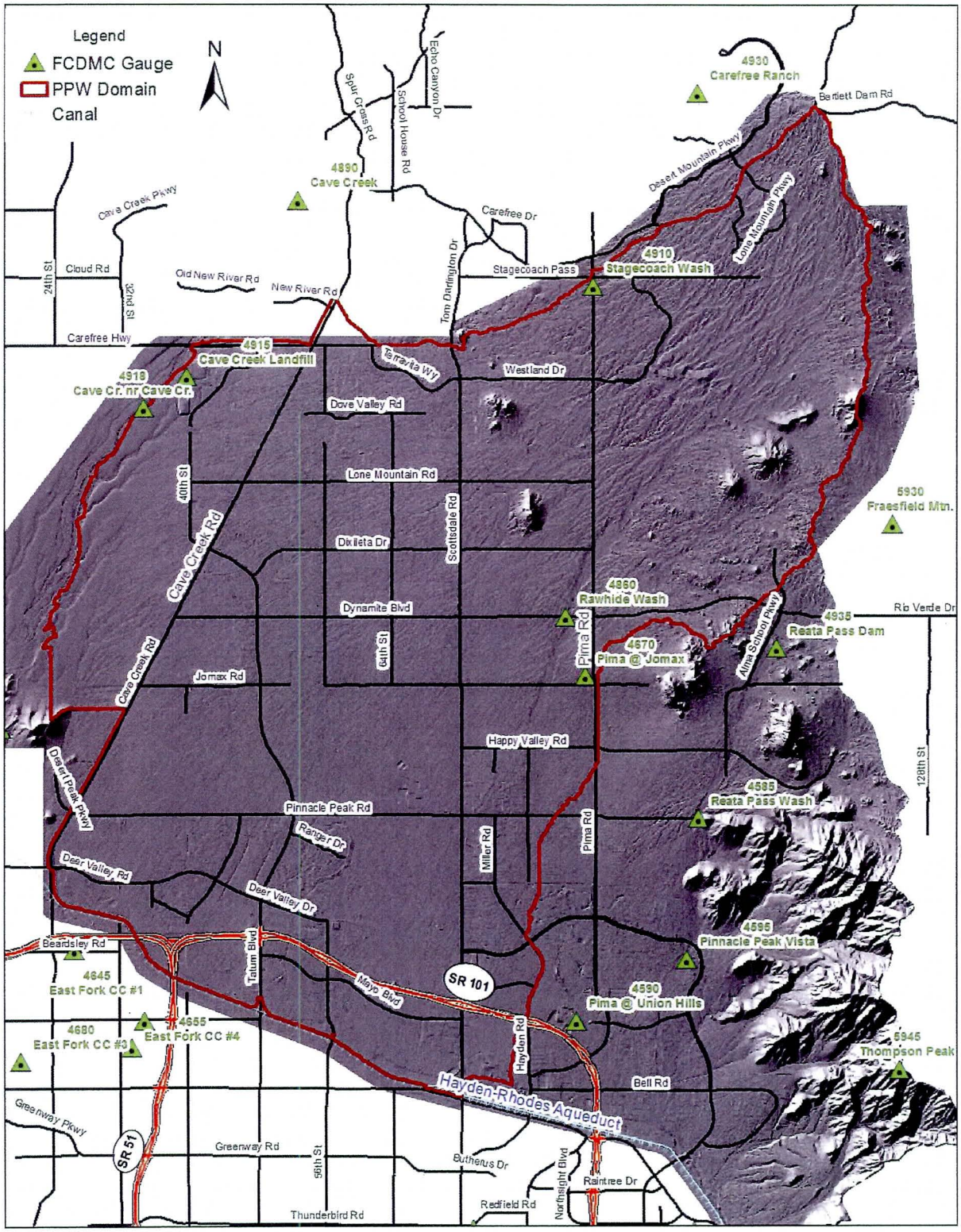

Figure 6. PPW ADMS Gage Locations

4.2.4. Grid Element Elevations Adjustments

In general, the conversion from the TDS to a 20-foot pixel raster produced results that closely represented the existing topographic information (See Section 3). However, the elevations of certain locations, such as culvert inlets/outlets and locations where the mapping data did not accurately reflect the existing ground (due to recent development that occurred after the mapping data was collected) were manually adjusted to more accurately represent the true drainage characteristics. The adjustments were tracked spatially with a GIS polygon shapefile. This shapefile tracked numerous changes throughout the modeling effort including Area Reduction Factor (ARF) changes (See Section 5.3.4 for a discussion on ARF values), n-value changes for model stability reasons, and elevation changes for model stability issues, topographic changes in the watershed, or hydraulic structure modeling (See Section 5.5.2.4). This shapefile is provided on the digital data disk enclosed within this notebook. There are several primary areas where the mapping data did not reflect the true existing drainage conditions due to changes since the mapping data was generated. These locations are Rawhide Wash, downstream of Pinnacle Peak Road to Scottsdale Road (Silverstone Channel), Legacy Boulevard, Sonoran Boulevard, and the Mayo Boulevard Channel which is located south of Mayo Blvd and east of the Mayo Hospital/medical center. These are discussed in more detail in the following sections.

4.2.4.1. Rawhide Wash

The 2007 mapping data used to develop the initial FLO-2D elevation data was outdated and did not reflect changes made along Rawhide Wash from Pinnacle Peak Road to Scottsdale Road. The changes included revisions to Pinnacle Peak Road (road widening) and a bridge at Pinnacle Peak Road where it crosses Rawhide Wash replacing the original low water crossing. Downstream of the Pinnacle Peak Road Bridge an existing interim channel was graded to convey flow towards Scottsdale Road. Currently, Scottsdale Road is under construction and a bridge is built where it crosses Rawhide Wash, replacing the original low water crossing. The Pinnacle Peak Road Bridge, existing interim channel, and the Scottsdale Road bridge construction are shown in Figure 7, Figure 8, and Figure 9 respectively.

JEF was informed by the City of Scottsdale that an "Ultimate Channel" design had been approved by the City that will connect the Pinnacle Peak Road Bridge to the bridge being constructed at Scottsdale Road. The "Ultimate Channel" construction is underway to replace the interim channel downstream of Pinnacle Peak Road. The FLO-2D FPLAIN.DAT elevation revisions reflect the design of the "Ultimate Channel" and not the interim channel. JEF obtained the "Ultimate Channel" design contours, the Scottsdale Road bridge construction contours and the surrounding existing ground data (Dated 2007) from Pinnacle Peak Road south to Williams Drive and from Scottsdale Road east to Miller Road. Figure 10 shows the proposed channel contours from Pinnacle Peak Road to Scottsdale Road overlaid on the contour data that shows the existing interim channel downstream of Pinnacle Peak Road. All contour data received was in 1-foot contour interval accuracy. In addition to the contour data, JEF obtained design plans of the Pinnacle Peak Road widening in pdf format. A small interim construction channel has been constructed to outlet Rawhide Wash onto Arizona State Land west of Scottsdale Road downstream of the bridge construction, this channel is shown in Figure 11 and the contours, shown in Figure 10, were included with the data collected. Although this channel west of Scottsdale road is considered to be in an interim condition, the final channel design into Arizona State Land is not expected to be built in the near future and the exact design of the channel is unknown; therefore, the interim channel was modeled for this study.

The elevations in the FLO-2D FPLAIN.DAT file were revised to reflect the changes listed above. Due to the large area covered by the required elevation revisions, the elevations were not manually revised. Instead, a terrain dataset was built within ArcGIS using the contour data. The terrain dataset was converted to a raster

with 20-foot pixel resolution using the same methodology as when the initial FLO-2D grid elevation data was developed. The resulting raster grid value became the revised grid elevation. Initial review of the terrain dataset developed from the 1-foot contour interval data indicated that the Pinnacle Peak Road widening was not reflected in the contour data. The PPS elevation data was compared to the elevation data shown in the pdf design plans of the road widening and the elevations were close enough that the PPS elevation data was used along Pinnacle Peak Road from Scottsdale Road to Miller Road overriding the elevations computed from the terrain dataset to raster function; this was only done for Pinnacle Peak Road

Figure 7. Rawhide Wash Looking Upstream at Pinnacle Peak Road Bridge – Photograph Dated June 2013

Figure 8. Rawhide Wash Existing Interim Channel Looking Downstream From Pinnacle Peak Road Bridge – Photograph Dated June 2013

Figure 9. Rawhide Wash Looking Upstream at the Scottsdale Road Bridge Construction – Photograph Dated June 2013

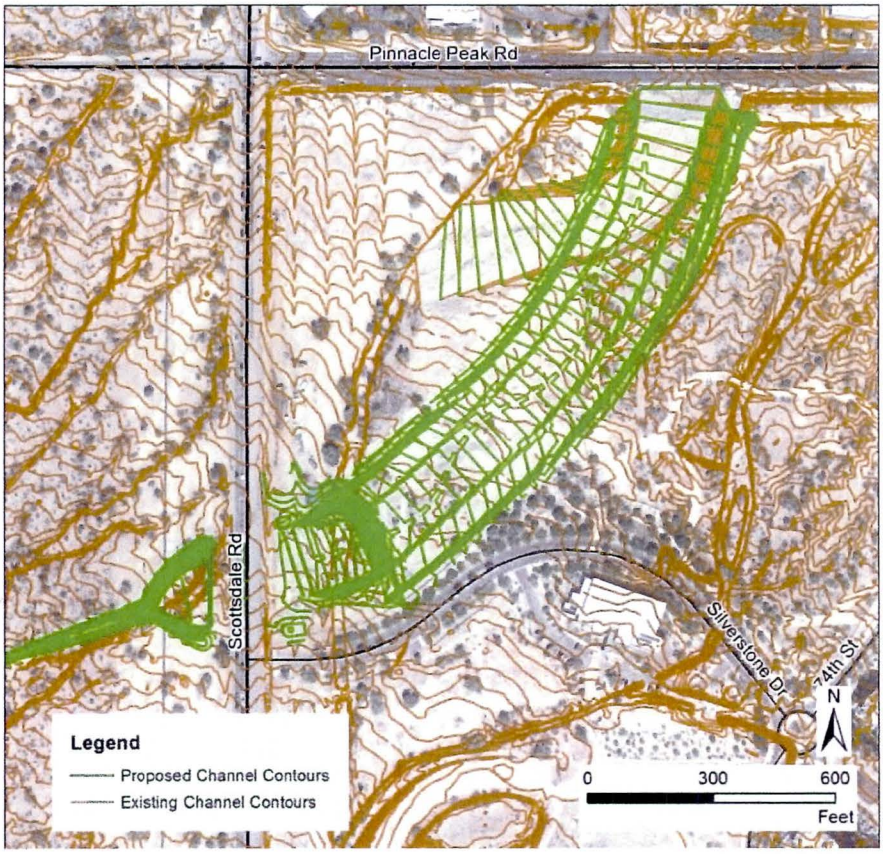

Figure 10. Rawhide Wash Proposed Channel Contours Between Pinnacle Peak Road and Scottsdale Road – Note the Existing Contours Show the Existing Interim Channel Downstream of the Pinnacle Peak Road Bridge.

Figure 11. Rawhide Wash Interim Channel downstream of Scottsdale Road – Photograph Dated February 2013

4.2.4.2. Legacy Boulevard

Similar to the Rawhide Wash revisions discussed in Section 4.2.4.1, Legacy Boulevard, located south of Thompson Peak Parkway and north of the Loop 101 and extending from Scottsdale Road east to Hayden Road, was constructed after the 2007 mapping data was developed. JEF obtained 1-foot contour interval data for the roadway design and this data was used to develop a terrain dataset in ArcGIS. The contour data however, did not cover the entire roadway and the portion of Legacy Boulevard from Scottsdale Road to approximately 1,500 feet east was not included in the digital contour data or the terrain dataset. The grids covered by the terrain dataset were revised using the same methodology as the initial FLO-2D grid elevation sampling and the Rawhide Wash revisions where the terrain dataset was converted to a 20-foot pixel raster using ArcGIS and the raster value became the revised grid elevation. JEF obtained as-built pdf documents for the portion of Legacy Boulevard that is not covered by the terrain dataset. The PPS elevation data was sampled in this area and is close to the same elevations in the as-built pdf document, therefore, the PPS elevations were used in the area not covered by the digital data.

4.2.4.3. Sonoran Boulevard

Sonoran Boulevard, located west of Cave Creek Road and approximately midway between Dynamite Boulevard and Jomax Road, was incorporated into the model(s) through grid element elevation revisions. The roadway was constructed after the 2007 mapping data was completed. Digital topographic data of the road was collected from the City of Phoenix. A terrain dataset was generated using the digital data and the terrain dataset was converted to a raster within GIS. This is the same procedure used for the initial FLO-2D grid element elevation. The digital data collected from the City of Phoenix was on the NGVD 29 vertical datum and had to be adjusted to convert the elevations to NAVD 88 datum. An adjustment of the elevation data was performed using the National Geodetic Survey (NGS) VERTCON tool. A point was selected at approximately the central location of the roadway alignment relative to its location within the Area CB model, see Figure 12. The VERTCON point adjustment factor was 1.903 feet and this factor was added to the raster generated from the terrain dataset to raster conversion. The adjusted raster became the base elevation data for the topographic revisions.

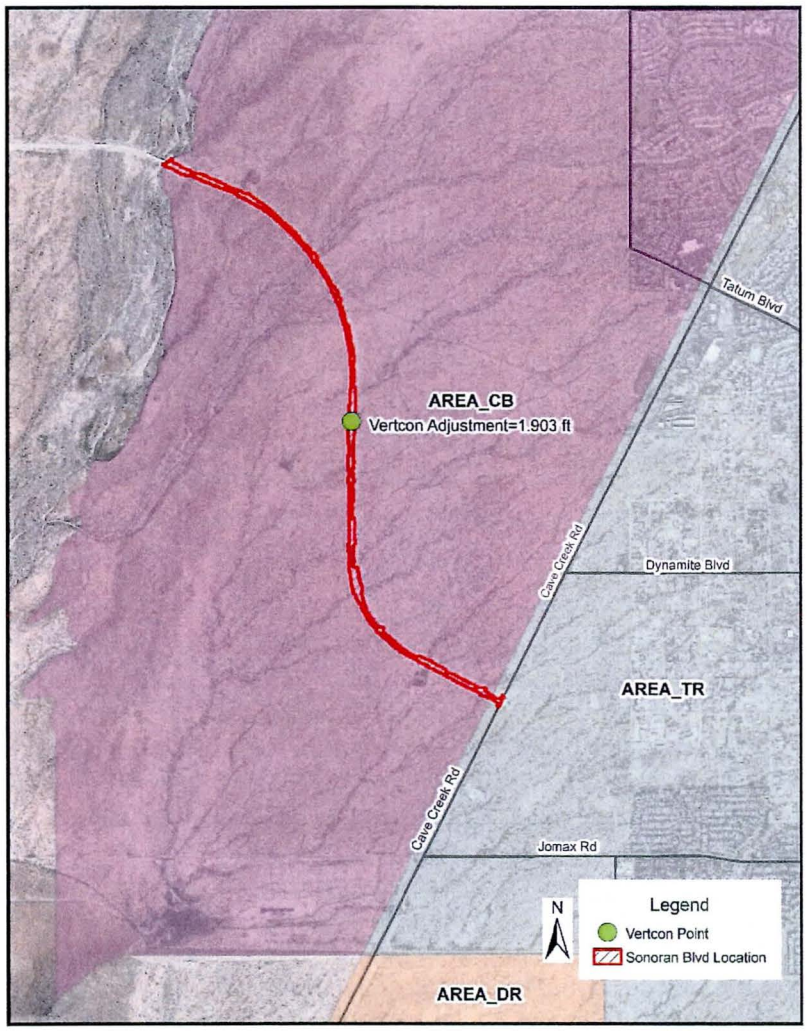

Figure 12. Sonoran Boulevard Vertcon Adjustment Location

4.2.4.4. Mayo Boulevard Channel

The Mayo Boulevard Channel is located just south of Mayo Boulevard and east of 56th Street. The channel, which is not reflected in the 2007 mapping data, conveys flow to the east from structure CID S_PPW_LR_1 to CID S049-06A/B. The flow patterns in the 2007 mapping data conveys the flow south through a series of small incised channels. Due to recent and on-going construction of the Mayo medical center (on-going at the time of this report) the incised channels have been filled and the Mayo Channel constructed. The channel has been included in the Area LR model by the lowering of grid elevations to simulate the channel. A typical cross-section and the typical longitudinal channel slope was surveyed to determine the geometry of the channel; the survey was completed by JEF in December, 2013. The surveyed channel typical section is 6-feet deep with a top width of roughly 80-feet and 4V:1H side slopes; the longitudinal slope is roughly 0.25% which is consistent with the existing ground slope.

4.2.4.5. Minor Adjustments

Locations that required small minor adjustments relative to the scale of the Rawhide Wash and Legacy Boulevard revisions are also made throughout the model as needed. These locations are primarily at culvert structure locations to stabilize the model hydraulic structure computations. The grid element 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 outlet. The method used to determine the culvert invert elevations is discussed in Section 5.5.2.1. To prevent the model from inaccurately representing a structure tailwater condition due to this elevation disparity between adjacent grid elevations and structure outlet elevations, the downstream elevations were manually adjusted to create a positive slope outfall where needed. The adjustments were made on a case-by-case basis and primarily at structures that FLO-2D adjusted to stabilize the model. There are locations however, where the structure outlet is in a legitimate and real sump condition and these locations were not modified. A detailed discussion on the FLO-2D adjustments of the structure rating curves is provided in Section 5.5.2.4.

Other minor revisions include increasing the n-values in deep ponding areas to prevent model surging and to lower grids where culvert structures exist but were not surveyed and a rating was not applied. In this situation, the grids were lowered and Area Reduction Factors (ARF) values applied to mimic a structure opening. For example, if grids were being lowered for a 36-inch pipe, an ARF value of 0.85 was applied to represent the 3-foot opening relative to a 20-foot grid. The revisions were documented in a GIS shapefile to summarize the change(s) and the reason for the change(s). Further discussion on grid element ARF values is provided in Section 5.3.4.

4.2.5. Rainfall/Precipitation Data (RAIN.DAT)

4.2.5.1. Rainfall Depths

Rainfall precipitation depths for the PPW ADMS project were obtained through the Drainage Design Software for Windows (DDMSW) Version 4.6.0 NOAA Atlas 14 data for the 10-, 25-, and 100-year 24-hour events. The rainfall precipitation depths vary across the watershed from roughly 5.70 inches in the northern mountainous areas near the Town of Carefree to 3.70 inches at the downstream limits of the watershed near the CAP Canal for the 100-year, 24-hour rainfall depths (Figure 13), from roughly 4.25 inches to 2.90 inches for the 25-year, 24-hour rainfall depths (Figure 14), and from roughly 3.50 inches to 2.40 inches for the 10-year, 24-hour rainfall depths (Figure 15). As a result of this variation in rainfall data, spatially varied rainfall was modeled for the entire study area using the actual NOAA Atlas 14 rainfall statistics at each grid. This was

accomplished by selecting the maximum point-precipitation depth for each sub-area and assigning a reduction factor (RAINARF) to the remaining grid elements in each sub-area based on a percentage of the maximum. Rainfall depths and the associated reduction factors are provided in the FLO-2D RAIN.DAT input file for each sub-area. The selection of the maximum point-precipitation depth and subsequent rainfall reduction factor depending on that maximum value was conducted for each PPW FLO-2D sub-area model domain for the 10-, 25-, and 100-year, 24-hour event. The maximum point rainfall depths for each sub-area are listed in Table 9. This reduction should not be confused with traditional Rainfall Aerial Reduction. Rainfall aerial reduction was not used for this study to ensure conservative flood hazard model results.

Table 9. PPW FLO-2D Sub-Area Maximum Point Rainfall Depths

PPW Sub-Area	100-Year, 24-Hour Maximum Rainfall Depth (in)	25-Year, 24-Hour Maximum Rainfall Depth (in)	10-Year, 24-Hour Maximum Rainfall Depth (in)
Area LT	5.695	4.315	3.503
Area UR	5.445	4.133	3.359
Area WR	4.993	3.763	3.047
Area TR	4.517	3.447	2.812
Area LR	4.511	3.472	2.844
Area CB	4.366	3.341	2.731
Area DR	4.083	3.156	2.593
Area R-11	3.844	2.988	2.464

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 actual point rainfall depth on those FLO-2D grids that intersect multiple DDMSW rainfall depth grids.

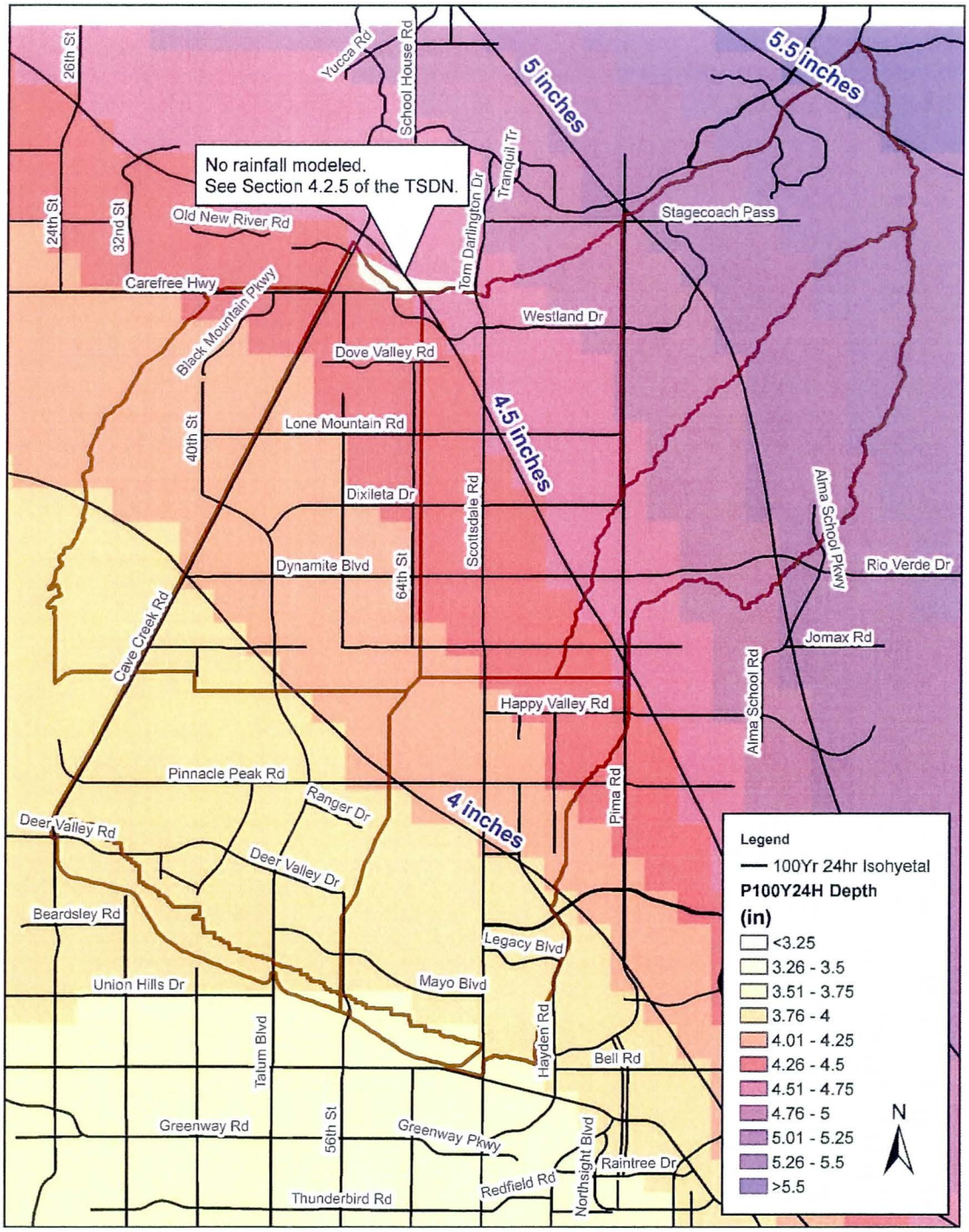

Figure 13. 100-year 24-hour Rainfall Depths

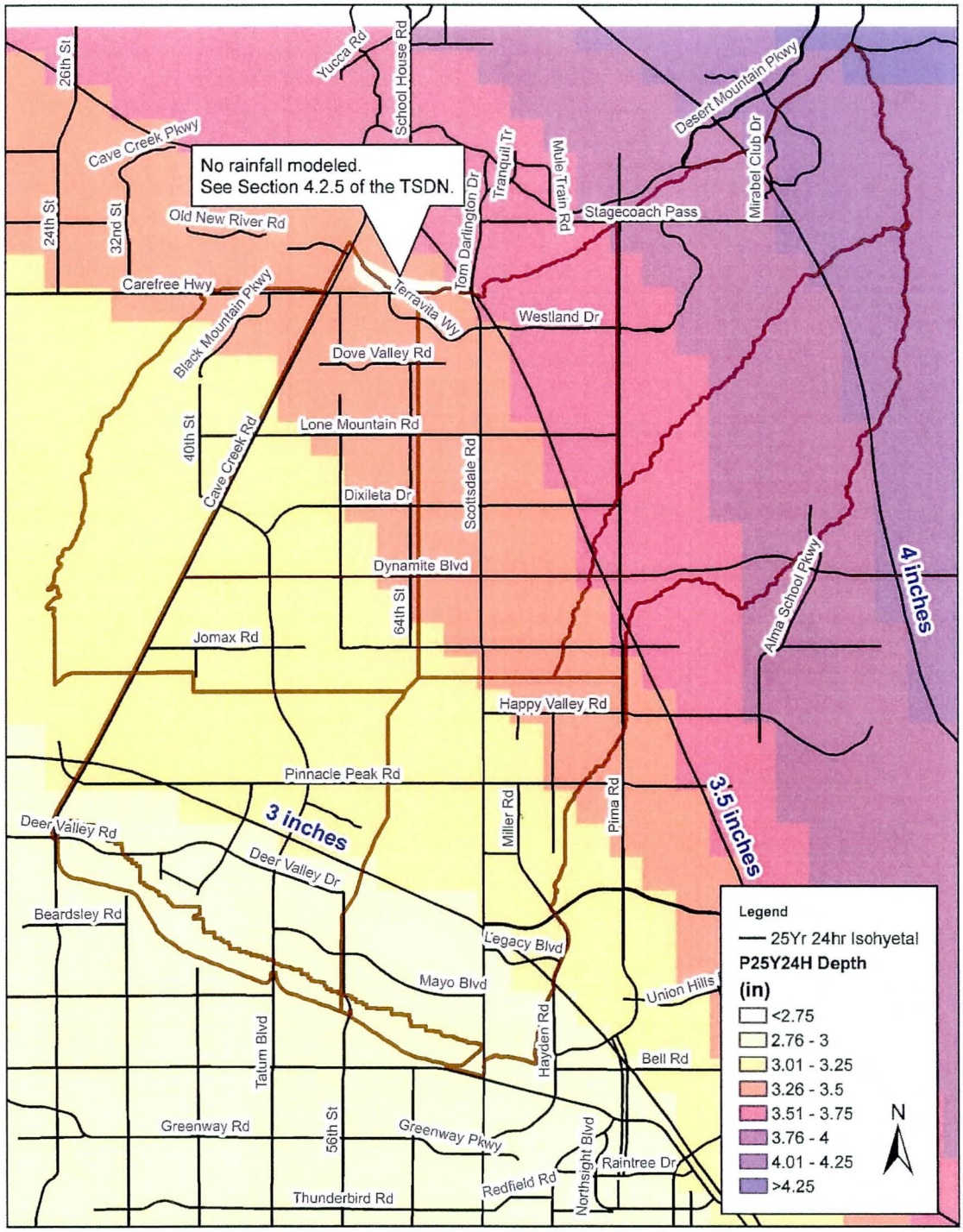

Figure 14. 25-year 24-hour Rainfall Depths

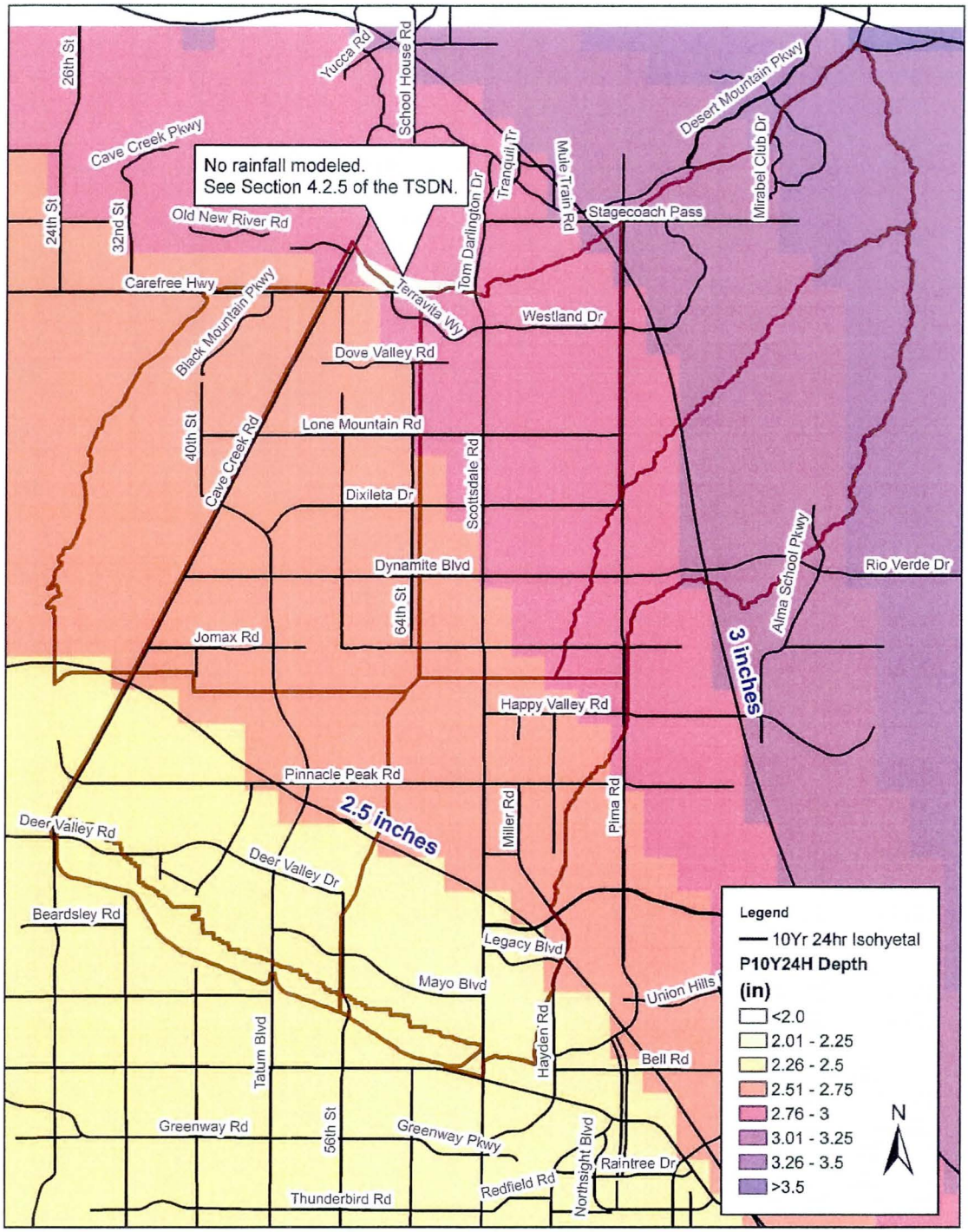

Figure 15. 10-year 24-hour Rainfall Depths

As discussed in Section 4.2, the PPW watershed has inflows at the northern boundary from the Unnamed Central Tributary to Cave Creek HEC-1 Model (Carefree DMP) north of the Carefree Highway. The HEC-1 model sub-basins include some of the PPW FLO-2D model domain in the northern portions of model sub-areas TR and WR. For the area of the FLO-2D domain included in the HEC-1 model sub-basins a rainfall depth of zero inches was assigned by use of a 0.0 reduction factor for the overlapping grid cells. This was done to prevent "double-counting" of rainfall over this area since it is already included in the HEC-1 model. FLO-2D grid elements that intersect the boundaries between the HEC-1 domain and the FLO-2D domain receiving rainfall are area-weighted. As a result, rainfall depths around the perimeter of the HEC-1 domain range from 0 inches to the full rainfall event depths. The intersecting area between the two models with a rainfall depth of zero inches is shown on the rainfall exhibit maps in Figure 13, Figure 14, and Figure 15. A detailed discussion of the inflows from the HEC-1 model is discussed in Section 4.3.

The IRAINBUILDING feature of FLO-2D was utilized for this project to capture rainfall runoff from buildings coded with Area Reduction Factors (ARF). Use of the IRAINBUILDING option allows rainfall to run off grids with ARF values onto the adjacent grid elements. See Section 5.3.4 for a more detailed discussion on model ARF values.

4.2.5.2. Temporal Distribution

The SCS Type II 24-hour rainfall distribution was used for all RAIN.DAT files and is shown in Figure 16.

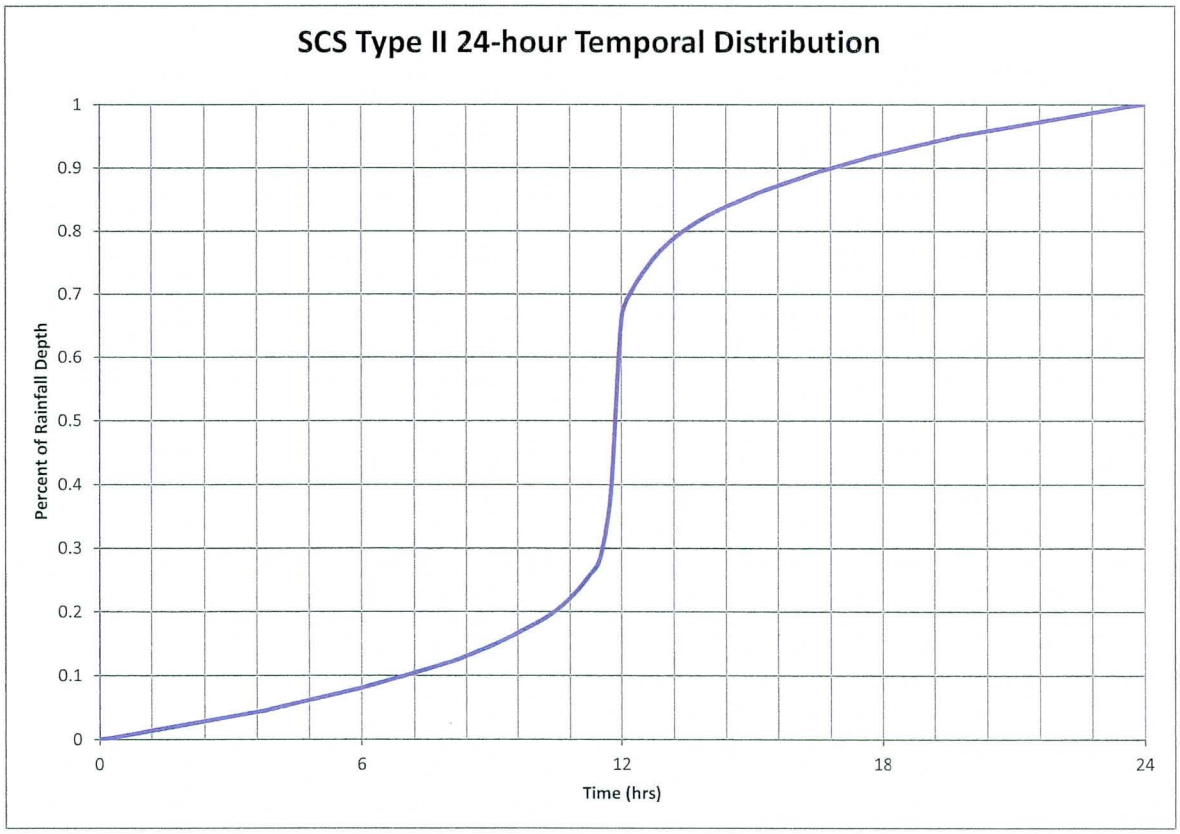

Figure 16. SCS Type II 24-Hour Temporal Distribution

4.2.6. Infiltration (INFIL.DAT)

The Green and Ampt infiltration method was used for the PPW ADMS. The required parameters for infiltration are hydraulic conductivity (XKSAT), capillary suction (PSIF), initial moisture deficit (DTHETA), initial abstraction (IA), Percent Impervious (RTIMP), and a limiting depth of infiltration. The limiting depth function stops infiltration of runoff into the soil once it reaches a specified depth. The development of each of the aforementioned parameters is described in the following sub-sections, exhibits showing the infiltration parameters over the watershed are located in the Appendix.

4.2.6.1. Soils Data (XKSAT, PSIF, DTHETA, Limiting Infiltration Depth)

The soil data used for the majority 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, and is covered by the Tonto National Forest Terrestrial Ecosystem Survey (TES) soil data. However, the TES has not been finalized at the time of this study. 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 17. The detailed NRCS soils data is 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). Land use is discussed in the following section. 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 and is 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 method of assigning the soil parameters for each grid element 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 the DDMSW relationship relating the PSIF and DTHETA values to a given XKSAT value. The DTHETA initial condition across the watershed is considered to be normal except for areas of water which were considered to be saturated and the resulting DTHETA value is 0.0. The DTHETA type is selected by the location of the FLO-2D 20-foot grid centroid. If the centroid falls on the saturated land use type then the DTHETA value for that grid is saturated, likewise for normal. No area-weighting was conducted for the DTHETA value. The DTHETA initial 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 grid. The effective percent impervious assigned to each FLO-2D grid is also dependent on the coverage of percent impervious as it related to land use type. This is discussed in more detail in the Land Use discussion in the following section. The Rock Outcrop percentage listed in the soils data summary in Table 13 is only the percent impervious as it relates to the rock outcrop per the soil coverage;

it is independent of the land use impervious area coverage. As previously stated, the grid effective impervious percentage is a value dependent on the land use coverage and the soils rock outcrop coverage. This is discussed in more detail in Section 4.2.6.2.

Initial model runs were simulated using the FLO-2D limiting infiltration depth parameter and was varied by soil map units. The initial model limiting depths were based off of estimated limiting infiltration horizons from the soils data. For example, the depth to clay or bedrock based on the soil map unit description was used to determine the estimated limiting infiltration depth for FLO-2D. However, these initial results had relatively high percentages of runoff when compared to typical upper Sonoran desert watersheds and were in the range of 60% runoff. It was determined that this percentage of runoff was too high and a calibration of the limiting infiltration depth parameter was performed. The limiting depth now varies for each sub-area instead of by soil map units and is based upon a calibration of the total percentage of FLO-2D runoff with the total percent runoff of a HEC-1 model where the infiltration limiting depth variable was the calibrated parameter. The HEC-1 model development is discussed in more detail below. The seven primary FLO-2D sub-areas (Area-R11 was not included in the calibration due to being a model developed for the Reach 11 ponding) were simulated with rainfall only and did not account for any upstream runoff, i.e. rainfall and infiltration were modeled and an INFLOW.DAT file was not used. The infiltration parameters for the FLO-2D models are spatially varied and assigned according to the development of the INFIL.dat file as discussed in this TSDN (XKSAT, PSIF, DTHETA, IA, and RTIMP). The only calibration control parameter was the limiting infiltration depth. The Reach-11 model limiting depth values correspond to those of the LR and DR models. The grids of R-11 that are coincident with the DR domain use the DR limiting depth values and the grids of R-11 that are coincident with the LR domain use the LR limiting depth values. Calibration to the HEC-1 results was selected because no other rainfall-runoff data were available to calibrate the computed runoff volumes. There are very few gages in the watershed and very little real-event data to provide any other data for calibration. Gage data and actual event data were used for results verification and this topic is discussed in Section 4.11.

A single HEC-1 model was developed with seven sub-basins, one for each FLO-2D sub-area neglecting the R-11 sub-area. The basin area is that of the individual FLO-2D sub-area sizes and the rainfall and infiltration parameters were averaged based upon the FLO-2D input files. 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 10 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 computed HEC-1 runoff percentages are listed in Table 12. The HEC-1 model input and output are included in Appendix D.1.

Table 10. HEC-1 Model Parameters.

Subarea	100-Year Average Rainfall (in)	IA (in)	DTHETA	PSIF (in)	XKSAT (in/hr)	RTIMP (%)
LT	5.166	0.364	0.217	6.499	0.171	6.88
UR	5.007	0.369	0.230	5.980	0.182	8.53
WR	4.511	0.297	0.226	5.714	0.223	15.35
LR	3.981	0.286	0.250	4.204	0.354	17.58
TR	4.140	0.277	0.228	5.056	0.310	17.46
DR	3.868	0.289	0.249	4.309	0.337	14.67
CB	4.086	0.310	0.205	6.104	0.263	8.70

The percent runoff of each FLO-2D model was computed using the average depth of infiltration calculated from the FPINFILTRATION.OUT file. This output file gives the total depth of infiltration in feet for each grid in the model. This depth needed to be adjusted for the RTIMP of each grid and to add back in the depth lost due to the TOL value of 0.004 ft (0.048 in) and the initial abstraction. The equation used to compute the depth of infiltration for each grid in the model is below. Some grids, outflow nodes and grids that are totally blocked by area reduction values (ARF=1.0), have a depth of zero reported in the FPINFILTRATION.OUT file. These grids were ignored and not included in the average computation and the equation below was applied only to those grids greater than 0.0. The model average was computed by calculating the average of the grid total depth (D) for each sub-area.

- $D_i=((FPinfil_i*(1-RTIMP_i)+(IA_i/12))*12)+0.048$

  Where:

  D_i = depth of infiltration in inches for a specific grid cell,

  $FPinfil_i$ = the depth of infiltration in feet from the FPINFILTRATION.OUT file for a specific grid cell,

  $RTIMP_i$ = percent impervious for a specific grid cell,

  IA_i = Initial abstraction in inches for a specific grid cell.

The limiting depth of infiltration was varied in each sub-area based upon the wash bottom characterization in the Surface Feature Characterization of the watershed. Each sub-area had a single limiting depth value assigned to the wash bottom areas and a single value assigned to all other locations. Grids that intersect the wash bottom characterization and non-wash bottom were area-weighted averaged. The values for wash bottoms and non-wash bottoms for each sub-area are summarized in Table 11, the summary of results is located in Table 12.

Table 11. FLO-2D Limiting Depth Values per Sub-Area.

FLO-2D Sub-Area Limiting Infiltration Depths (ft)		
Area	Wash Bottom	Non Wash Bottom
LT	5	1.6
UR	4	0.95
WR	3	0.93
LR	3	0.75
TR	3	0.85
DR	3	0.72
CB	3	1

Table 12. FLO-2D Limiting Depth Calibration.

Subarea	No. Grids	Area (sq mi)	HEC-1 Loss (in)	HEC-1 Excess (in)	HEC-1 % Runoff	FLO-2D Loss (in)	FLO-2D % Runoff
LT	837058	12.010	2.90	2.26	**44%**	2.918	43.5%
UR	927138	13.303	2.82	2.19	**44%**	2.849	43.1%
WR	1057569	15.174	2.48	2.03	**45%**	2.521	44.1%
LR	1098474	15.761	2.32	1.66	**42%**	2.341	41.2%
TR	1098242	15.758	2.36	1.78	**43%**	2.372	42.7%
DR	1097612	15.749	2.34	1.53	**40%**	2.270	41.3%
CB	694837	9.970	2.54	1.54	**38%**	2.478	39.3%

Table 13. PPW Soils Data Parameters.

Map Unit ID (MUID)	Soil Unit Description	*Potential Limiting Horizon Based on MU Description	XKSAT (in/hr)	Rock Outcrop (%)	PSIF (in)	DTHETA (Normal)
6451	Antho sandy loams		0.41	0	3.92	0.25
6452	Antho gravelly sandy loams		0.41	0	3.92	0.25
6453	Antho-Carrizo-Maripo complex		0.58	0	3.4	0.27
6456	Anthony-Arizo complex		0.62	0	3.31	0.27
64512	Carefree cobbly clay loam, 1 to 8 percent slopes	clay at 1"	0.01	0	12.4	0.05
64518	Cheriono-Rock outcrop complex, 5 to 60 percent slopes	cemented material at 10", bedrock at 18"	0.33	15	4.35	0.25
64521	Cipriano very gravelly loam	cemented material at 6"	0.38	0	4.1	0.25
64526	Continental cobbly clay loam, 1 to 8 percent slopes	clay at 2"	0.01	0	12.4	0.05
64533	Eba very gravelly loam, 1 to 8 percent slopes	very gravelly clay at 3"	0.23	0	5	0.25
64534	Eba very gravelly loam, 8 to 20 percent slopes	very gravelly clay at 3"	0.23	0	5	0.25
64544	Ebon very gravelly loam, 1 to 8 percent slopes	very gravelly clay at 3"	0.03	0	10.1	0.13
64550	Estrella loams		0.26	0	4.7	0.25
64552	Gachado-Lomitas-Rock outcrop complex, 7 to 55 percent slopes	bedrock at 7"	0.16	20	5.8	0.25
64554	Gila fine sandy loams		0.29	0	4.55	0.25
64555	Gilman loams		0.27	0	4.65	0.25
64560	Glenbar loams		0.26	0	4.7	0.25
64561	Gran-Wickenburg complex, 1 to 10 percent slopes	bedrock at 12"	0.15	0	6	0.25
64563	Gran-Wickenburg-Rock outcrop complex, 1 to 7 percent slopes	bedrock at 12"	0.14	25	6.2	0.23
64572	Lehmans-Rock outcrop complex, 8 to 65 percent slopes	bedrock at 20"; clay at 2"	0.09	30	7.3	0.15
64575	Mohall loam	clay loam at 7"	0.23	0	5	0.25
64576	Mohall loam, calcareous solum	clay loam at 7"	0.23	0	5	0.25

Map Unit ID (MUID)	Soil Unit Description	*Potential Limiting Horizon Based on MU Description	XKSAT (in/hr)	Rock Outcrop (%)	PSIF (in)	DTHETA (Normal)
64577	Mohall clay loam	clay loam at 2"	0.05	0	8.8	0.15
64578	Mohall clay loam, calcareous solum	clay loam at 6"	0.05	0	8.8	0.15
64590	Momoli gravelly sandy loam, 1 to 5 percent slopes		0.39	0	4	0.25
64593	Nickel-Cave complex, 8 to 30 percent slopes		0.33	0	4.35	0.25
64596	Pinaleno-Tres Hermanos complex, 1 to 10 percent slopes		0.07	0	8	0.15
64598	Pinamt-Tremant complex, 1 to 10 percent slopes	Tremant component clay loam at 5"	0.37	0	4.15	0.25
645101	Rillito loam, 0 to 3 percent slopes		0.28	0	4.6	0.25
645103	Rock outcrop-Gachado complex, 5 to 55 percent slopes	bedrock at 7"	0.1	65	7	0.15
645110	Suncity-Cipriano complex, 1 to 7 percent slopes	cemented material at 6" (Cipriano) and 9" (Suncity)	0.13	0	6.4	0.21
645112	Tremant gravelly sandy loams	clay loam at 9"	0.39	0	4	0.25
645113	Tremant gravelly loams	gravelly clay loam at 9"	0.39	0	4	0.25
645118	Tremant-Rillito complex	Tremant component clay loam at 9"	0.42	0	3.88	0.25
645121	Tres Hermanos-Anthony complex, 1 to 5 percent slopes	clay loam at 2"	0.12	0	6.6	0.19
645122	Vado gravelly sandy loam, 1 to 5 percent slopes		0.33	0	4.35	0.25
645124	Valencia sandy loams		0.39	0	4	0.25
s316	Gran and Lehmans soils	Gran soils weathered bedrock at 12", Lehmans soils unwatered bedrock at 14"	0.02	0	11.2	0.1

*not used for Infiltration Limiting Depth

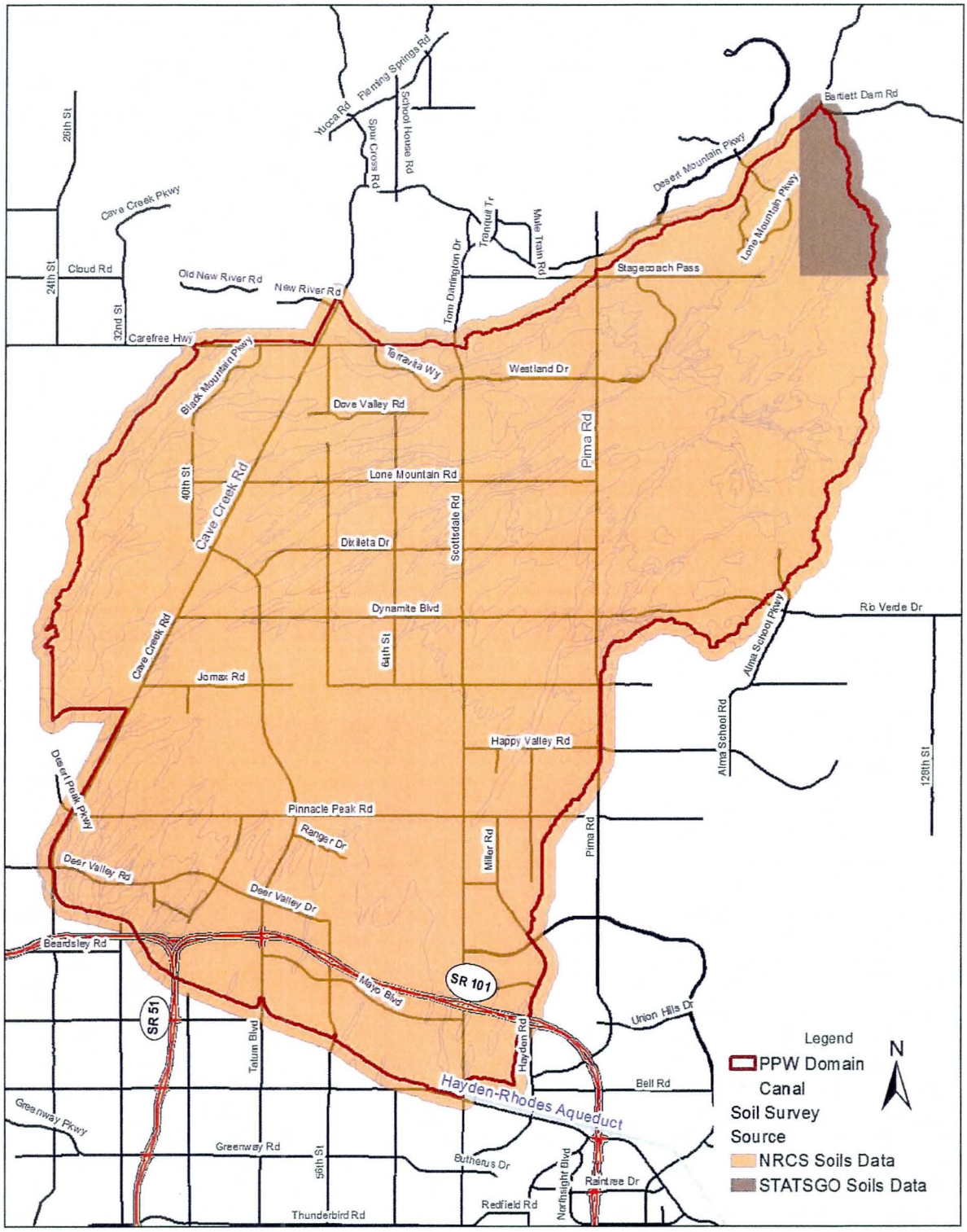

Figure 17. Soils Survey Coverage Over the PPW Watershed

4.2.6.2. Land Use (Surface Feature Characterization)

A comprehensive existing conditions land use (surface feature characterization) coverage was provided by the District based on planimetric features digitized with photogrammetry. 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.). The data was received by JEF in three separate datasets which JEF modified to merge into a single shapefile. JEF also digitized small areas not covered in the surface feature data received by the District. JEF used the same classifications for the additional digitized areas. The resulting shapefile was used to assign infiltration parameters, see Table 14, and Manning's roughness coefficients as discussed in Section 5.3.1. An exhibit showing the surface feature coverage over the PPW watershed is located in the Exhibit Maps section in the Appendix. Figure 18 shows the areas that were merged/digitized by JEF to stitch together multiple datasets to produce the single shapefile dataset. The values assigned to each FLO-2D grid are area-weighted.

The IA values listed in Table 14 are the full initial abstraction for the surface feature characterizations. The IA value in the INFIL.DAT files are the area-weighted IA values from Table 14 minus the surface detention value (TOL) from the TOLER.DAT file. The FLO-2D model control parameters, including TOL, are discussed in Section 4.5.

Table 14. PPW Surface Feature Characterization Parameters.

Classification	Description	IA (in)	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	Normal	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	Normal	0.055
Upper Hillslopes	Steep and rocky mountain slopes	0.10	0	Normal	0.060
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	Normal	0.030
Water	Lakes, canals, ponds	0.00	100	Saturated	0.020

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 grid was located on a soils type with a rock outcrop of 20% and was also on a grid with a land use percent impervious of 30%, then the RTIMP reported to the INFIL.DAT file for that specific grid is 50%.

The initial moisture condition is related to the DTHETA value, the watershed is considered to have a "normal" initial moisture condition except for the "water" surface feature characterization which is considered to be "saturated". If a FLO-2D grid is within the normal category then the DTHETA value assigned to that grid is based on the XKSAT value which is related to DTHETA through the DDMSW relationship of XKSAT and DTHETA. 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. The initial moisture condition is not area-weighted.

The dates of the surface feature characterization coverage is commensurate with the mapping data as discussed in Section 3.3 and is dated 11/2007 for the portion east of Scottsdale Road and 06/2010 west of Scottsdale Road. There have been, in some locations, newly constructed areas that have been built after the date of the surface feature characterization and, generally, these areas were not revised since it was considered that they do not have a large impact on the overall watershed characteristics. Revisions to the surface feature characterization data were made to include the development at the northwest and southeast intersection of Williams Drive and Miller Road. Other revisions made to adjust the elevation, area reduction factor, or Manning's n-values were made on a grid-by-grid basis and are not revised in the base surface feature characterization data. Major elevation revisions include the inclusion of Rawhide Wash/Silverstone Channel, Legacy Boulevard, Sonoran Boulevard, and the Mayo Boulevard Channel. These revisions are discussed in more detail in Section 4.2.4; the section also includes a discussion on the minor revisions throughout the watershed to improve model stability and culvert/structure hydraulics.

4.2.6.3. Existing FEMA Floodplains

There are existing FEMA floodplains located within the PPW study limits. The existing floodplains consist of riverine Zone AE with floodways as well as Zone AO and Zone A designations. There are several alluvial fan designations within the watershed, including the Rawhide Wash floodplain system. The current existing FEMA floodplains are shown on Figure 19.

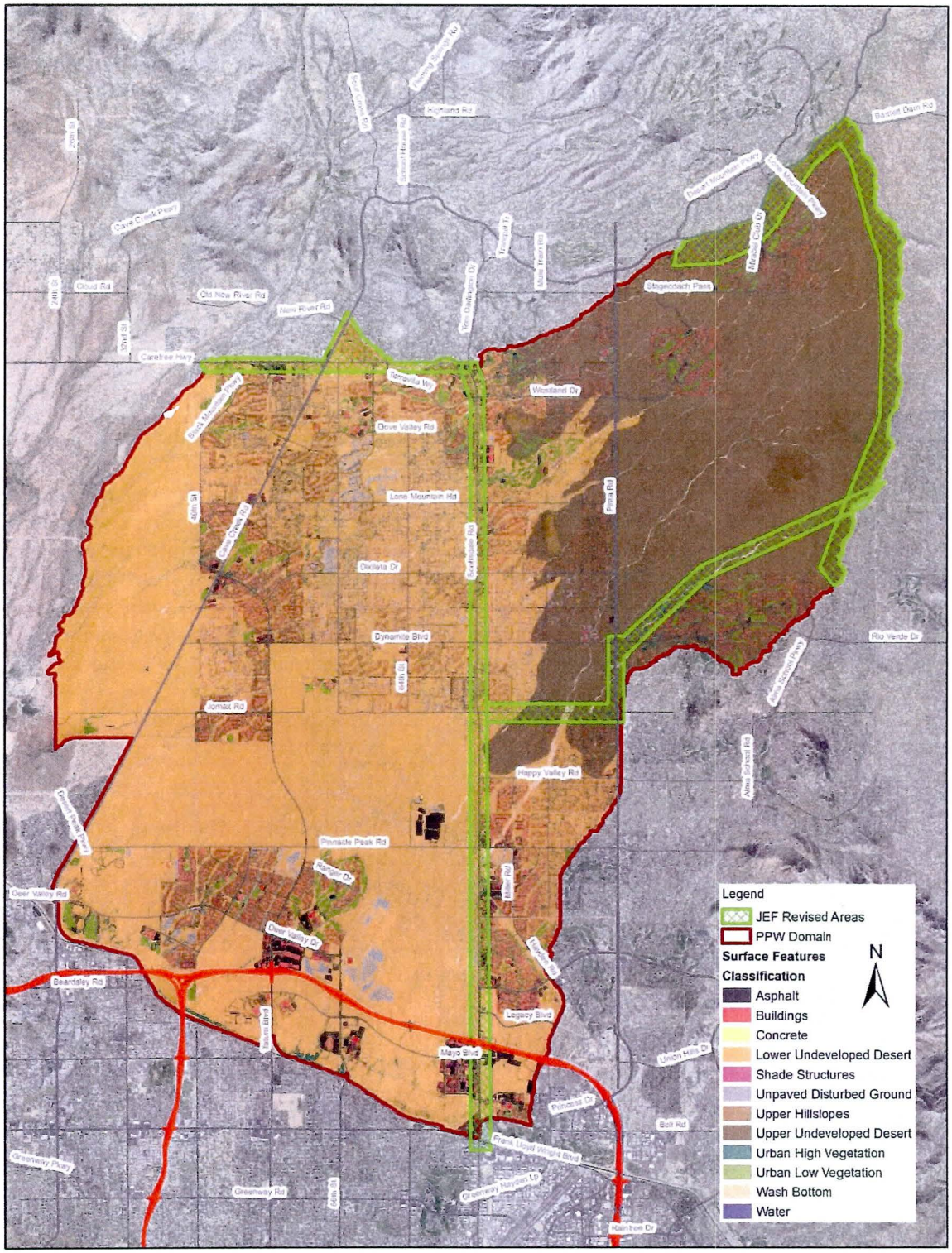

Figure 18. Surface Feature Dataset Areas Merged and Digitized to Stitch Datasets Together

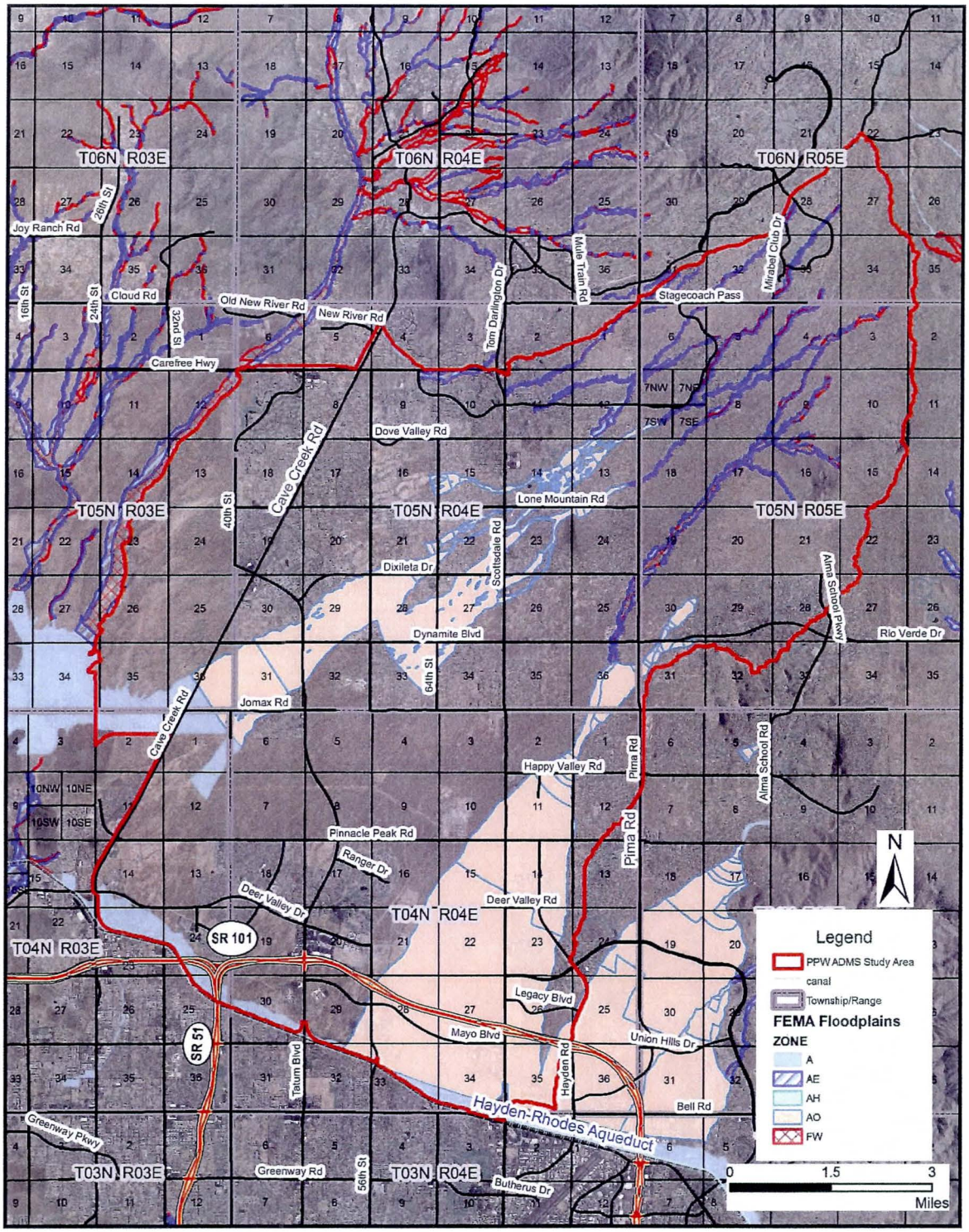

Figure 19. Existing FEMA Floodplains

4.2.7. Outflow nodes (OUTFLOW.DAT)

The FLO-2D OUTFLOW.DAT files are set up to make use of the Outflow-to-Inflow routine within FLO-2D. This routine takes the outflow hydrograph written to the OUTNQ.OUT file of an upstream model and writes it as an inflow hydrograph to the downstream grid. The routine requires that outflow nodes that are to be written to downstream inflows be designated with a unique numerical identifier to link the outflow node to the downstream CADPTS.DAT file.

The numerical identifier is a value of 1 to 9 and is placed after the "O" in the OUTFLOW.DAT file. Hence, the identifier becomes "O1" for example rather than simply "O". Outflow grids not written to an inflow file will remain identified with simply "O". The downstream CADPTS.DAT grid file is copied into the upstream model folder and is named with the same numerical identifier. The PPW upstream and downstream grids have exactly a one-cell overlap at the internal model boundaries so that the inflow nodes of the downstream grid are mapped exactly to the same X-Y coordinates of the outflow nodes in the upstream grid, see Figure 20.

As an example, Area-LT has outflow nodes that will be written as inflow nodes into Area-UR, see Figure 2. Therefore, the outflow nodes in the Area-LT OUTFLOW.DAT file that will become Area-UR inflows are identified with "O1". The CADPTS.DAT file from Area-UR is copied into the Area-LT model run folder and renamed as CADPTS_DS1.DAT, this is done so FLO-2D can map the outflow grid Cell ID to the appropriate grid Cell ID in Area-UR for the inflow location. The mapping is done based on the X-Y coordinates in the CADPTS.DAT files. The resulting inflow file is written as Inflow1_DS.DAT when Area-LT is writing the model output. This process is repeated for each boundary that has an outflow-to-inflow relationship as listed in Table 15. It should be understood that it is not an issue to have the same identifier used in different models as long as they are independent models. For example, the "O1" identifier in Area LT does not interfere with the "O1" identifier in Area LR. These models are independent of each other.

Table 15. PPW Outflow-to-Inflow Relationships

Upstream Grid	OUTFLOW.DAT Identifier	Downstream Grid
AREA LT	O1	AREA UR
AREA LT	O2	AREA WR
AREA UR	O3	AREA WR
AREA UR	O4	AREA LR
AREA WR	O5	AREA TR
AREA WR	O6	AREA LR
AREA TR	O7	AREA CB
AREA TR	O8	AREA DR
AREA LR	O9	AREA DR
AREA LR	O1	AREA R-11
AREA DR	O2	AREA R-11

In the event that an outflow does not generate an outflow hydrograph in the OUTNQ.OUT file (if there is little to no contributing area to that node for example), an inflow hydrograph is not mapped.

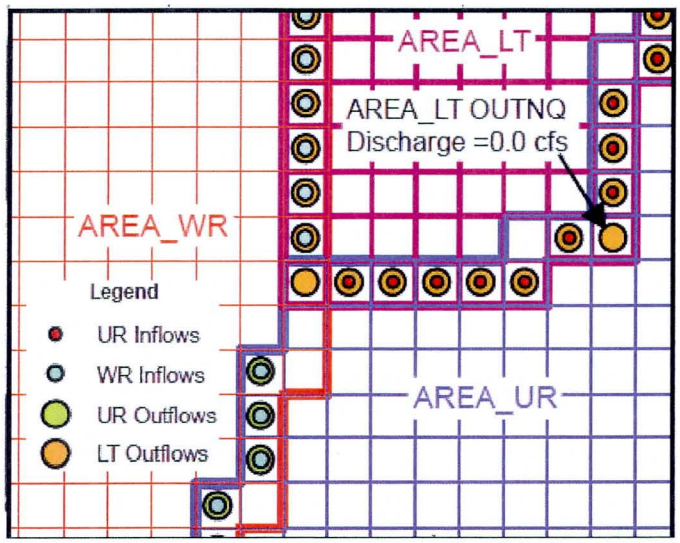

Figure 20. PPW Internal Boundary Inflow/Outflow

4.3. Off-Site Inflows

The PPW ADMS watershed receives inflows from two adjacent watersheds, the Carefree DMP from the north and the PPS ADMS from the east. The one-dimensional hydrologic model, HEC-1, was used for the Carefree DMP. HEC-1 model output was used for inflow hydrograph input into PPW ADMS FLO-2D sub-areas WR and TR. The two-dimensional FLO-2D model hydrologic and hydraulic model was used for the PPS ADMS. Outflow from the PPS ADMS model was used for inflow hydrograph input into PPW FLO-2D sub-areas UR and LR. The details of the generation of the off-site inflows from the two studies are detailed in the following sections.

4.3.1. Carefree DMP Inflow Hydrographs

Runoff from the Unnamed Central Tributary to Cave Creek (UCT) that flows from a northeasterly to southwesterly direction near Carefree Highway between Cave Creek Road and Scottsdale Road was added as inflow to the adjacent PPW FLO-2D models. The HEC-1 model runoff flows onto the FLO-2D grid along the UCT in the northern extents of the Area TR and WR models (Figure 21). The inflow hydrographs were derived from existing HEC-1 models developed as part of the Carefree DMP. The HEC-1 model for the Unnamed Central Tributary was modified for the purposes of the PPW ADMS. Two changes were made to the model – 1) rainfall depths on the JD records, and 2) additional combination KK blocks were added to obtain intermediate combined hydrographs for sub-basins UC78 and UC79, (I7879) as well as for sub-basins UC62 and UC66 (I6266).

Rainfall depths were taken from NOAA Atlas 14 as incorporated in the DDMSW version 4.6 software. An average rainfall depth was computed using the DDMSW software using a polygon for the contributing watershed inflows to the PPW ADMS FLO-2D models.

A 25-year, 24-hour and 10-year, 24-hour model were also developed directly from the 100-year model with the only change being the point rainfall depths. The rainfall depths for the revised HEC-1 model are shown in Table 16.

Table 16. Comparison of Point Rainfall Depths for the Carefree ADMP HEC-1 Model

Storm	Rainfall Depth (inches)	
	Initial Carefree DMP	Revised Model*
10-year 24-hour	N/A	2.924
25-year 24-hour	N/A	3.601
100-year 24-hour	4.60	4.755

*The Revised Model was updated with NOAA Atlas 14 rainfall and used to generate inflow hydrographs into the PPW ADMS FLO-2D model(s).

The HEC-1 model area is shown with the DDMSW precipitation depths in Figure 22.

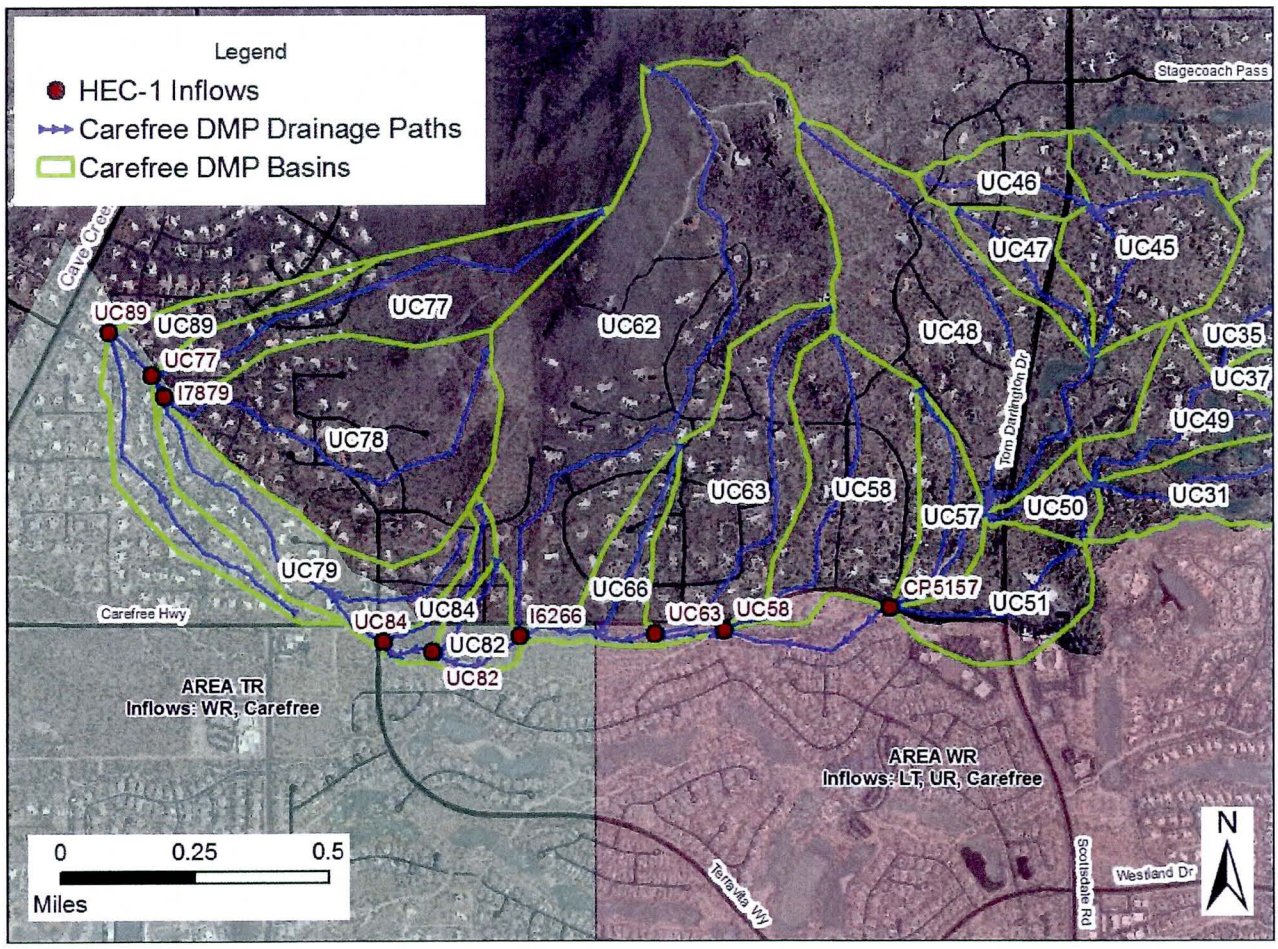

Figure 21. Carefree DMP HEC-I Sub-basins and Inflow Locations.

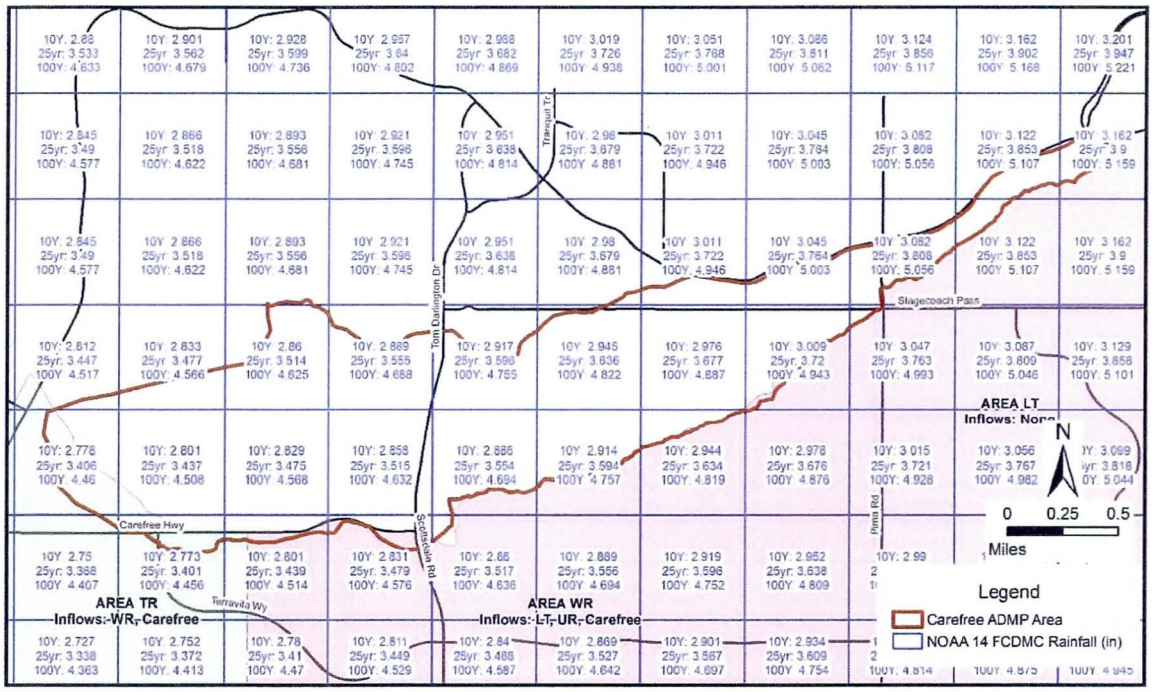

Figure 22. Carefree DMP HEC-I Model Area-NOAA Atlas 14
10- and 100-year, 24-hour Rainfall point precipitation depth (in)

Hydrographs for each model inflow point were generated from the HEC-1 models using HEC-DSS. HEC-DSS is the HEC Data Storage System program that is used to store, retrieve, and view sequential data located in a database. HEC-1 works with the HEC-DSS database through the use of the ZW records in the model that writes the model hydrograph to the HEC-DSS database. Figure 23 shows a plot of the central portion of the 100-year 24-hour inflow hydrographs. A summary of the Carefree DMP inflow hydrographs and the subsequent location of inflow onto the PPW models are shown in Table 17. To maintain stability in the model, the inflows for concentration points CP5157, I7879, and I6266 were split across multiple FLO-2D grids to maintain the 1cfs per 1 sq-ft ratio recommended by FLO-2D, this ratio correlates to 400cfs per 20-foot grid. Concentration points I6266 and I7879 were split into two grid elements and CP5157 was split into 6 different grids.

Table 17. HEC-1 Inflow Peak Discharges

Hydrograph ID	Inflow to FLO-2D Model Domain	FLO-2D Grid Cell ID	100-Year, 24-Hour		25-Year, 24-Hour		10-Year, 24-Hour	
			Qpeak (cfs)	Tpeak (hrs)	Qpeak (cfs)	Tpeak (hrs)	Qpeak (cfs)	Tpeak (hrs)
UC84	Area-TR	27498	20.95	12.033	13.98	12.03	9.73	12.07
UC82	Area-TR	29333	38.56	12.033	25.50	12.03	17.57	12.03
UC63	Area-WR	66458	217.42	12.033	153.57	12.03	114.7	12.03
UC58	Area-WR	65070	222.32	12.033	160.70	12.03	123.31	12.03
UC77	Area-TR	2960	188.27	12.033	134.36	12.03	101.63	12.03
CP5157*	Area-WR	Multiple**	2,361.43	12.133	1,671.38	12.13	1,267.4	12.10
UC89	Area-TR	1456	113.33	12.067	78.78	12.07	57.69	12.07

Hydrograph ID	Inflow to FLO-2D Model Domain	FLO-2D Grid Cell ID	100-Year, 24-Hour		25-Year, 24-Hour		10-Year, 24-Hour	
			Qpeak (cfs)	Tpeak (hrs)	Qpeak (cfs)	Tpeak (hrs)	Qpeak (cfs)	Tpeak (hrs)
I7879*	Area-TR	3953, 4047	477.33	12.000	330.62	12.00	241.31	12.03
I6266*	Area-TR	26485, 26125	747.57	12.033	531.41	12.03	399.62	12.03

*Inflows split into multiple FLO-2D grid cells.
**CP5157 inflow elements are 59264, 58687, 58113, 57543, 56976, and 56414.

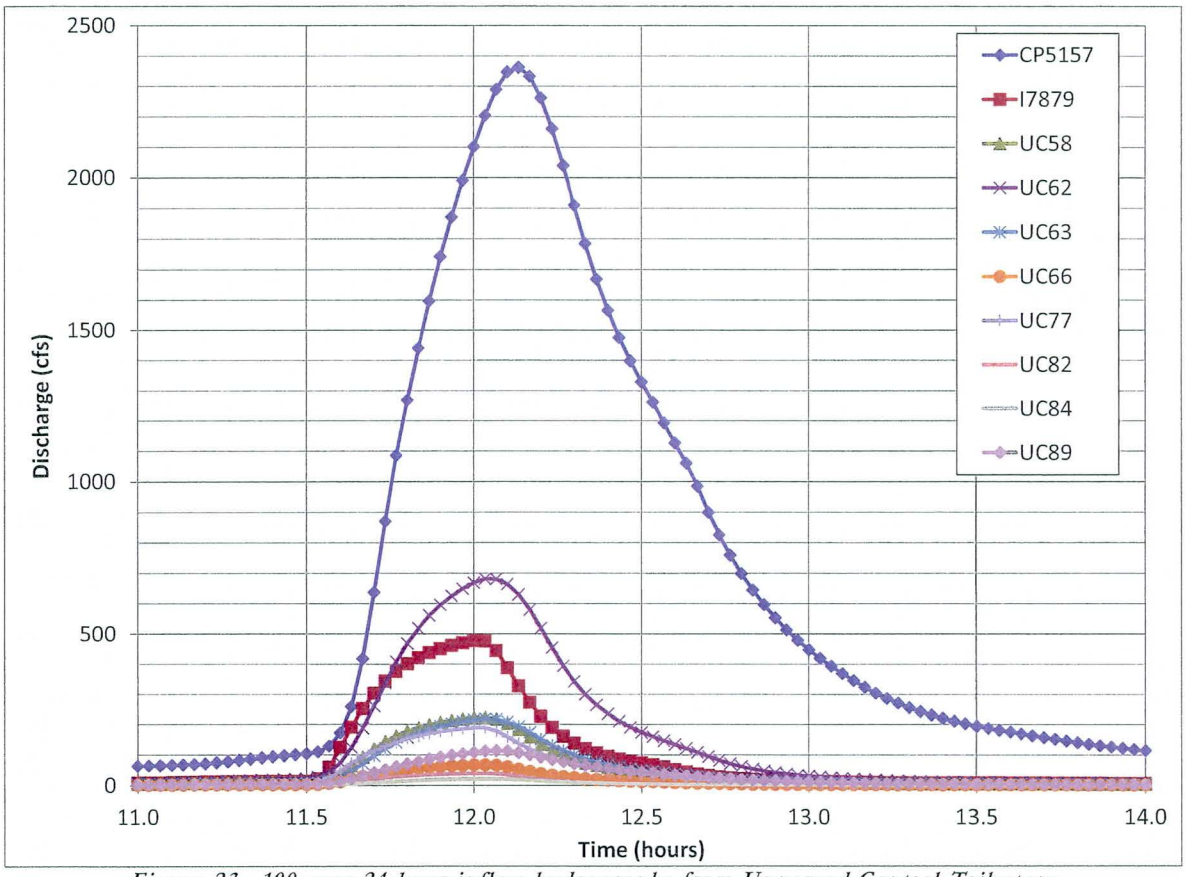

Figure 23. 100-year 24-hour inflow hydrographs from Unnamed Central Tributary

4.3.2. Pinnacle Peak South (PPS) ADMS Inflow Hydrographs

The PPS ADMS flows westerly into the eastern sides of PPW ADMS FLO-2D sub-areas UR and LR. The PPS model domain extends into the PPW domain, as shown in Figure 24, in order to delineate the inflow boundary location for the PPS inflows into PPW. The overlap area is roughly 9.7 square miles. In order to write the PPS outflow hydrographs into the PPW models, additional outflow nodes were added into the PPS model. The additional nodes extended the full north-south length of the domain boundary between the PPW and PPS models, see Figure 24. With the additional outflow nodes in place, the models were run and the outflow hydrographs generated from the outflow nodes in the OUTNQ.OUT file were extracted for writing to the INFLOW.DAT files of the Downstream PPW models. It should be noted that the PPS models were run with an older, different

version of the FLO-2D software (Version 2009.06, Build No. 09-12.02.08, dated 04-03-2012) per direction from the District.

The PPS model is constructed using a 30-foot grid whereas the PPW grid is constructed with a 20-foot grid size. The CADPTS coordinates of the PPW models are set in a manner so that every other node of the PPW CADPTS line up along the PPS/PPW boundary with the PPS CADPTS, see Figure 25. The generation of the PPW CADPTS files is discussed in more detail in Section 4.2.1. The locations where the PPW and PPS CADPTS are in the same X-Y location, the PPS outflow hydrograph is written directly to the corresponding PPW node as inflow with a one-to-one relationship (the hydrographs are exactly the same). At locations where the PPS CADPTS are in between two PPW CADPTS, the PPS outflow hydrograph is divided by two with each PPW CADPT on either side of the PPW outflow node receiving equal halves of the outflow hydrograph as inflow. Outflow nodes that do not have any discharge written to the PPS model OUTNQ.OUT file are not written as inflow nodes.

Due to the process of mapping the PPS model outflow nodes directly to the PPW model inflow nodes, the ratio of 1 cfs to 1 sq-ft as recommended by FLO-2D was not adhered to. In the attempt to maintain the PPS flow patterns as closely as possible, and due to the high number of inflow grid elements coming from the PPS model, the outflow nodes were mapped directly into the PPW FLO-2D model as shown in Figure 25 and no distinct inflow hydrograph division was done to maintain the 1cfs to 1 sq-ft ratio. The PPW FLO-2D model indicates stable transitions along the inflow boundary; the recommended FLO-2D ratio is strictly a recommendation based on model stability and runtime and exceeding the ratio is not a direct FLO-2D error. The PPS model area that overlaps into the PPW model domain, see Figure 25, was not used in the PPW model(s). The PPS model routed flow to the outflow nodes lining the PPS/PPW boundary and those flows were directly imported into the PPW models.

The PPS models were used to generate inflows for the 100-year 24-hour and 10-year 24-hour events. The models were run under two scenarios, a with-LEVEE.DAT scenario and a without-LEVEE.DAT scenario. The PPS model uses the LEVEE.DAT file to model the property walls in the watershed. A discussion regarding the property walls in the PPW watershed is detailed in Section 5.5.3. The PPS with-LEVEE.DAT scenario was used to generate the inflows for the two "With-Walls" scenarios(one with failure and one without failure) for PPW and the PPS without-LEVEE.DAT scenario was used for the "Without-Walls" scenario for PPW (see Section 5.5.3 for the PPW scenario discussion). The PPS watershed was not modeled under the 25-year event; the FCDMC ratios of the 100-year event to the 2-, 5-, and 10-year recurrence intervals was used as a basis to determine the ratio of the 100-year event to the 25-year event. The FCDMC ratio data (Table 6.1 in the District Hydrology manual), shown in the table below, was used to compute the 25-year ratio of 0.55 through linear interpolation (PPS Q25=Q100*0.55). The hydrographs from the PPS 100-year model were multiplied by 0.55 to generate the inflow hydrographs for the 25-year PPW models.

Table 18. District Ratios to 100-Year Flood Hydrographs

Recurrence Interval	Ratio %
2	10
5	25
10	35
25	55*
100	100

*Interpolated Value

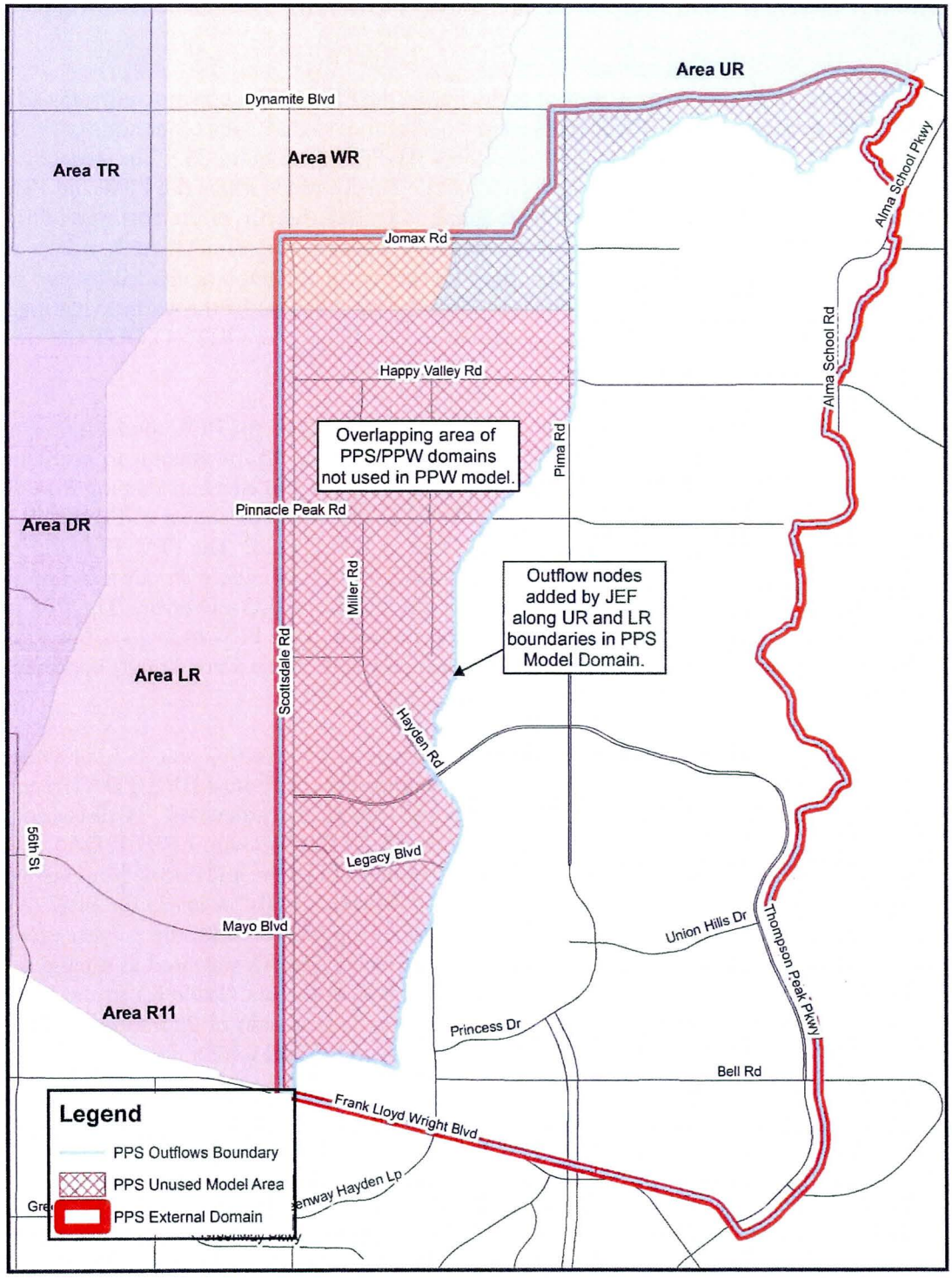

Figure 24. PPS/PPW Boundary with Additional Outflow Nodes

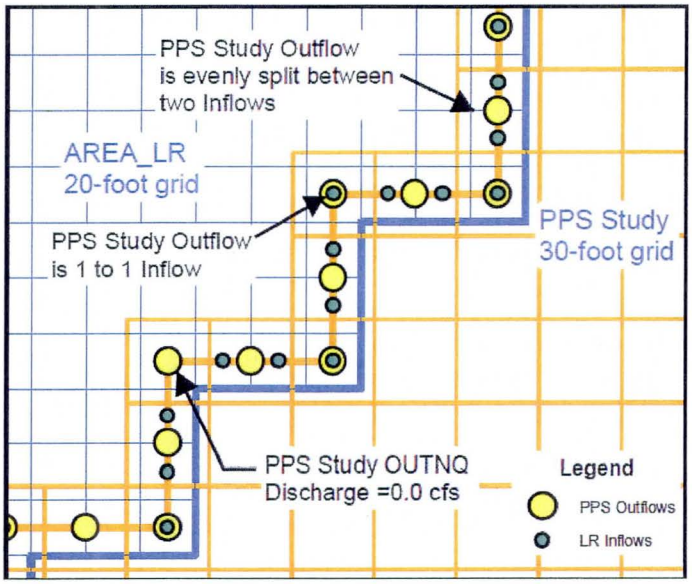

Figure 25. PPS/PPW CADPTS Alignment

4.4. 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. There are approximately 1,600 floodplain cross-sections in the complete PPW study area.

4.5. Model Control Parameters (CONT.DAT and TOLER.DAT)

The control parameters and stability criteria that were used for the FLO-2D models are summarized in Table 20. The default SHALLOWN value of 0.10 was selected for the models. 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) and spatially variable Froude Number in FPFROUDE.DAT were set at a value of 1.00 limiting the flow regime to critical for the entire watershed with the exception of a drop structure on Rawhide Wash at Pinnacle Peak Road. At this structure, the spatial limiting Froude number was set to 2.0 to allow for supercritical flow down the drop. The watershed-wide critical flow regime determination was set through the evaluation of output from preliminary model runs. The preliminary runs were modeled assuming a sub-critical flow regime with the global Froude numbers limited to 0.9 throughout the watershed and another allowing supercritical flow by setting the Froude number to a global value of 1.1. The grid element roughness revisions in the model ROUGH.OUT files were evaluated and plotted spatially to see the location of the majority of the highest revisions. FLO-2D model roughness revisions can occur as the model increases the grid element roughness to force the flow regime into the sub-critical regime specified by the limiting Froude number of 0.9. The roughness revisions were primarily in some of the larger washes where flow was concentrated. In general, the number of revisions were a low percentage of the overall number of grid elements for each model, typically less than 1%. The Froude number was set to 1.0 based on the following factors:

- The overall number of revisions is minor (less than 1% of the total number of grid elements) in the preliminary model runs,
- Timing impacts on hydrographs were minor when compared to model runs that allowed super-critical flow(see Figure 26 and Table 19),
- Should the study be used for future flood hazard identification for FEMA studies, the flow regime is not desired to be above critical.

Two floodplain cross-section locations were checked in the WR sub-area model where grid element adjustments occurred, WR_061 and WR_289. This location is at a wash split located downstream of Leaning Rock Rd, WR_061 is on the main wash and WR_289 is on the split wash. As expected, the sub-critical model sent slightly more flow down the split as a result of slightly higher water surfaces and the super-critical model sent more flow down the main wash. A comparison of the flows is shown in Table 19 below and the hydrographs are shown in Figure 26. The impacts on the hydrographs and peak discharges are negligible.

Table 19. Comparison of Peak Discharges and Time to Peaks of Sub-Critical and Super-Critical

Floodplain Cross-Section ID	Sub-Critical (FPFROUDE=0.9)		Super-Critical (FPFROUDE=1.1)	
	Qp (cfs)	Tp (hrs)	Qp (cfs)	Tp (hrs)
WR_061	1,653.53	13.46	1,659.9	13.43
WR_289	525.6	13.47	520.76	13.48

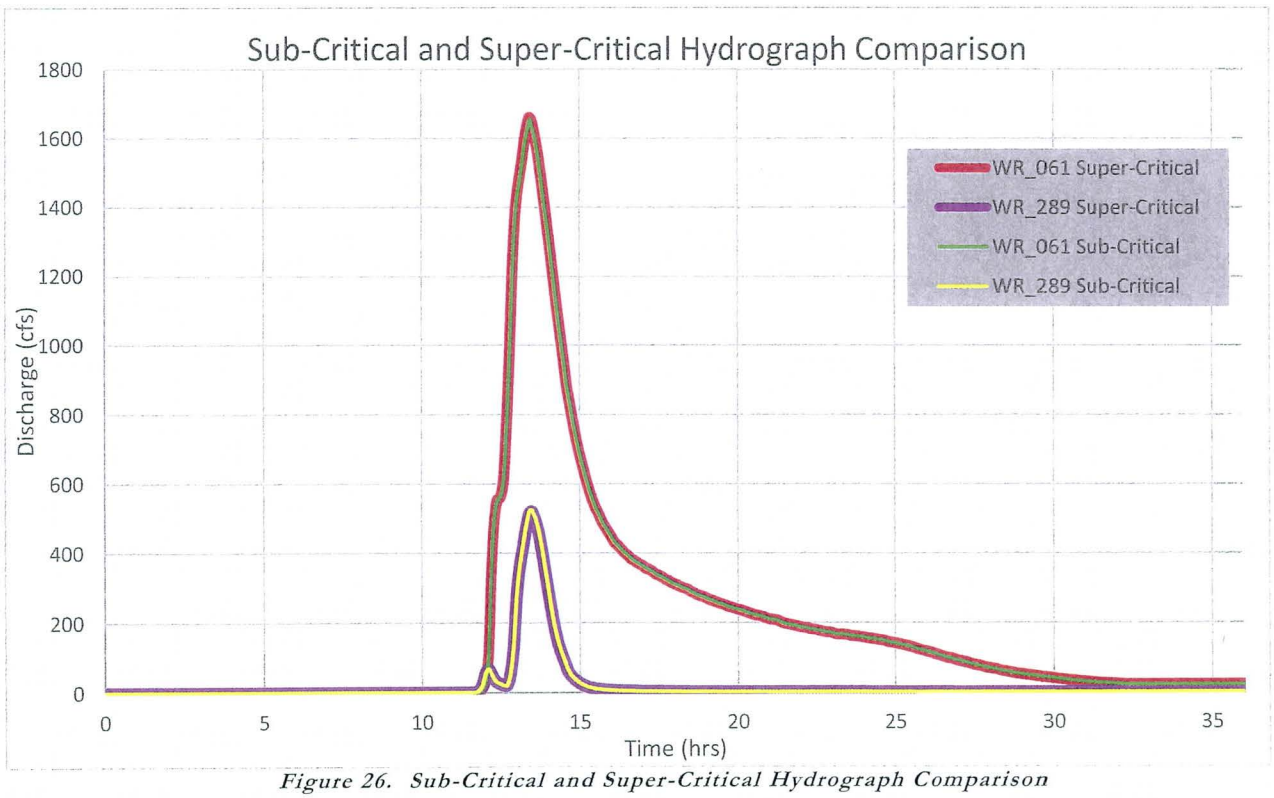

Figure 26. Sub-Critical and Super-Critical Hydrograph Comparison

The surface detention parameter (TOL) was set as close as possible to the lowest initial abstraction value of 0.05 inch to 0.048 inches (0.004 feet). The TOL value is subtracted from initial abstraction (IA) values in the INFIL.DAT file, therefore, initial IA values of 0.05 inches become 0.002.

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 initial model files use a default Courant number of 0.60 with an incremental timestep change coefficient default of 0.10. A Courant value of 0.2 was used for the channel routine to improve the stability of the channel computations. Initial model runs with a channel Courant at 0.6 indicated some velocity surging in the channels. Reducing the courant value to 0.2 stabilized the model and did not appear to have a large impact on model runtime.

Table 20. Summary of FLO-2D 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	0.60
COURANTC	Channel Courant values	0.20
TIMEACCEL	Time Acceleration	0.10
FRFROUDE.DAT		
FPFROUDE	Spatially Varying Froude Number	1.00

4.6. Issues Encountered During the Study

There are no unique issues that were encountered as part of the hydrologic modeling. However, since FLO-2D is a hydrologic and hydraulic model combined, there are conditions of the FLO-2D models that are more unique to the hydraulics of the model, which relate mostly to the hydraulic structures and one-dimensional channels. While it is understood that the hydraulic modeling within FLO-2D affect the hydrologic results, any issues related directly to the hydraulics, regardless of the fact that they impact the hydrology, are discussed in the hydraulics portion of this report in Section 5.7.

4.7. Special Issues and Solutions

4.7.1. Central Arizona Project Canal

The CAP canal is not modeled in the PPW study; it is located outside of the computational domain. The CAP canal system itself is not impacted by the PPW watershed. The embankment on the north side and adjacent to the CAP canals' northern bank (Reach 11 Dikes) is elevated to fully retain runoff from events much larger than the 100-year, the runoff would be impounded against the embankment preventing overflow. While there are outflow pipes with gates on the Reach 11 dikes that are able to outlet ponded water to the CAP canal, these were not modeled to provide a conservative ponding estimate against the embankment. There are two outlet pipe locations in the Reach 11 Dike embankment on the north side of the CAP canal, one near the 62nd Street

alignment and another near the 44th Street alignment. It is not expected that these outlet pipes will be open during a 100-year event for the following reasons:

- It is unknown when CAP would consider the opening of the outlets but it is anticipated to be only under conditions where the dike may be overtopped, an event much greater than the 100-year event would be required for overtopping.
- It is unlikely that CAP would consider opening the outlets and allow debris and sediment laden water into the canal system unless the integrity of the dikes is threatened.
- It is more conservative for assessment of the upstream ponding limits to model the scenario with the outlet gates closed.

4.7.2. Sub-Area R-11 (Reach-11 Dikes) Model

The R-11 model was developed to simulate the areas of deep ponding that occur against the Reach-11 dikes; the R-11 model domain area overlaps with the DR and LR models. Initially, the PPW watershed models were generated without the R-11 model domain and, after several iterations of lengthy model runtimes for Areas DR and LR due to the deep ponding, it was decided to create the R-11 model to decrease the DR and LR model runtimes. Instead of recreating the DR and LR model domains and re-number structures and channels with new grid numbers, the DR and LR model domains were kept the same and outflow nodes were placed along the LR/DR and R-11 interaction boundary line. The outflow from LR and DR is transformed into INFLOW.DAT input files for the R-11 model using the same methods as described in Section 4.2.7.

Due to the overlap between the R-11 and LR/DR models, the RAIN.DAT files for LR and DR were revised to avoid duplication of rainfall in the overlapping area; the R-11 model accounts for the rainfall in this area. The RAINARF values for LR and DR were set to 0.0 in the area that overlaps with the R-11 domain, see Section 4.2.5.1 for a discussion on RAINARF values.

4.8. Modeling Warning and Error Messages

The following error messages are reported in the FLO-2D output file ERROR.CHK, these warnings are related to hydrologic and hydraulic modeling components:

- 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 GLOBALLY UNDERESTIMATED THE RTIMP PARAMETER.
  FOR THIS SIMULATION THE RTIMP IS RESET TO THE ARF VALUE, HOWEVER, YOU SHOULD REVIEW ALL THE RTIMP ASSIGNMENTS.

  This message occurs because the maximum RTIMP assigned to grid elements 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 elements 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 on roofs for example. This error is considered conservative, but will likely be unperceivable in the model results.

- WARNING: THE FOLLOWING HYDRAULIC STRUCTURES HAVE LEVEES IN EITHER THE INLET OR OUTLET ELEMENTS: (THIS WOULD ONLY BE A PROBLEM IF THE LEVEES INTERFERE WITH THE HYDRAULIC STRUCTURE RATING TABLE OR CURVE.)
  NAME NO. INLET NODE OUTLET NODE

This message is occurring as the result of having levees with hydraulic structures. Primarily this is the result of the wall opening structures. The levees do not interfere with the hydraulic structure routine.

- WARNING: THE FOLLOWING HYDRAULIC STRUCTURES HAVE CONTIGUOUS INLET AND OUTLET ELEMENTS AND THE FLOW IS ONLY SHARED BETWEEN THEM THROUGH THE HYDRAULIC RATING TABLE OR CURVE (THERE IS NO OVERLAND FLOW):
  NAME NO. INLET NODE OUTLET NODE

  This message is typically the result of the wall openings which transfer flow though contiguous elements to move flow across a levee via a wall opening modeled as a hydraulic structure.

- 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).

  This warning is related to a depth/discharge disparity between the flow entering a structure inlet grid and the flow through the structure as referenced by the structure rating curve data. For example, an inlet grid might have 0.25 feet of depth and 1 cfs discharge, however, the hydraulic structure rating curve might reference a discharge of 1.5 cfs at 0.25 feet of depth. Almost all of these generated warnings relate to the leading and/or receding limbs of the hydrographs and flows are relatively minor and negligible, these warnings do not impact the results or affect the peak of the hydrograph.

- WARNING: THE RATING TABLE FOR HYDRAULIC STRUCTURE: WAS ADJUSTED TO BETTER MATCH THE STREAM FLOW CONDITIONS.

  This warning is notifying the modeler that the rating curve has been adjusted in order to stabilize the model/structure; revised ratings are written to a separate output file REVISED_RATING_TABLES.OUT. This occurs primarily where there is a structure outlet into a sump condition. These warnings were investigated during the preliminary modeling phase of the study and adjustments were typically made to either the outlet grid elevations or by blending the revised rating curve section from the REVISED_RATING_TABLES.OUT file and the original rating curve to create a smooth curve. A discussion of this procedure is in Section 5.5.2.4.

- 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

  This warning indicates that the water surface elevation is higher at the outlet than the inlet. The PPW models all assume that flow will only go downstream through a structure and that runoff will not back up through the structure. Structures that have this warning during the typical peak hours (11.0 through 15.0) were investigated to determine if it would be prudent to allow the flow to back up through the structure. In all locations it was considered to be unnecessary to change the INOUTCONT switch to allow the upstream flow. Locations where this typically occurred were either in open desert at roadway crossings or in retention basins/engineered channels.

- *** THERE ARE DRY OUTFLOW NODES FOR THE FOLLOWING DOWNSTREAM GRID SYSTEM: *** GRID CELL: ***

This warning is due to the placement of outflow nodes along long portions of the model boundaries. These nodes are located along peaks and ridges that receive no contributing runoff.

- THE HYDRAULIC STRUCTURE NO. RATING TABLE WAS REVISED. REVIEW THE SUGGESTED RATING TABLE IN REVISED_RATING_TABLE.OUT FILE.

  This warning is related to the warning indicating that structures were adjusted to better match flow conditions. It is a warning to alert the user that the ratings were adjusted and to look in the REVISED_RATING_TABLES.OUT file for the revised segment of the rating table.

- WARNING: THE HYDRAULIC STRUCTURE OUTFLOW NODE: IS REPEATED MORE THAN ONCE WITHOUT ASSIGNING A D-LINE CONVEYANCE CAPACITY LIMITATION. EITHER REVISE THE OUTFLOW NODES OR ADD A D-LINE

  This warning indicates that there are multiple outlet locations in the HYSTRUC.DAT file. This warning is not an issue since in many cases there are multiple structures that have outlets to the same location/grid. The conveyance capacity does not need to be limited since the capacity is specific to the generated rating table of each individual structure. In some cases there are large culverts that span multiple grids that outlet to a channel grid element. In this case, the outlet must be a single grid that designates the channel left bank element.

- WARNING: THE RATE OF CHANGE IN THE HYDRAULIC STRUCTURE NO. RATING TABLE MAY BE UNREASONABLE FOR THE OR PREVIOUS STAGE INTERVALS

  This warning is stating that the rate of change may be too great in between the depth intervals. These warnings do not impact the peak and are related to the leading and receding limbs of the structure hydrographs. The hydrographs of the structures appear reasonable and the ratings with these warnings are typically adjusted by FLO-2D through the REVISED_RATING_TABLES.OUT file.

4.9. Calibration

As noted in the limiting infiltration depth discussion in Section 4.2.6.1, the FLO-2D model infiltration was calibrated to HEC-1 models developed for each sub-area so that the FLO-2D models have roughly the same percentage of infiltration as the HEC-1 models. The limiting infiltration depth parameter was adjusted until the FLO-2D results showed nearly the same amount of infiltration and interception as the HEC-1 models.

There are several gages present in the watershed, see Section 4.2.3, but stream gages are sparse and have not been in place long enough to produce sufficient data to calibrate to. Stream gage ID 4918, Cave Cr. Nr Cave Creek, has been in place the longest at 20-years but, while just barely located within the PPW watershed, the gage measures flow in Cave Creek which is not a part of this study. There are two other stream gages within the watershed, IDs 4913 and 4863, Stagecoach Wash and Rawhide Wash respectively; these gages have been in place less than 20-years. While the data from these gages are used to verify the model results, they were not used for calibration of the model(s).

There have been some significant rainfall events recorded within the watershed and, similar to the stream gages, this data was used to verify the model results but not for calibration. With the lack of applicable data to calibrate to, with respect to actual events and recorded data, the sole source of model calibration is the infiltration calibration noted in Section 4.2.6.1.

4.10. Final Results

FLO-2D extracts the peak discharges and hydrographs from the model through the use of floodplain cross-sections (FPXSEC.DAT input file). Although they are called cross-sections, they are not a hydraulic modeling parameter, rather they record the flux of flow across the section. Nearly 1,600 floodplain cross-sections are located throughout the PPW watershed. Due to the high number of sections, a summary table is not placed in the text of the report. A complete summary of the hydrologic output, peak discharge, time-to-peak, and hydrograph volume is located in the appendix. The floodplain cross-section output (HYCROSS.OUT) and spreadsheets containing the hydrograph plots are located with the model input and output with the digital data. A shapefile of the floodplain cross-sections attributed with the peak discharge and time-to-peaks for each wall scenario is included with the digital data.

4.11. Verification of Results

The results of the PPW ADMS FLO-2D models have been compared against a variety of data sources for verification of results and include FEMA FIS data, existing hydrologic studies, indirect methods verification using the FEMA FIS data, and recorded/observed data from actual flow events.

4.11.1. FEMA FIS DATA

There are three primary FEMA floodplain systems in the PPW watershed, Rawhide Wash, Fans 5, 6A, 6B, and 6C (Fans 5 and 6), and Stagecoach Pass Wash. There are several smaller washes that have available FEMA FIS discharges associated with them, Eastern Pima Wash, Unnamed Tributary to Stagecoach Pass Wash, and various tributaries to upper Rawhide Wash. A summary of the FEMA FIS discharges (Effective 2013) compared to the PPW model results are shown in Table 21, the PPW results are from the "Without-Wall" scenario.

Table 21. Summary of FEMA FIS discharges compared to PPW Results

FEMA Flooding Source	Area (sq. mi.)	100-Year Peak Discharge (cfs)	PPW Floodplain Cross-Section ID	PPW 100-Year Peak Discharge (cfs)
Basin 5 (At Apex)	3.09	2,849	**WR_008**	1,846
Basin 6A (At Apex)	3.32	3,382	**LT_011**	2,811
Basin 6B (At Apex)	0.43	562	**WR_060**	275
Basin 6C (At Apex)	1.49	1,475	**WR_063**	451
Eastern Pima Wash (Approx 40 ft. Upstream from Stage Coach Pass)	0.13	323	**LT_107**	279
Rawhide Wash (Downstream of Dynamite Boulevard)	9.3	7,319	**Multiple***	6,988
Rawhide Wash (At Dynamite Boulevard)	8.9	7,157	**UR_016**	6,973
Rawhide Wash (Downstream of Via Dona Road)	8.8	7,153	**UR_015**	6,994
Rawhide Wash (Downstream of Lone Mountain Alignment)	8.6	7,150	**UR_014**	6,949
Rawhide Wash (96th St. Alignment)	6.8	6,755	**UR_073**	6,194
Rawhide Wash (Confluence with Tributary 1 to Rawhide Wash)	6.4	6,600	**UR_012**	6,032

FEMA Flooding Source	Area (sq. mi.)	100-Year Peak Discharge (cfs)	PPW Floodplain Cross-Section ID	PPW 100-Year Peak Discharge (cfs)
Rawhide Wash (Downstream of Confluence with Tributaries 2,3, and 4 to Rawhide Wash)	6.1	6,445	**UR_010**	5,943
Rawhide Wash (Upstream of Confluence with Tributaries 2,3, and 4 to Rawhide Wash)	5.3	5,666	**UR_008**	5,195
Rawhide Wash (At Diversion)	1.4	731	**LT_077**	797
Rawhide Wash (South of Tonto National Forest Boundary, approximately 3,340 feet)	1.1	1,111	**LT_096**	1,842
Rawhide Wash (South of Tonto National Forest Boundary, approximately 2,560 feet)	0.8	867	**LT_057**	1,673
Tributary 1 to Rawhide Wash (Approximately 5,600 feet upstream of confluence with Rawhide Wash)	0.6	648	**UR_009**	995
Tributary 2 to Rawhide Wash (Approximately 7,400 feet upstream of confluence with Rawhide Wash)	1.3	2,841	**UR_059**	2,287
Tributary 3 to Rawhide Wash (Approximately 3,800 feet upstream of confluence with Tributary 2 to Rawhide Wash)	1.1	1,633	**UR_054**	1,327
Tributary 4 to Rawhide Wash (Approximately 4,000 feet upstream of confluence with Tributary 2 to Rawhide Wash)	1.5	1,875	**UR_060**	2,053
Tributary 4 to Rawhide Wash (Approximately 7,700 feet upstream of confluence with Tributary 2 to Rawhide Wash)	1.3	1,766	**UR_003**	2,055
Stagecoach Pass Wash Unnamed Tributary (At confluence with Stagecoach Pass Wash)	0.12	225	**LT_102**	154
Stagecoach Pass Wash (At confluence with Scottsdale Road)	1.6	1,996	**WR_025**	1,149
Stagecoach Pass Wash (At confluence with Pima Road)	1.1	1,308	**LT_137**	837
Stagecoach Pass Wash (Approximately 600 feet upstream of confluence with Stage Coach Pass)	1.01	1,116	**LT_104**	830
Stagecoach Pass Wash (At east boundary with Town of Carefree and City of Scottsdale)	0.54	844	**LT_249**	561

FEMA Flooding Source	Area (sq. mi.)	100-Year Peak Discharge (cfs)	PPW Floodplain Cross-Section ID	PPW 100-Year Peak Discharge (cfs)
Stagecoach Pass Wash (At the northeast corner of Section 32, Township 6N, Range 5E)	0.304	758	**LT_099**	529

*A floodplain cross-section was not located near the exact spot as the FEMA FIS study. PPW floodplain cross-sections UR_016, UR_126, and UR_128 were added together (neglecting the instantaneous peaks) to obtain the PPW discharge value.

In some locations there are significant differences between the PPW model results and the effective FEMA FIS discharges, primarily near the more northern sections of Rawhide Wash, the downstream end of Stagecoach Pass Wash, and Fan 5, Fan 6B, and Fan 6C. Aside from different input parameters (different rainfall and infiltration parameters), the different modeling methods used between the FEMA FIS data and PPW is a primary reason of some of the disparities. The PPW discharges are being compared to the discharges and the associated contributing area listed in the FIS study. In a one dimensional analysis where flow is routed from concentration point to concentration point, the contributing area can be easily calculated and documented as flow is conveyed from point to point via single section flow routing. However, in FLO-2D, the contributing area is more difficult to analyze since numerous flow splits and/or confluences can affect the discharge results and the associated contributing area. Furthermore, the effects of transmission losses and flow attenuation due to hydraulic structures, varying streambed sections, and on-grid storage outside of the streambed, items not typically accounted for in a 1D model, also can have a significant impact on the FLO-2D discharge. More specific details regarding the more significant discharge disparities are discussed in the following sub-sections.

Rawhide Wash System:
Generally, the PPW discharges are similar to the majority of the FEMA discharge locations along Rawhide Wash with the exception of the FIS points near the Tonto National Forest Boundary, Tributary 1 upstream of the confluence with Rawhide Wash, and Tributary 4 7,700 feet upstream of Tributary 2 (the PPW discharges are higher than the FEMA FIS).

In the upper watershed of the Rawhide Wash system, the PPW flow results are higher due primarily to higher modeled rainfall depths. The PPW FLO-2D models use the actual point rainfall depths from NOAA Atlas 14 data; the rainfall depths are assigned on a grid-by-grid basis, see Section 4.2.5.1 for more detail on the rainfall data development. In the upper watershed for PPW, the 100-year, 24-hour rainfall depths are typically in the range of 5.5 inches in the most upstream portion of the watershed to 4.99 inches at the confluence of Rawhide Wash and Tributary 1 to Rawhide Wash. The FEMA FIS discharges are the results from the Upper Rawhide Wash Floodplain Delineation Study (Rawhide Wash FDS). The Rawhide Wash FDS modeled the 100-year, 24-hour event using NOAA Atlas II point rainfall data with area reduction factors applied using HEC-1. The maximum point rainfall depth was 4.60 inches, considerably less than the 5.5 to 4.9 inch range used on the grid-by-grid basis in the FLO-2D models. The higher rainfall combined with different modeling techniques and infiltration parameters contribute to the higher PPW discharges.

Moving downstream along Rawhide Wash, the rainfall of the PPW ADMS begins to match more closely with the Rawhide Wash FDS and the effects of flow splits, infiltration, and floodwave attenuation begin to bring the PPW ADMS discharges to roughly the same magnitude as the FEMA FIS discharges. The difference in modeling of

the flow splits that occur near Rawhide Wash and upper Fan 5 also contribute to the differences in the PPW Study and the Rawhide Wash FDS, see Section 4.11.2 for more detail on the Rawhide Wash FDS.

Fans 5 and 6:
The effective FEMA discharges for Fans 5 and 6 are based on a 1991 study completed by Water Resource Associates (WRA). There is little to no data available for this study so it is unknown exactly how the discharges were calculated. In the FIS data, Table 3. Summary of Discharges states for Basin 5 that the "Area includes portion of Basin 4D from which runoff can be diverted into Basin 5". This statement eludes to the fact that there are flow splits and/or diversions that impact the discharge for this basin. The PPW results show that there are two major flow splits that route flow from the area contributing to the Fan 5 apex and into the Rawhide Wash system (floodplain cross-sections UR_031 and LT_077). The combined peak discharge of these cross-sections is roughly 1,000 cfs. While it is unknown how these splits were modeled in the WRA study, they can clearly have an impact on the model results and that is a potential source of differences between the two models, aside from input parameter differences. The North Scottsdale Floodplain Delineation Study (North Scottsdale FDS-discussed in more detail in Section 4.11.2), covers the Fans 5 and 6 area and the resulting discharge from that study indicates a peak discharge of 2,019 cfs, lower that the FEMA 2,849 cfs discharge and closer to the PPW discharge of 1,846 cfs. While there are differences between the North Scottsdale FDS results and modeling techniques when compared to PPW, the intent is to illustrate that other studies have been conducted in the area with lower discharges than the listed effective FEMA discharges.

Like Basin 5, the effective FEMA discharge for Fan 6B is from the 1991 WRA study. At the Basin 6B apex the PPW discharge is 275 cfs, a significant decrease when compared to the FEMA discharge listed at 562 cfs. However, there is a hydraulic structure located where the streambed crosses Pima Road; this structure was likely not constructed in 1991, and therefore not modeled in the WRA study. This structures provides an excellent example of the attenuation that can occur in a FLO-2D model as flow can pond on the upstream end of the structure and be detained and metered through the culvert. As previously noted, the PPW downstream discharge is 275 cfs whereas upstream of the structure the discharge is 342 cfs, making the difference between the PPW results and the FEMA discharge less significant. The remaining differences may be attributed to modeling input parameter differences, transmission losses and on-grid flow attenuation in the watershed outside of the streambed.

The effective FEMA FIS discharge of 1,475 cfs for Basin 6C is from the 1991 WRA study. When compared to the 451 cfs from cross-section WR_063 of the PPW results, this is a significant change in peak discharge. The watershed contributing to Basin 6C consist of a more braided shallow flow pattern when compared to surrounding contributing watersheds (Basin 5, Basin 6A). The Basin 6C watershed also contains more hydraulic structures. The braided flow patterns and numerous structures not only have the impact of flow attenuation but they also generate some significant flow splits that divert runoff out of the Basin 6C watershed. A large flow split occurs at Lone Mountain Parkway where roughly 200 cfs leaves the system to the west. Splits also occur at Legend Trail Parkway, Pima Road, and Westland Drive, these three splits have an approximate cumulative discharge of 160 cfs. Furthermore, the impacts of hydraulic structures in series can be seen by the decrease in discharge as runoff is routed through a golf course and subdivision from Legend Trail Parkway to downstream of 92nd Place. Upstream of Legend Trail Parkway, the peak discharge is 636 cfs at cross-section LT_124, downstream of 92nd Place, the discharge has dropped to 550 cfs at cross-section LT_216 after being routed through 4 structures in series. The overall cumulative impact of these structures throughout the Basin 6C watershed in conjunction with flow splits, on-grid attenuation and transmission losses is shown by the decreased peak discharge at the apex location.

Stagecoach Pass Wash:
The FEMA FIS discharge data for Stagecoach Pass Wash is based on the results of the North Scottsdale FDS, details of that study are discussed further in Section 4.11.2 below. Generally, the PPW results are lower than the North Scottsdale FIS due primarily to the modeled land use scenario. The North Scottsdale FDS appears to have modeled future land use conditions and the resulting sub-basins have impervious percentages in the range of 17% to 58% depending on the sub-basin. In the PPW FLO-2D models the percent impervious was based on a very detailed surface feature characterization shapefile of the existing conditions of the watershed (existing conditions based on 2007/2010 mapping data). The shapefile categorized data in a level of detail that separated buildings, asphalt pavement, and concrete sidewalks from open space. As a result, the FLO-2D models have a lower percentage of impervious area than the HEC-1 models from the North Scottsdale FDS, this yields lower discharges.

4.11.2. Comparison of PPW ADMS Results to Other Studies

There are three primary existing studies within the PPW watershed using one-dimensional lumped parameter modeling methods (HEC-1) and are summarized in Table 22. The PPW ADMS discharges used to compare against the other studies is the "Without-Wall Scenario" for the 100-year 24-hour event.

Table 22. Summary of Existing Studies used to Verify Results

Study Name	Date Completed	Modeled Event/Duration
Cave Creek Below Carefree Highway FIS Floodplain Delineation Study	1997	100-Year, 24-Hour
Upper Rawhide Wash Floodplain Delineation Study	2002	100-Year, 24-Hour
North Scottsdale Floodplain Delineation Study	2005	100-Year, 6-Hour

Cave Creek below Carefree Highway FIS Floodplain Delineation Study:
The Cave Creek below Carefree Highway FIS Floodplain Delineation Study (Cave Creek FDS) is difficult to use to validate the PPW results due to the flow splits and diversions in the PPW TR and CB subareas. The Cave Creek FDS is not used to validate the results further up in the watershed upstream of Scottsdale Road (PPW sub-areas WR and LT) since this area is covered by the more recent North Scottsdale FDS.

In the TR sub-area it was difficult to find locations that were not affected by flow splits when comparing results between PPW and the Cave Creek FDS. There has been a lot of development in the area since the Cave Creek FDS was completed and flows are now detained by hydraulic structures and/or rerouted by new roadways and developments. Note in Figure 27 how, according to the PPW FLO-2D results, flow goes from Cave Creek FDS sub basin 3125 and into 3130. The flow then crosses the 3130/3050 boundary and a portion of flow remains in 3050 flowing towards Cave Creek Road, however, a portion crosses Dove Valley Road at 56th Street and back into 3130. Flow crossing the boundaries in this manner occur in several locations, making it difficult to conduct an accurate comparison with the Cave Creek FDS results. Several locations that appeared to have equal contributing areas and were minimally impacted by flow splits/diversions were checked to verify PPW results versus the Cave Creek FDS, these locations are summarized in Table 23. There are, of course, disparities between the models results due to different modeling methods and input parameters. The purpose of this comparison is to verify that the PPW results are of the same order and magnitude as the previously completed FDS. The PPW peak discharge results are lower than the Cave Creek FDS and the time to peaks are generally longer than the Cave

Creek FDS. This is likely due to the cumulative impact of hydraulic structure attenuation further up in the watershed and transmission losses not accounted for in HEC-1 models.

Table 23. PPW Model Results compared to Cave Creek FDS Results

Location (Cave Creek FDS concentration point/PPW Cross-Section)	Cave Creek FDS Peak Q (cfs)	Cave Creek FDS Time to Peak (hrs)	PPW Peak Q (cfs)	PPW Time to Peak (hrs)
60th Street north of Smokehouse Trail (C3045/TR_067)	1,553	13.08	1,167	13.64
West of 40th Street and north of Ashler Hills Dr. Alignment (C3090B/CB_029)	3,003	13.83	2,006	14.94
West of Scottsdale Road and South of Amber Sun Drive (C1240/PPW_WR_030)	1,535	13.0	1,202	13.45

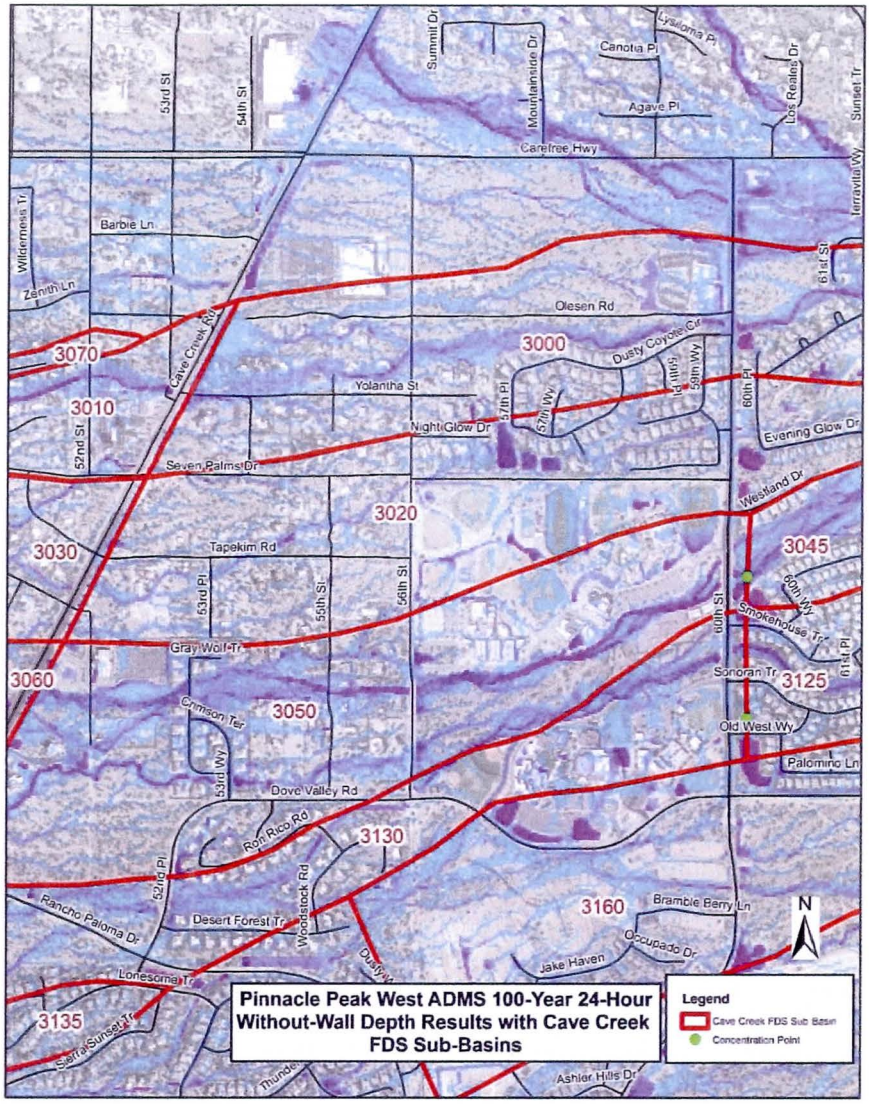

Figure 27. Cave Creek FDS Sub basins with FLO-2D Depth Results

Upper Rawhide Wash Floodplain Delineation Study:

The Upper Rawhide Wash Floodplain Delineation Study (Rawhide Wash FDS) was used as the basis for the effective 2013 FEMA FIS data for Rawhide Wash and its associated tributaries. See Table 21 for a comparison of the FEMA FIS discharge values. Some of the locations in Table 21 are also listed in Table 24 to compare the time to peaks between the PPW results and the Rawhide Wash FDS, additional locations that are not included in the FEMA FIS data are included in Table 24.

Generally, the Rawhide Wash FDS and the PPW results match up very well in both peak discharge and time to peak. As discussed in Section 4.11.1, in the upper watershed of the Rawhide Wash system the PPW results are higher than the FDS due to higher rainfall depths. The flow split located at Rawhide Wash and the upper Basin 5 watershed is modeled as a single diversion in the Rawhide Wash FDS. The incoming flow to the split is 1,111

cfs per the FDS, 423 cfs is diverted out of the HEC-1 model and 599 cfs is routed downstream into Rawhide Wash. The PPW FLO-2D models however, show two splits actually occurring. The first is at floodplain cross-section LT_057 where 213 cfs splits from Rawhide Wash and into Tributary 4 to Rawhide Wash, this is not modeled in the FDS and is another reason why the PPW model is higher than the FDS in Tributary 4. The PPW flow split at Rawhide Wash and upper Basin 5 shows 1,842 cfs coming in, 797 cfs flowing down Rawhide Wash, and 1,035 cfs going into upper Basin 5. Moving downstream, the Rawhide Wash FDS becomes slightly higher than the PPW results due to the rainfall being more in line between the two models and the overall effects of attenuation and transmission losses in the PPW models.

Table 24. PPW Model Results compared to Rawhide Wash FDS Results

Location (Rawhide Wash FDS concentration point/PPW Cross-Section)	Rawhide Wash FDS Peak Q (cfs)	Rawhide Wash FDS Time to Peak (hrs)	PPW Peak Q (cfs)	PPW Time to Peak (hrs)
Hayden Road North of Happy Valley (CP070/LR_019)	9,940	13.07	9,527	13.34
Rawhide Wash north of Jomax Road (CP068/UR_134)	9,990	12.90	8,595	13.23
Rawhide Wash Downstream of Via Dona Road (CP051/UR_015)	7,153	12.87	6,994	13.08
Rawhide Wash at Dixileta Drive (CP043/UR_013)	6,720	12.70	6,976	12.93
East of Rawhide Wash and north of Troon North Drive (CP058/UR_021)	2,569	12.33	2,205	12.48
Tributary 2 to Rawhide Wash - Approximately 7,400 feet upstream of confluence with Rawhide Wash (CP024/UR_059)	2,481	12.10	2,287	12.19
Rawhide Wash - Upstream of Confluence with Tributaries 2,3, and 4 to Rawhide Wash (CP030/UR_008)	5,666	12.43	5,195	12.73
Tributary 4 to Rawhide Wash - Approximately 7,700 feet upstream of confluence with Tributary 2 to Rawhide Wash (CP015/UR_003)	1,766	12.33	2055	12.52
Rawhide Wash - South of Tonto National Forest Boundary, approximately 3,340 feet (CP012/LT_096)	1,111	12.43	1,842	12.37

North Scottsdale Floodplain Delineation Study:

The North Scottsdale Floodplain Delineation Study (North Scottsdale FDS) was used to verify flow rates in the upper PPW watershed (sub-areas LT and WR). However, there are several factors that exaggerate the differences between the models. The North Scottsdale FDS models the 100-year 6-hour event whereas the PPW ADMS models the 100-year 24-hour event. The North Scottsdale FDS appears to be modeling the future land use conditions and the HEC-1 sub-basins have high impervious areas whereas the PPW ADMS is modeling the existing conditions land use resulting in much lower impervious percentages. Lastly, there was a soils parameter

correction in the North Scottsdale FDS. Per Section 4.3.1 of the North Scottsdale FDS TDN (North Scottsdale TDN, 2005), "*In September 2004, an error was discovered in the HEC-1 analysis consisting of soil type parameters incorrectly entered into the analysis software. Soil types were verified for the area and entered into the model correctly. End results were most significant in the Fan 6A, Fan6A North and Fan 6A South washes. 100-yr flood discharges were replaced with the new discharge values in Fan 6A, only. Reductions in the other washes can be considered statistically insignificant, therefore no changes were made to Stagecoach Pass Wash, Upper Boulders Wash, Fan 6C, or Upper Fan 5.*" While the report states that the differences in washes other than Fan 6A can be considered statistically insignificant, it is unknown exactly what those differences are and how they impact the comparison against the PPW results.

As shown in Table 25, especially for Basin 6C, the differences in modeling methods, the attenuation from the hydraulic structures and the effects of flow splits have a large impact when comparing the results between the two studies.

Table 25. PPW Model Results compared to North Scottsdale FDS Results

Location (North Scottsdale FDS concentration point/PPW Cross-Section)	North Scottsdale FDS Peak Q (cfs)	North Scottsdale FDS Time to Peak (hrs)	PPW Peak Q (cfs)	PPW Time to Peak (hrs)
South of Cave Creek Road and east of 102nd Street (CSCP11/LT_092)	676	4.15	510	12.35
Stagecoach Pass Wash at Pima Road (CSCP06/LT_266)	1,308	4.42	835	12.93
Stagecoach Pass Wash at Scottsdale Road (CSCP01/WR_025)	1,996	4.63	1,149	12.91
East of Mirabel Club Drive and North of Standing Stones Road (CF6AN7/LT_006)	1,227	4.20	1,033	12.37
Northwest of the intersection of Whitewing Drive and 92nd Place (CUB11/LT_209)	735	4.15	355	12.26
Northeast of the intersection of Legend Trail Parkway and Calvary Drive (CF6AN2/LT_079)	1,527	4.47	1,209	12.80
Stagecoach Pass west of Lone Mountain Parkway(CF6AN5/LT_136)	1,225	4.33	1,097	12.60
Southwest of the intersection of 96th Street and Aniko Drive (CSCP09/LT_099)	758	4.28	529	12.57
Pima Road south of Black Mountain Road (CUB09/LT_268)	1,270	4.18	642	12.46
Pima Road south of Black Mountain Road (CF6C2/LT_269)	1,369	4.53	523	13.28
Basin 6C apex (CF6C1/WR_063)	1,584	4.63	451	13.74
Southwest of the intersection of Legend Trail Parkway and Desert Parkway (CF6A1/LT_011)	2,698	4.85	2,811	13.19

Location (North Scottsdale FDS concentration point/PPW Cross-Section)	North Scottsdale FDS Peak Q (cfs)	North Scottsdale FDS Time to Peak (hrs)	PPW Peak Q (cfs)	PPW Time to Peak (hrs)
Northwest of the intersection of Pima Road and Ranch Road (CUF51/WR_008)	2,019	4.55	1,846	13.22

4.11.3. Indirect Methods Verification

The data from the effective FEMA FIS was used to make an indirect methods verification assessment. The indirect methods are as follows:

Method 1 - Unit Peak Discharge Curves (Figure 28)

Method 2 – USGS Data for Arizona (Figure 29)

Method 3 – Regional Regression Equations (Region 12) (Figure 30)

The FEMA FIS data and the PPW 100-year, 24-hour results listed in Table 21 have been plotted on each of the three indirect methods figures (FEMA discharges are red circles, PPW discharges are green triangles). As shown on the plots, the PPW results are contained under the applicable envelope curves for Method 1, within the 75% tolerance limits of Method 2(with the exception of the Basin 6C data point) and within the cloud of data points trending along the curve for Method 3. As noted in Section 4.11.1, the impacts of flow splits and the attenuation of flow from the hydraulic structures have a large effect on the Basin 6C discharge. Since flow leaves the Basin 6C system, technically the contributing area listed in the FEMA FIS is not realistic and it becomes extremely difficult to try and compute a true contributing area.

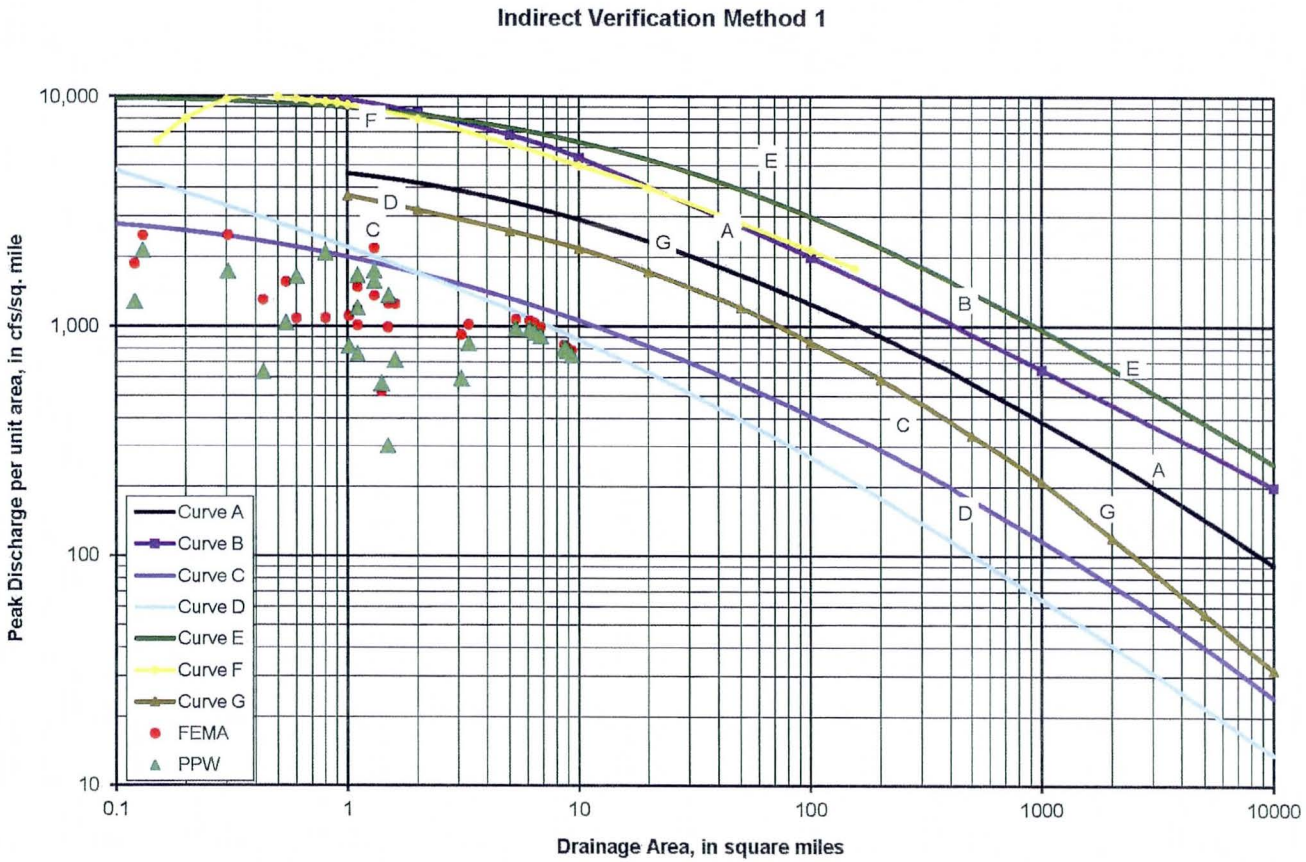

Figure 28. Indirect Verification Method 1

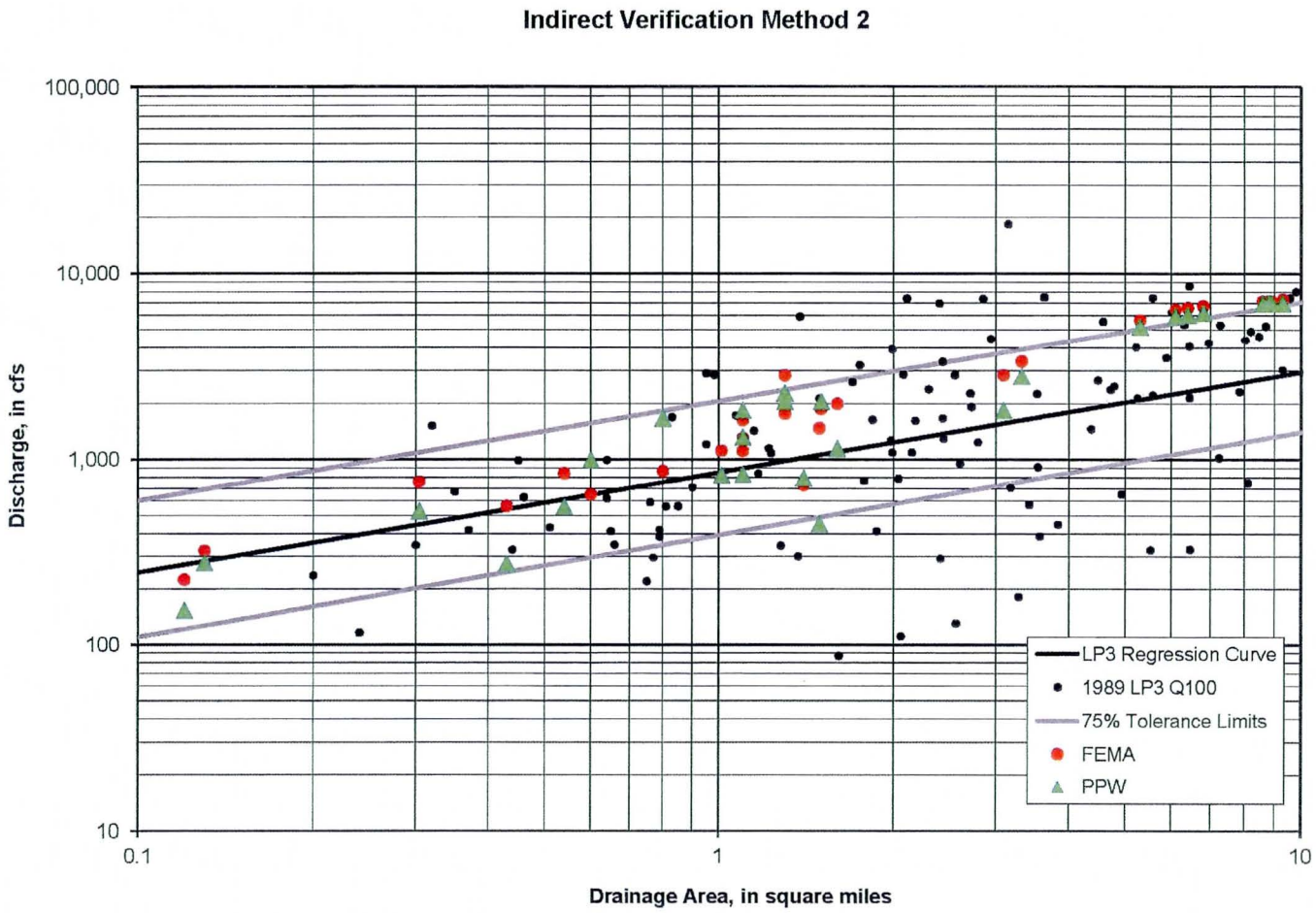

Figure 29. Indirect Verification Method 2

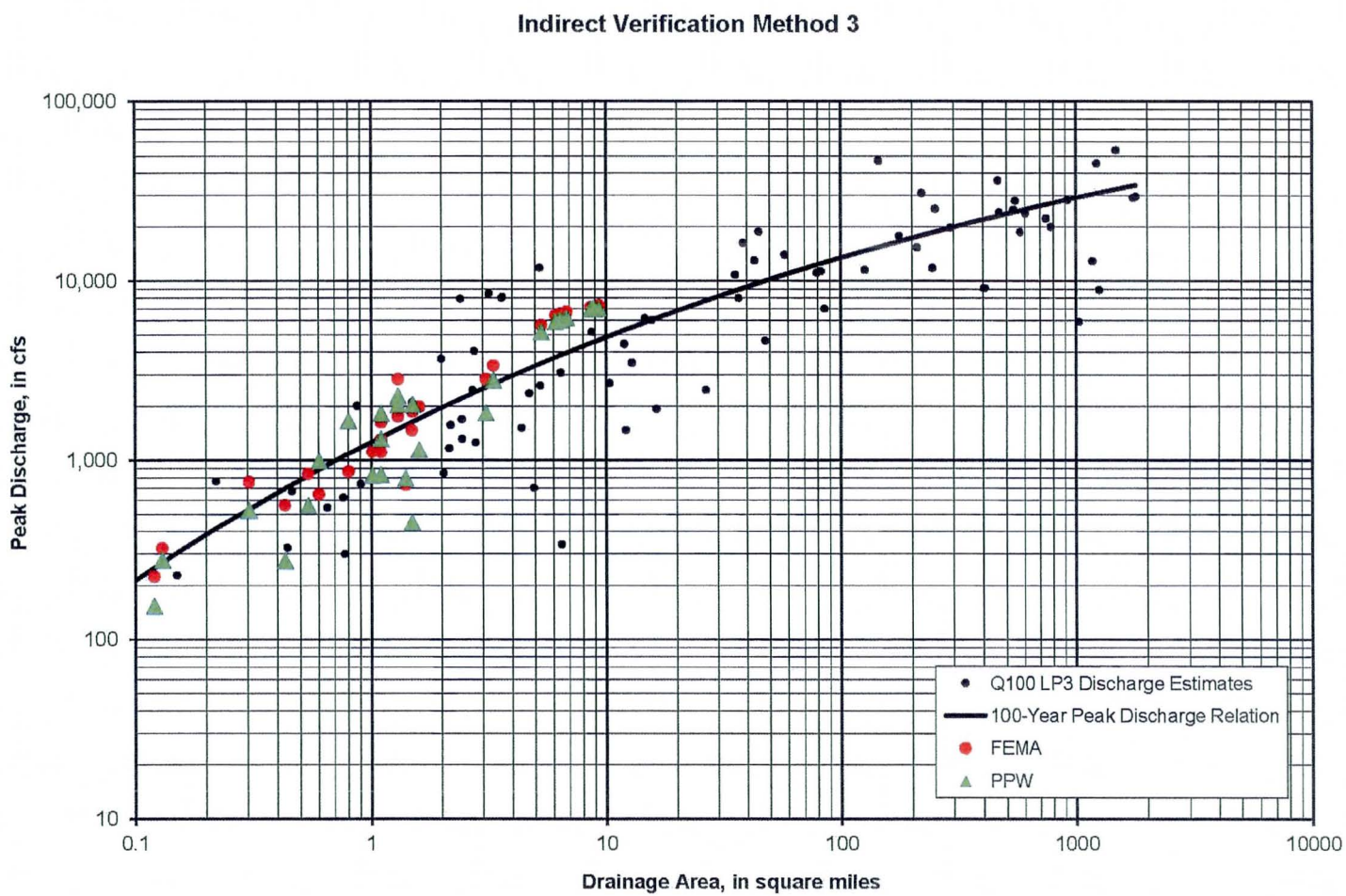

Figure 30. Indirect Verification Method 3

4.11.4. Gage Data and recorded Storm Event Verification

As noted in Section 4.9, there are two stream flow gages in the study area, one on Stagecoach Pass Wash (ID 4913) and another on Rawhide Wash (ID 4863). Both gages are maintained by the District; the Stagecoach Pass Wash gage was installed in June of 2001 and the Rawhide Wash gage was installed July 1999. As part of the District's Alert data, Flood Flow Frequency analysis are performed on each gage based on a Bulletin 17B analysis of the recorded data. A summary of the gage flood frequency and maximum recorded event is in Table 26.

Table 26. FCDMC Stream Gage Data Summary

Gage Name	ID	Period of record	100-Year Flood Flow Frequency Peak Discharge (cfs)	Maximum Recorded Flow (cfs)	Date of Maximum Recorded Flow	Nearest PPW Floodplain Cross-Section/Peak Discharge(cfs)
Stagecoach Wash	4913	06/13/2001-Current	564	802	07/31/2007	LT_137/837 cfs
Rawhide Wash	4863	07/27/1999-Current	6,340	446	09/09/2006	UR_014/6,949 cfs

In July of 2007, Stagecoach Pass Wash received a large rainfall event; the Stagecoach Wash rain gage (ID 4910) recorded nearly 2.7 inches of rain in about 2 hours. An FCDMC storm report was prepared after the event and, per the report, the computed peak discharge from this event was 802 cfs (Stagecoach Wash Storm Report, 2007). This 802 cfs is the maximum recorded flow for the gage. The NOAA Atlas 14 Point precipitation data at the Stagecoach Wash gage indicates that a 2-hour rainfall depth of 2.7 inches is roughly a 100-year rainfall depth, this is noted in the FCDMC storm report that the storm was near a 100-year event. Although the PPW models are based on a 100-year 24-hour event, the PPW discharge of 837 cfs is very close to the near 100-year event recorded at the gage.

The Rawhide Wash gage, while not recording a large rainfall event like the Stagecoach Wash gage, has a flood flow frequency estimate of 6,340 cfs, very close to the PPW results of 6,949 cfs.

5. HYDRAULICS

5.1. Method Description

The hydraulic analyses for the PPW ADMS were completed through the use of the FLO-2D software. The watershed is comprised of generally shallow distributary flow with significant drainage splits and is most appropriately modeled with a two-dimensional software package. The computation of the flood wave movement across the watershed is completed during the same simulation as the hydrologic modeling computations. For this reason, a separate hydraulic analysis is not required for this study. The typical parameters associated with a hydraulic model are included with the hydrologic model under this two-dimensional modeling scenario. Refer also to Section 4 for the Hydrologic method description and parameters.

5.2. Work Study Maps

Traditional detailed work study maps at a scale of 1-inch=200-feet are impractical given the large size of the study area and the nature of the hydraulic data output. Therefore, pertinent hydraulic output is displayed on 1-in=1,200-feet exhibit maps in the appendix. This output includes grid based data for the maximum flow depth, maximum peak discharge, and maximum velocity for each of the three wall scenarios for the 100-year model results.

5.3. Parameter Estimation

The primary parameters associated with the two-dimensional hydraulic analysis are the Manning's roughness coefficient (n-value) associated with each surface feature characterization, obstruction to flow (buildings and property walls), and hydraulic structures (channels, culverts, bridges, and wall openings).

5.3.1. Roughness Coefficients

The Manning's n-value assigned to each grid element was based off of surface feature characterization and associated roughness estimates (See Section 4.2.6.2). The roughness value assigned to each surface feature characterization was based on default values provided by the District and revised by JEF through the use of aerial photography, topography, and field reconnaissance. The n-value for each grid element was computed within ArcGIS V10.1 and is area-weighted based on the coverage of each surface feature characterization over any specific grid. The surface feature characterization and associated n-values are listed in Table 14 in Section 4.2.6.2.

The "Unpaved Disturbed Ground" classification accounts for areas of the watershed that have been rough graded or undergone preliminary grading for future development. This study accounts for the existing conditions only and did not anticipate the development of the rough graded areas for future conditions. The rough graded areas are similar to a dirt road where the soil may be bare natural ground but has undergone compaction from construction equipment or vehicular traffic. The ground is generally smooth and flat with little to no vegetation.

As discussed previously, the surface feature characterization delineations were provided by the District based on mapping products provided by the photogrammetrist. A single characterization was provided for undeveloped desert areas that covered the entire study area. It was noted that the surface roughness in the undeveloped desert in the upper watershed was significantly different than the desert in the lower watershed. The upper watershed has more surface irregularities, rock outcrops, and denser vegetation than the lower watershed. Therefore, the undeveloped desert polygons were 'split' or divided along a location that's indicative of a slope break and/or vegetation break as shown in Figure 31. This line was compared to a flooding context map that was prepared previously and is similar to the break from tributary to distributary flow. Furthermore, the line was compared to general vegetation mapping and generally corresponds to a break from Cacti to Bursage plant communities. The

lower desert will be given a Manning's n value of 0.040 and the upper desert will be given a Manning's n value of 0.055.

The Manning's n associated with building structures was assigned to accomplish two objectives. The first is to account for the roughness elements typical to residential homes such as landscaping, barbeques, and other appurtenances including contraction and expansion losses from contact with the building itself as a watercourse at flood stage enters the lot. This value would be relatively high such as 0.065. The second is to account for the smooth surface of a roof system as rain drains off the roof. This value would be on the low side such as 0.020. Therefore, the Manning's n value assigned to buildings of 0.035 is a compromise.

The roughness coefficients specified for FLO-2D are based on the surface feature characterization roughness values and also through the use of the shallow-n function. The shallow-n (SHALLOWN) is a roughness value that is specified for extremely shallow flow in the model control file CONT.DAT. As the flow depth increases on floodplain grid elements, the n-value adjusts to the land use roughness value (nbase) in the FPLAIN.DAT file using the following criteria:

- Flow depth<0.2 ft. — n= SHALLOWN
- 0.2 ft<flow depth<0.5 ft — n=1/2 * SHALLOWN
- 0.5 ft<flow depth<3 ft — $n=nbase*1.5*e^{-(0.4depth/dmax)}$
- 3 ft<flow depth — n=nbase

There are several surface characterizations that have base n-values greater than 0.05 (1/2*SHALLOWN), Urban High Vegetation (n=0.060), Upper Hillslopes (n=0.060), and Upper Undeveloped Desert (n=0.055). As noted in the bullet points above, at flow depths between 0.2 and 0.5 ft the n-value will be 1/2*SHALLOWN or 0.05. The base n-values will come into the equation above flow depths of 0.5 foot. While a higher SHALLOWN value could be used to keep the n-values above the base values for all flow depths greater than 0.2 foot, a higher value was not selected since the SHALLOWN of 0.1 is reasonable given the watershed surface feature characterizations. Furthermore, a higher SHALLOWN will keep n-values higher across the full watershed for all surface feature characterizations. The SHALLOWN was kept on the lower end of the typical range of values so that smoother surface feature characterizations (Concrete, Asphalt, Buildings, Lower Undeveloped Desert, etc.) would not be modeled with a higher n-value than necessary for the lower flow depths.

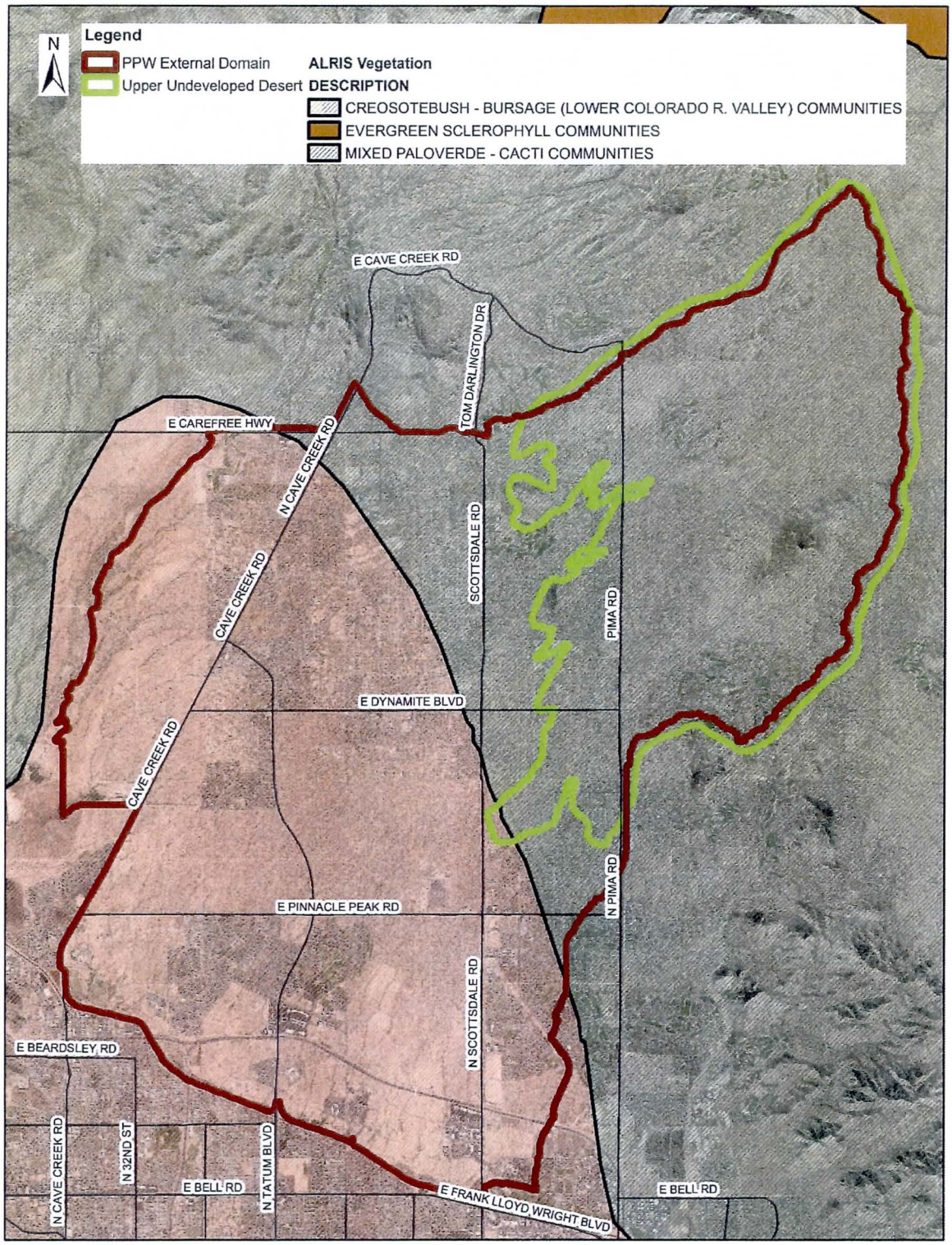

Figure 31. Upper and Lower Undeveloped Desert Roughness Divide

5.3.2. Grid Element Roughness Adjustments

The Manning's n-values were manually adjusted at various locations to prevent model surging and instabilities. These locations generally include 1-D channel elements that have excessively high flow velocities (greater than 12 ft/sec when the channel bed does not support such high velocities) and large areas of deep ponding depths. Table 27 lists the n-value changes in flood plain elements based on the depth of ponding.

Table 27. Assigned n-values Based on Depth of Ponding.

Ponded Depth* (ft)	Assigned n-Value
5 to 8	0.08
8 to 10	0.10
10 to 15	0.20
15 to 20	0.30
>20	0.35

*The lowest increase in n-value was used to prevent model instabilities, generally this was 0.1 even if flow was greater than 10-feet but in several locations the n-value had to be adjusted higher than 0.1, primarily in the R11 model.

The adjustments were tracked spatially with a GIS polygon shapefile. This shapefile is provided with the digital data.

5.3.3. Expansion and Contraction Coefficients

There are no expansion and contraction coefficients to specify in FLO-2D and are therefore, not a part of this study. However, n-value assignment for various surface feature characterizations and at certain hydraulic structures may have been increased to account for potential losses due to expansion and contraction.

5.3.4. Grid Element Width and Area Reduction Factor (ARF.DAT)

An area reduction factor (ARF) was applied to each grid element that had some percentage of area covered by a building structure. The factor reduces the area of a grid cell available for flood storage. ARF values were computed using the building features that were captured by the aerial mapping data sets (See Section 3.3). The building features were extracted out of the mapping data and intersected with the FLO-2D grids using ArcGIS V10.1 to compute the area of blockage. The maximum ARF value is 1.0, which is a completely blocked grid due to complete coverage of a building structure. Since the PPW study has ARF values set in between 0.95 to 1.0, the IRAINBUILDING feature in the RAIN.DAT files (See Section 4.2.5) is switched on (set to 1.0). This will allow the rainfall to run off of that grid to the downstream grid as if the building is totally impervious and no floodplain storage will occur.

Width Area Reduction factors (WRF) were not applied in the model, all WRF fields are coded as zero in the ARF.DAT input file. Large areas of obstruction were blocked out by the ARF value and the LEVEE.DAT file was used where property walls impacted the flow patterns.

5.4. Cross Section Description

Hydraulic cross-sections are not a part of two-dimensional modeling and are not a part of this study. The floodplain cross-sections (FPXSEC.DAT) discussed in Section 4.4 should not be confused with hydraulic cross-sections. Floodplain cross-sections are only used to extract maximum discharge and hydrograph data out of the FLO-2D model; they have no impact on the hydraulic computations.

5.5. Modeling Considerations

5.5.1. Hydraulic Jump and Drop Analysis

In a FLO-2D model there are no direct methods to deal with the modeling of jumps and drops other than to adjust the limiting Froude number and/or the Manning's n-values of the grid. As noted in Section 4.5, the majority of the watershed was modeled assuming sub-critical flow (i.e. a limiting Froude number of 1.0). The only deviation to this criteria is on Rawhide Wash at the Pinnacle Peak Road Bridge where there is a large concrete drop structure. This location was modeled to allow the super-critical flow regime with a limiting Froude number of 2.0.

5.5.2. Bridges and Culverts (HYSTRUC.DAT)

Hydraulic structures accounted for in the FLO-2D models include culverts and wall openings. Based on the detailed mapping, there are approximately 4,000 culverts and an unknown number of wall openings within the study area. This number represents all culverts regardless of size and significance from a drainage perspective. Existing information documenting the location and physical dimensions of all 4,000 potential culverts and wall openings is limited, unavailable or non-existent. Through the use of existing, low resolution FLO-2D modeling combined with interpretation of the terrain and aerial photography a total of 1,182 structures, 1,049 of which represent culverts and 133 represent wall openings, were selected for inclusion in the model. The locations of the modeled structures is shown in Figure 32. The number of structures per model area is listed in Table 28.

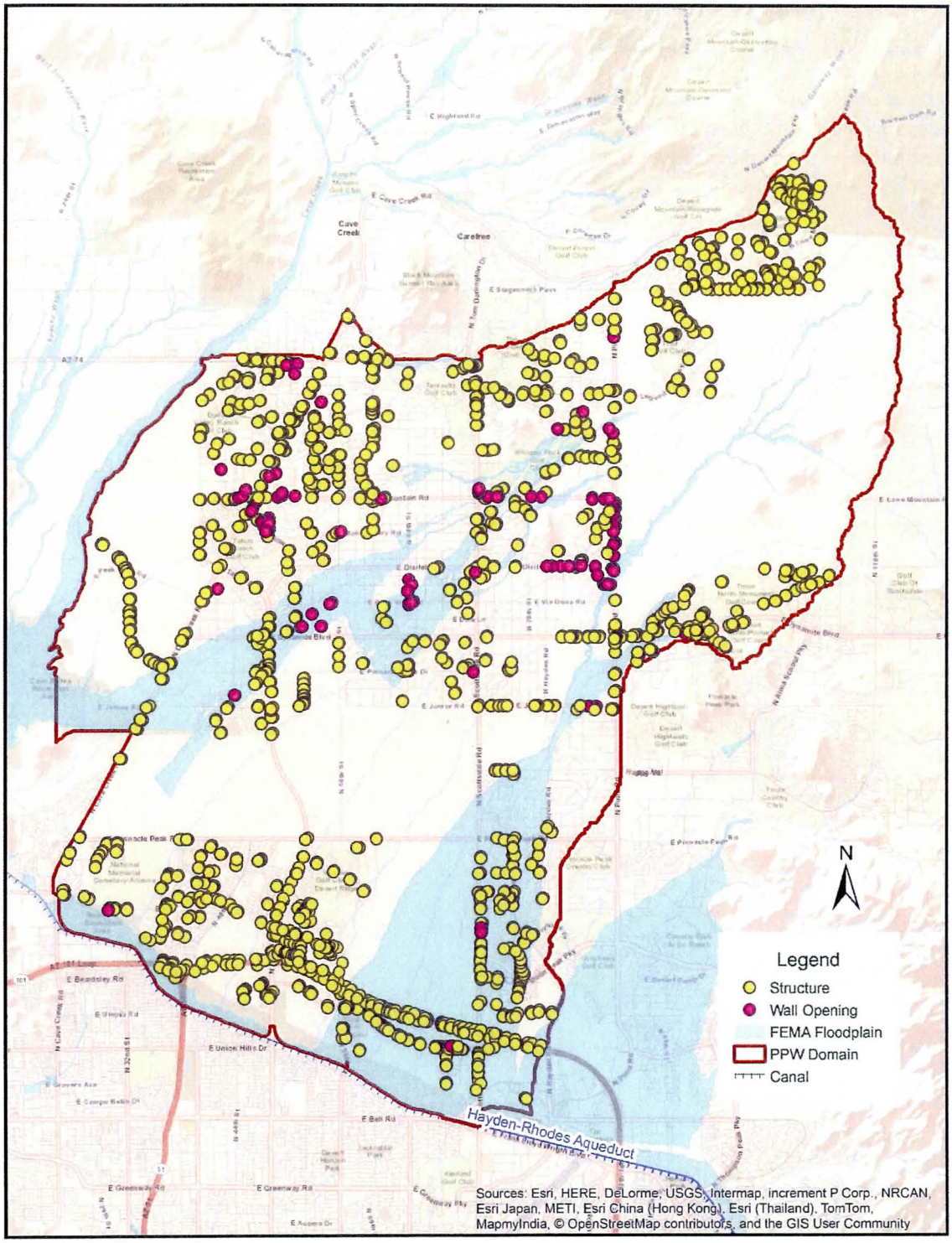

Figure 32. Model Hydraulic Structure Locations

Table 28. Number of Hydraulic Structures by Model Area

Model Area	Culverts	Wall Openings
Legend Trail	160	12
Upper Rawhide Wash	93	31
Whisper Rock	204	28
Tatum Ranch	226	45
Lower Rawhide Wash	190	6
Cave Buttes	92	10
Desert Ridge	217	1
Reach-11	15*	1*

*Due to R-11 overlapping with DR and LR, the structures in the R-11 model are duplicates of the structures that are located in the DR and LR overlapping area.

Physical dimensions of the culverts and wall openings are obtained from available as-built plans, the City of Phoenix GIS database and/or field measurements. Physical dimensions taken from as-built plans are generally limited to those structures within the ADOT right-of-way. A summary of the physical dimensions for each culvert is provided in Appendix C. This data is also provided digitally in shape file format referenced to a point feature taken at each culvert inlet and outlet as well as each wall opening.

5.5.2.1. Invert Elevation Determination

Invert elevations for each structure are estimated from the detailed topographic mapping using one of two methods:

For structures with a headwall:

- A measurement from the top of the headwall to the bottom invert of the structure was taken during the field inventory
- In ArcMAP using 3D elevation breaklines provided by the FCDMC, a breakline representing the headwall was found and a maximum elevation of the headwall was recorded.
- The invert of the structure was then calculated by taking the recorded maximum elevation of the headwall and subtracting the field measured distance from the top of the headwall to the bottom invert of the structure.

For structures without a headwall:

- Using a hand-level and a Philadelphia rod, a measurement from the top of the defined location (i.e. top of road, top of structure) to the bottom invert of the structure was taken during the field inventory. A GPS point was taken at the location of the measurement.
- In ArcMAP using 3D elevation breaklines provided by the FCDMC, a breakline representing the defined location was found and an elevation of that location was recorded.
- The invert of the structure was then calculated by taking the recorded elevation of the location and subtracting the field measured distance from the top of the location to the bottom invert of the structure.

In circumstances where the above procedure caused an adverse or zero slope for the structure, the wash bed slope in the vicinity of the structure was found. The calculated slope would then be assumed to be the slope of the hydraulic structure and by keeping the calculated inlet invert the same, used to determine the invert of the outlet.

5.5.2.2. Rating Curve Development

In FLO-2D, hydraulic structures are simulated with a stage-discharge relation. The stage-discharge relation specified for all hydraulic structures is in the form of a rating table. The rating tables for each structure are coded in the HYSTRUCT.DAT input file. Also included in this input file are the following parameters and settings:

- Unique identifier for each structure
- Inlet and outlet grid cells that the rating curve is associated with
- Type of grid cell the rating curve is assigned to
  - 0 – Floodplain
  - 1 – One-dimensional channel
  - 2 – Floodplain element for the inlet and channel element for the outlet
- **Reference elevation (HEADREFEL)** – set to 0 for all structures (The water surface elevation is based on the floodplain grid elevation)
- **Tailwater option (INOUTCONT)** – set to 0 for all structures (This assumes inlet control based on headwater depth)
- **Length (CLENGTH)** – set to 0 for all structures (The culvert length is not required since the long culvert routine option is not used)
- **Diameter (CDIAMETER)** – set to 0 for all structures (The culvert diameter is not required since the long culvert routine option is not used)

Rating tables for the culverts within the study area were calculated using three different versions of HY-8 (version 6.1, 7.1 and 7.3). Originally, over 800 culverts were selected to be modeled for work assignment 1. Due to the large quantity, version 6.1 was chosen because this version allowed an automated script to calculate rating curves in a batch process. However, it was determined that version 6.1 of HY-8 did not adequately calculate rating curves for culverts with an elliptical opening. Therefore version 7.1 was used for elliptical culverts. After work assignment 1, it was determined that over 100 additional culverts were required for work assignment 3. During the time between work assignment 1 and 3 version 7.3 of HY-8 was released. It was determined that this version would be the easiest way to calculate the additional rating curves. Calculation steps used to derive the rating tables are as follows.

Using HY-8 version 6.1, rating tables for culverts were calculated using a batch process. The batch process uses the HY-8 program as the calculation engine. The process is summarized in the following steps along with an illustrative example. HY-8 input and output files are provided on the digital data disk.

- A maximum discharge is calculated for the structure based on the physical dimensions and an assumed maximum overtopping depth above the top of headwall elevation of two feet.
- Based on the maximum discharge an incremental discharge is selected. The incremental discharges range from 2 cfs to 100 cfs.
- A HY-8 input file is generated with the physical dimensions and a discharge range from 0 cfs to 10 times the discharge increment. Generic tailwater and roadway overtopping data is also included in the input file. For the tailwater, a constant elevation is specified that is one foot above the outlet invert elevation. For the roadway data, level roadway surface is specified with a crest length of 1,000 feet, width of 24 feet and elevation set to 2 feet above the top of headwall.
- The data file is executed in an inlet control mode only and the output is saved. The output file name is the structure id along with the discharge iteration number.
- The HY-8 input file is recreated as many times as necessary at the selected discharge increment until the maximum discharge is reached.
- The inlet control results for each discharge iteration are read and the total rating curve for the entire range in discharge is assembled in the format required by FLO-2D.

Example:

Hydraulic structure S105-06 is a 2-barrel 8' x 4' box culvert 78.12 feet in length located in the Legend Trails subarea. The maximum discharge calculated for this structure is 1,000 cfs. The discharge increment selected is 20 cfs and the range for each iteration is 200 cfs. This results in a total number of 5 iterations. The file naming for this structure is S105-06-01, S105-06-02, etc.

Below summarizes the process used for calculated rating curves using HY-8 version 7.1 and 7.3

- A maximum discharge is determined for the structure based upon an iterative process of running the HY-8 program and checking the results to an assumed overtopping depth above the top of headwall elevation.
- Based on the maximum discharge an incremental discharge is selected. The incremental discharges range from 2 cfs to 320 cfs.
- A HY-8 input file is generated with the physical dimensions and a generated discharge range. Generic tailwater and roadway overtopping data is also included in the input file. For the tailwater, a constant elevation is specified that is five feet above the outlet invert elevation. For the roadway data, level roadway surface is specified with a crest length of 1,000 feet, width of 24 feet and elevation set to an elevation several feet above the top of headwall.
- The data file is executed in an inlet control mode only and the output is exported. The output file name is the structure id along with the discharge iteration number.
- The inlet control results for each discharge iteration are read and the total rating curve for the entire range in discharge is assembled in the format required by FLO-2D.

Rating curves for the wall openings are generated using a hybrid approach. The approach combines normal depth calculations for depths less than the opening height and a sluice gate approach for depths greater than the opening height. Calculation worksheets for each wall opening are provided in the Appendix. For the normal depth calculations, bed slope and a Manning's n-value is required in addition to the opening dimensions. Bed slope is estimated from the detailed mapping looking at an average reach both upstream and downstream of the wall. The Manning's n-value selected is the typical value for the wash bottom in the vicinity of the wall opening and is taken directly from the FPLAIN.DAT file. For the calculations, the Manning's n-value is varied with depth according to the same process described in Section 5.3.2. For the sluice gate calculations the only additional data required beyond the opening dimensions is a contraction coefficient. The contraction coefficient selected for all wall openings is 0.61. However, this coefficient is also used as an adjustment parameter to provide a smooth transition in the calculations between the normal depth and sluice gate conditions.

5.5.2.3. Culvert Clogging Factor Development

For all hydraulic structures within the study area, a clogging factor was used to simulate blockage within the structure. For all culverts, clogging factors were determined using a batch process and a standard 50% clogging factor was used for all wall openings. The batch process is summarized in the following steps:

- Using field surveyed data, determine which entrance type the culvert was built with. Entrance types are used as a relative indicator of clogging potential. For example, a culvert that has a wing walled entrance will likely have lower clogging potential than a projected culvert.
    - Projected or drop entrance
    - Mitered entrance

        - Straight entrance
        - Entrance with a wing wall
- Calculate the deposition ratio
        - Field survey data was measured in increments of 6, i.e. 0-6. 6-12, etc. To be conservative, use the higher of the two values and divide that number by the rise or diameter of the culvert. For example, a 24" culvert with 0-6 inch deposition will have a deposition ratio of 0.25 (6"/24").
- Group the culvert into categories based upon entrance type and the deposition ratio.
- Assign a clogging factor for each group based on the rise or diameter of the culvert per the following break points:
    - Projected or drop entrance
        - Deposition Ratio 0.0-0.2: 20% for culverts 36" and greater. 35% for less than 36"
        - Deposition Ratio 0.21-1.0: 35% for culverts 36" and greater. 50% for less than 36"
    - Mitered entrance
        - Deposition Ratio 0.0-0.5: 20% for culverts 36" and greater. 35% for less than 36"
        - Deposition Ratio 0.51-1.0: 35% for culverts 36" and greater. 50% for less than 36"
    - Straight entrance
        - Deposition Ratio 0.0-0.2: 10% for culverts 36" and greater. 25% for less than 36"
        - Deposition Ratio 0.21-0.8: 20% for culverts 36" and greater. 35% for less than 36"
        - Deposition Ratio 0.81-1.0: 35% for culverts 36" and greater. 50% for less than 36"
    - Entrance with wing wall
        - Deposition Ratio 0.0-0.4: 10% for culverts 36" and greater. 25% for less than 36"
        - Deposition Ratio 0.41-1.0: 35% for culverts 36" and greater. 50% for less than 36"
- Apply clogging factor to current rating table discharges.

There are a number of structures in the models that have a total structure width greater than the cell size. For these structures, the calculated discharge at every depth in the rating table was divided by the number of cells the structure spans and a duplicate structure was created with unique inlet and outlet grid cells. The identifiers assigned to each of the multiple ratings at these locations are appended with an alphabetic character. For example, the culvert S104-05 is a 4-barrel 10' x 4' box culvert. The total width of the opening is 40 feet and therefore spans two grid cells. A duplicate culvert is created and the identifiers are renamed S104-05A and S104-05B. Rating tables for each copy are generated using the same physical data with the reduction in discharge applied only in the HYSTRUCT.DAT file.

5.5.2.4. Rating Revisions

Preliminary model runs were performed using the structure rating curves with clogging factors. Elevation adjustments were made for the structure inlet and outlet grids based on the estimated invert elevations as discussed in Section 5.5.2.1. The purpose of these simulations was 1) to identify locations where FLO-2D was adjusting rating curves to improve model stability and 2) to revise those locations as necessary to improve numerical stability. The hydraulic structures are being modeled with the INOUTCONT parameter equal to 0 in the HYSTRUC.DAT file. By setting this parameter to zero, FLO-2D assumes that flow always goes downstream and is not allowed to back flow into the structure. In some cases, the outlet is in a sump or has some other adverse tailwater conditions that results in submergence of the outlet during at least part of the hydrograph. The increase in tailwater can affect the discharge computation and when using the depth/discharge rating curve specified in the HYSTRUC.DAT file. In order to stabilize the structure, FLO-2D will adjust the rating curve discharge to maintain a positive water surface slope through the structure; these rating curve revisions are reported in the REVISED_RATING_TABLES.OUT file. After completion of the preliminary model runs, the REVISED_RATING_TABLES.OUT file was reviewed and each structure reported in the output file was investigated for potential adjustment. The adjustments were completed by three possible solutions as outlined below:

- **Evaluate and revise downstream grid element elevations** – if the structure outlet is in an "artificial" sump condition but topographically should not be, the grid elements downstream of the structure outlet were lowered to allow for a positive outfall. The "artificial" outlet sump condition can occur as a result of an elevation disparity between the point specific invert elevations of the structure compared to surrounding grid elements whose elevations were assigned based upon an averaged elevation in the 20-foot grid.
- **Blend original HYSTRUC.DAT rating curve with the revised rating curve from the REVISED_RATING_TABLES.OUT file** – If the structure outlet has a positive outfall (it is not in a sump condition) and the rating curve is being revised near the hydrograph peak, the original rating curve was blended with the revised rating curve to provide a smoother transition between the two curves. This blended rating curve was then pasted back into the HYSTRUC.DAT file to replace the original rating. Figure 33 illustrates graphically the original rating curve, the revised segment from the REVISED_RATING_TABLES.OUT file and the blended (modified) rating curve.
- **Make no revision** – If the structure outlet has a positive outfall (it is not in a sump condition), and the rating is being revised but the revision is *not* affecting the peak (i.e. the revision is occurring at depths/discharges well below the structure hydrograph peak discharge), and the structure hydrograph shape is smooth and considered reasonable, then no revisions were made and FLO-2D was allowed to continue to adjust the rating. There are also cases where the outlet is realistically in a sump condition, those locations were not adjusted and FLO-2D was allowed to continue revising the rating. Realistic sump or tailwater conditions may include outlets into retention basins or channels with deep flow depths.

The rating curve adjustment process was completed in a series of iterative model simulations for each sub-area. In each simulation, the REVISED_RATING_TABLES.OUT file was reviewed and adjustments made until the revisions to structures listed in the revision file were either considered to be real or not affecting the structure hydrograph peak. A summary of revisions for each iteration for each sub area is included in Appendix E.

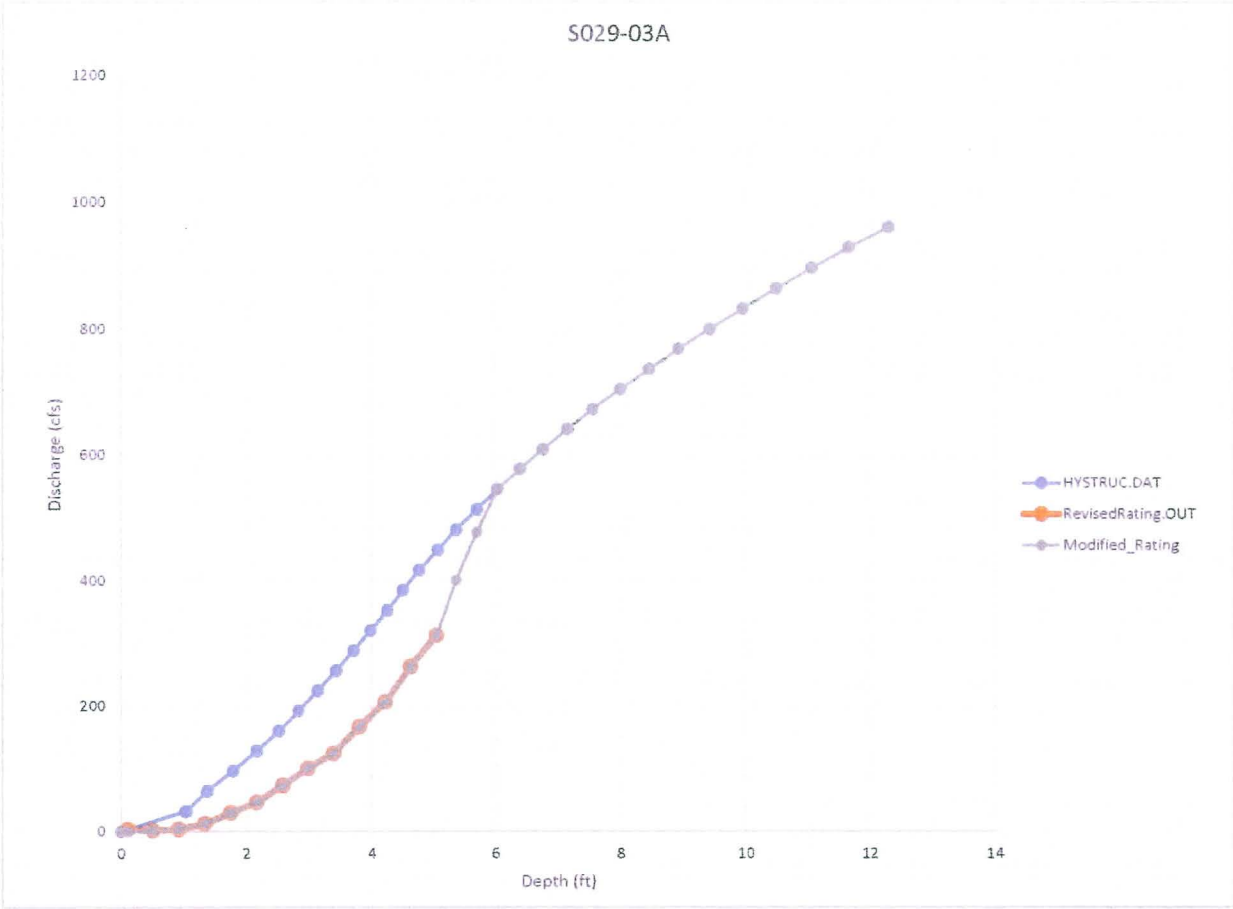

Figure 33. Blended Rating Curve

5.5.2.5. Severe Clogging Adjustments

The rating curves in the HYSTRUC.DAT files are developed using the clogging factor criteria as discussed above in Section 5.5.2.3. However, several structures as noted in the structures database, see Appendix E, or noted through a review of the structure photographs, are highly clogged or have high deposition. Generally, it is accepted that some sediment will be "cleaned out" during high flows, however, the culverts listed in Table 29 below are considered to be severely clogged and will not be clear of debris/sediment under a high flow. Therefore, an additional clogging factor was applied to the initial clogged rating curve. The additional clogging factor was determined through a visual inspection of the inlet/outlet photographs and the structure opening size. The structure data and additional clogging factor are listed below in Table 29. The clogging factor listed in the table is a percent of blockage, i.e. 0.75 indicates 75% blockage so the rating curve is reduced by 75%. The structures listed at 1.00 are adjusted to allow very little flow through the structure. FLO-2D requires discharge to increase with each depth/discharge ordinate so the rating curve discharges are increased by 0.01 for each depth increment in the initial rating curve table.

Table 29. Additional Clogging Factor Applied to Severely Clogged Culverts.

Culvert ID	Sub-Area	No. Barrels	Shape	Diameter (in)	Span (ft)	Rise (ft)	Material	Additional Clogging Factor
S052-03	DR	1	Circular	24	-	-	Concrete	1.00
S005-04	DR	6	Circular	36	-	-	Concrete	1.00
S_PW_DR_4	DR	2	Box	-	6.00	4.00	Concrete	1.00
S92-03	UR	2	Circular	18	-	-	Concrete	0.75
S018-01	TR	2	Box	-	8.00	4.00	Concrete	0.60
S018-02	TR	2	Box	-	10.00	4.00	Concrete	0.75
S018-03	TR	3	Box	-	10.00	4.00	Concrete	0.75
S100-36	UR	2	Circular	24	-	-	Smooth HDPE	0.50
S100-07.1	UR	1	Circular	24	-	-	Corrugated Metal	0.50
S100-11	UR	2	Box	-	10.00	4.00	Concrete	0.50
S056-02	TR	4	Circular	24	-	-	Corrugated Metal	1.00
S056-01	TR	3	Box	-	8.00	3.00	Concrete	0.60
S086-09	WR	2	Box	-	10.00	3.00	Concrete	0.60
S046-10	TR	3	Box	-	6.00	4.00	Concrete	0.60
S060-01	TR	4	Circular	48	-	-	Corrugated Metal	0.75
S088-08	WR	6	Box	-	10.00	3.00	Concrete	0.60
S060-04	TR	3	Box	-	8.00	3.00	Concrete	0.60
S060-06	TR	4	Box	-	10.00	4.00	Concrete	0.60
S105-5	LT	2	Circular	36	-	-	Concrete	0.60

5.5.2.6. Storm Drain Analysis

Based on the City of Phoenix GIS Database, there are approximately 1,000 storm drain inlets within the study area. This number represents inlets associated in the database with features classified as storm drains regardless of size and significance from a drainage perspective. After initial FLO-2D runs were completed for the study area, it was determined that the storm drain modeling would focus on areas that were experiencing significant flooding in urban areas where a storm drain system was present. For this study, only storm drains greater than 18" were considered for modeling. Through the described process, a total of 122 storm drain inlets were selected for inclusion in the model. There are three basic configurations of storm drains associated with the selected inlets:

- Street drainage
- Culverts with bends
- Major storm drain system

The location of each of the 122 storm inlet is shown in Figure 39. The number of structures per model area is listed in Table 30.

Table 30. Number of Storm Drains by Model Area

Model Area	Storm Drain Inlets	Miles of Storm Drains Model
Desert Ridge (DR)	112	9.3
Lower Rawhide (LR)	4	0.2
Tatum Ranch (TR)	6	0.2

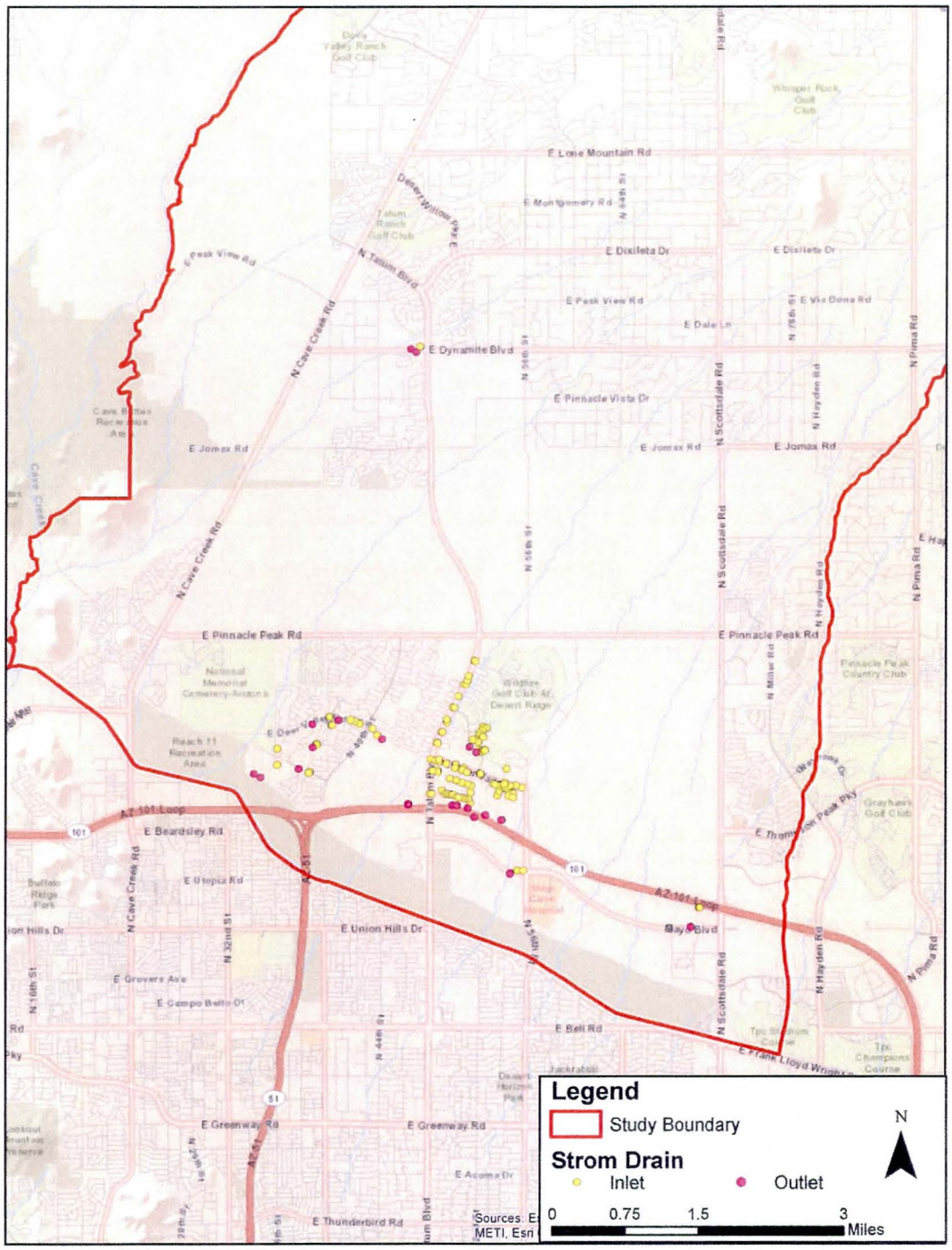

Figure 34 Model Storm Drains Locations

Storm drains were simulated using stage-discharge relations coded into the HYSTRUC.DAT file. The stage-discharge relation specified for all hydraulic structures is in the form of a rating table. Rating tables were developed using an iterative process involving multiple programs, summarized as follows. The physical dimensions of the storm inlets (structure type, material, size) were obtained from the City of Phoenix GIS Database, field measurements and/or Maricopa Association of Governments' (MAG) Uniform Standard Specifications and Details for Public Works Construction. The process is summarized in the following steps.

1. Rating tables are into two categories depending upon the inlet type.
    a. Grate or cub inlets: Rating tables for this category were generated using a hybrid approach. Inlet types were separated into four types found in the Maricopa Association of Governments' Uniform Standard Specifications and Details for Public Works Construction: Type "A", "C", "D", and "L" catch basin. Using the dimensions specified in the MAG, a general rating curve was developed for each inlet type by combing normal depth or weir flow depending on inlet type, and orifice flow calculations. Calculation worksheets for each wall opening are provided in the Appendix. For the normal depth calculations, the Manning's n-value is varied with depth according to the same process described in Section 5.3.1.
    b. Headwall inlets: Rating tables for storm drain inlets set in a headwall at the invert of a channel or natural wash were generated using HY-8 v7.3 and the inlet dimensions given in the City of Phoenix GIS database assuming inlet controlled conditions. For these calculations, the rating curves were developed according to the same process described in Section 5.5.2.
2. The rating tables were input into the HYSTRUC.DAT file in FLO-2D and then simulated for the entire storm duration.
3. Using the FLO-2D results, a maximum discharge at each inlet was recorded and used as an inflow for each inlet in a StormNET v4.21.0 model developed to simulate the storm drain conditions. Invert elevations for the inlets were taken from the City of Phoenix GIS database where available. If elevation data was not available, field measurements were taken at the inlet.
4. If the inflows did not cause a surcharge in the storm drain system, it was assumed that the storm drain was inlet controlled and the rating table within the HYSTRUC.DAT file was not altered.
5. If the inflows did cause a surcharge in the storm drain system, an iterative process of running StormNET to discover the maximum flow that can enter the storm drain inlet without causing a surcharge was found. The rating table within HYSTRUC.DAT was then altered to not allow the inlet to produce a discharge larger than the calculated maximum flow.

Elevations of the grids where a grate and curb inlet is present was not altered in the FLO-2D model. For culverts modeled through the storm drain process, the grid elevation was altered within FLO-2D to match the invert of given the City of Phoenix GIS database.

5.5.3. Property Walls

The FLO-2D models were simulated under three scenarios related to the property walls as outlined below:

- **Without-Walls (WOW):** Models were simulated without any property walls in the model, the LEVEE.DAT file was not used.

- **With-Walls; No-Failure (WW_NF):** Models were simulated with the property walls in place. The walls were not failed regardless of the depth of flow against them, the walls were allowed to overtop if flow was higher than the top of wall elevation.
- **With-Walls; Failure (WW_F):** Models were simulated with the property walls in place. The walls were failed when there was two feet of flow depth against them.

A discussion of the development of the LEVEE.DAT file used to model the walls is in the following sections.

5.5.3.1. LEVEE.DAT Development

Property walls significantly influencing runoff in the PPW watershed were modeled in FLO-2D using the LEVEE.DAT input file. Although most of the walls within the PPW study area are not flood walls nor considered levees, the LEVEE.DAT data file within FLO-2D offers the most flexibility in modeling the hydraulic effects of property walls on flood distribution. Wall locations and heights were generally derived from 3D lines provided for each mapping set by the District, see Section 3 for the mapping discussion and data set area limits. Within the 1309 mapping area, the 3D wall linework provided by the District was poorly defined and not very accurate. However, this area was generally covered by the PPS FLO-2D modeling area that overlaps the PPW Areas LR and UR. JEF obtained a shapefile that was used to define the LEVEE.DAT file for the PPS FLO-2D model. JEF then used this file to model the same walls modeled in the PPS FLO-2D model and generally maintained the same approach used in the PPS study for these walls. Since the walls were not covered by the 3D lines provided by the District, the walls did not contain elevation data. The wall vertex points were sampled on the Terrain Data Set (TDS) and a height of 6-feet was added to the TDS sampled elevation to obtain an estimated top-of-wall elevation. These walls are shown on Figure 35.

The complete file of 3D lines received from the District for mapping data sets 1310 and 1311 is extremely detailed in the coverage of walls and includes walls that are insignificant to the hydraulic routing of flow. The walls shown in Figure 36 are the complete set of 3D lines received from the District and include items such as multiple discontinuous minor landscaping and/or backyard walls. The walls that are considered minor, insignificant, and non-structural (such as the minor landscaping walls around homes) and walls in between homes in dense subdivisions, were removed from the dataset and consequently, not included in the model. Furthermore, walls that, although may not be minor from a structural point of view but did not significantly impact flow patterns were also excluded from the model. The walls left in the dataset for inclusion in the models were those that were structurally significant, such as perimeter walls around a subdivision, and had an impact on flow distribution.

Flow results from initial modeling efforts were examined to determine which walls might have a significant hydrologic or hydraulic effect, such as diverting a significant amount of flow or having the possibility to alter flow paths. In general, large lot residential walls were not included in the model. An example of modeled versus non-modeled walls are shown on Figure 37.

The 3D wall lines to be included in the models were converted to .xyz files in GIS and imported into the FLO-2D Grid Developer System software (GDS). Once imported, the resulting levees in the grid were reviewed and edited within the GDS for breaks, overlapping wall assignments and other quality control needs. The levee locations were then saved from the GDS to LEVEE.DAT for each sub-area model.

Initial model runs with the LEVEE.DAT file generated errors in the error.chk file where levee grids had adjacent grid cells with ground elevations higher than the top-of-wall elevation. This issue occurred primarily on walls that are short pony walls and where walls are located next to steep hilly slopes. To correct the errors, the top-of-wall elevation was increased to the nearest 0.1 foot above the adjacent ground elevation. For example, if a levee grid

with a levee in the north direction and a top-of-wall elevation at 1600.00 and the north grid ground elevation is at 1600.07, the levee elevation in the north direction was increased to 1600.10.

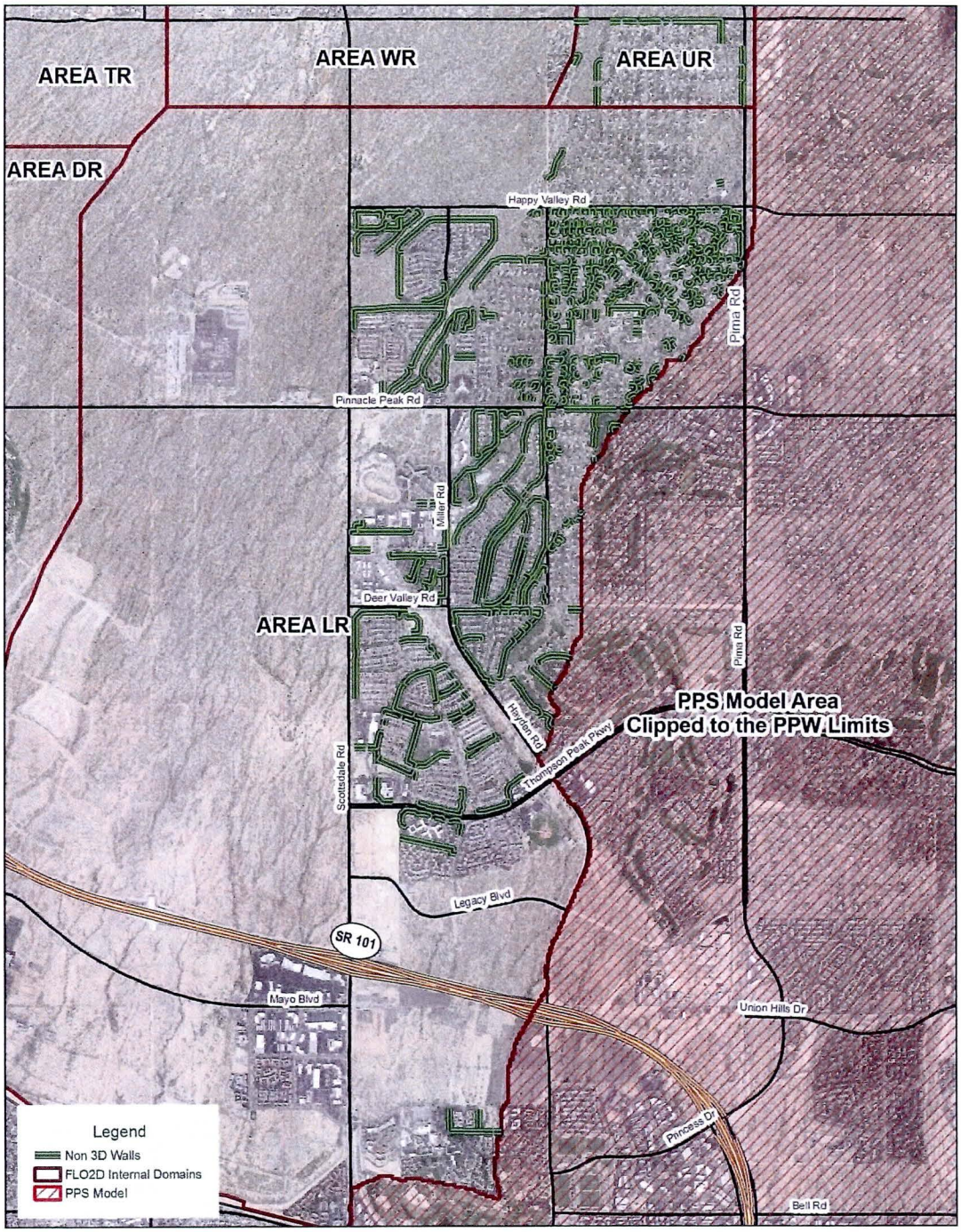

Figure 35. Modeled Walls Not Part of the 3D Data

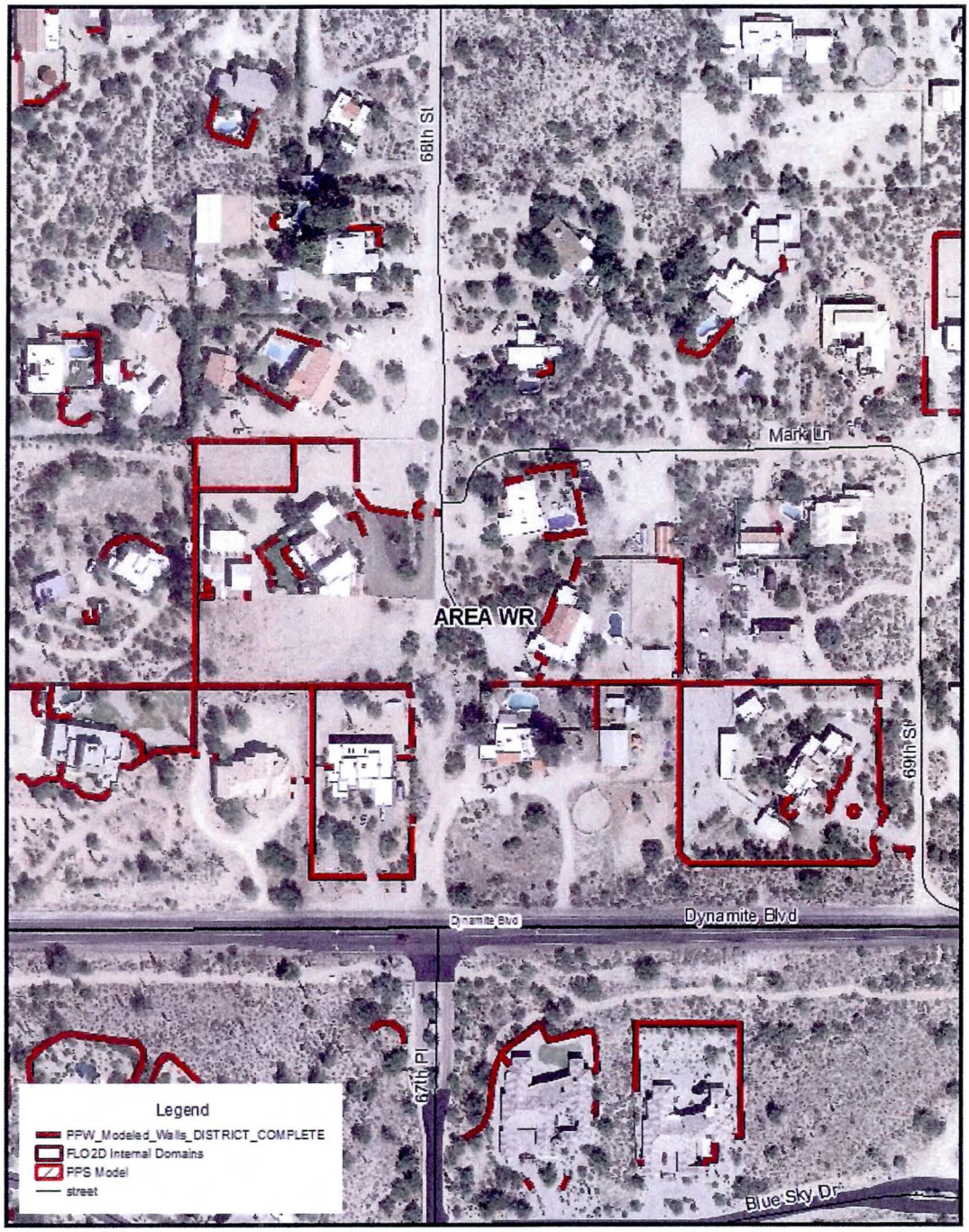

Figure 36. Initial Coverage of 3D Lines Including Insignificant Walls

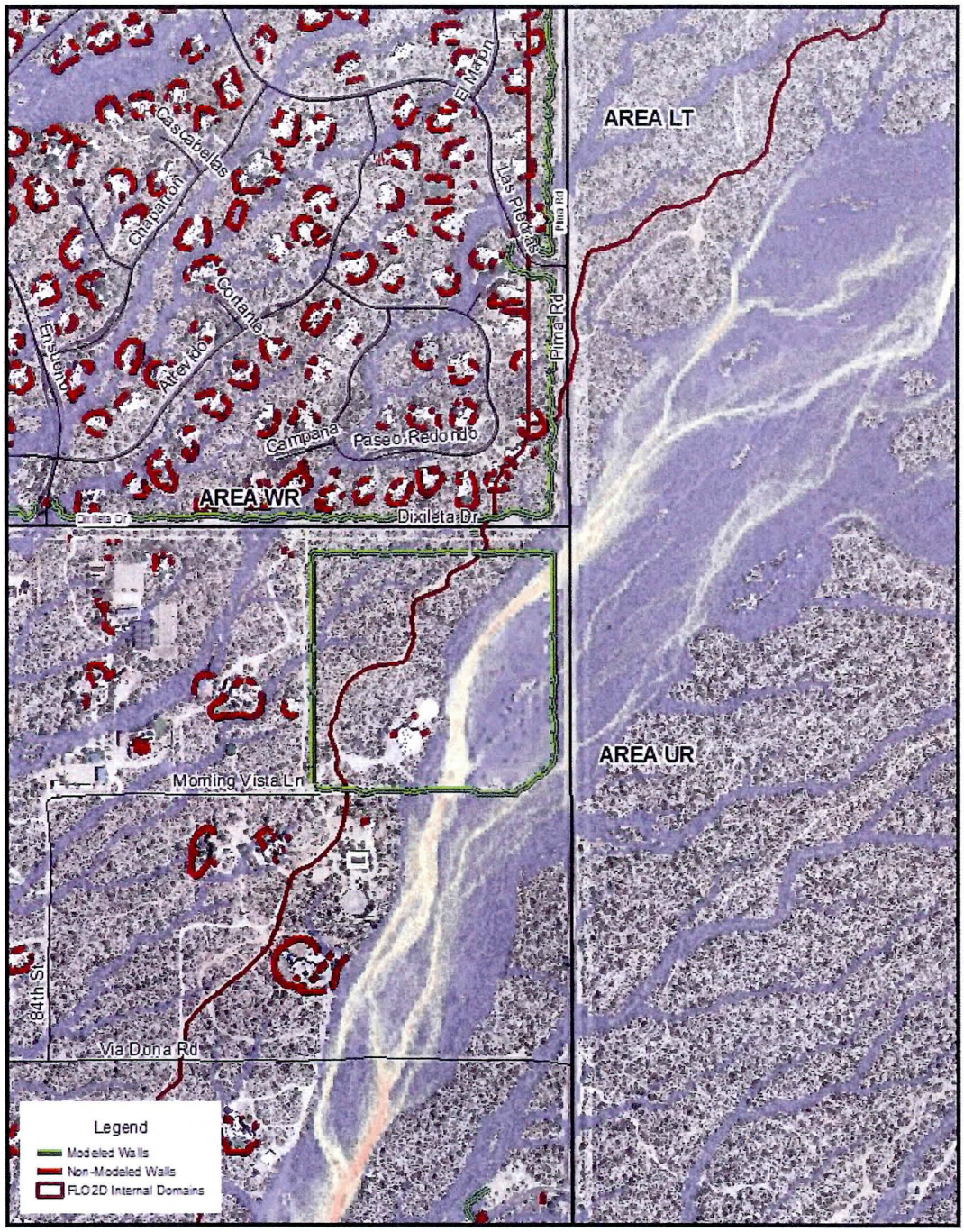

Figure 37. Modeled (Green) and Non-Modeled (Red) Walls

5.5.3.2. Property Wall Failure

Property walls are failed when flow depths reach a ponded depth of two feet against the levee or two feet above adjacent grid elevations, whichever is higher. There are many cases where adjacent grids have a higher ground elevation than the ground elevation of the levee grid, similar to cases where a wall is partly a retaining wall. In this situation, the adjacent grid plus two feet becomes the levee failure elevation in that direction. This method allows for two-feet of ponding against the levee itself rather than including the difference in ground elevation between the two grids in the failure depth. An example of this methodology is shown in Figure 38 where the arrows indicate the direction of flow/failure.

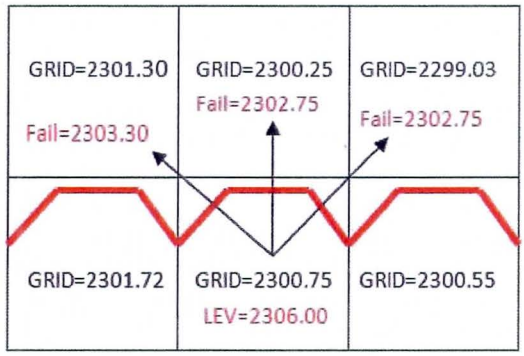

Figure 38. Method of Determining Property Wall Failure Elevation

5.5.3.3. Property Wall Failure Manual Adjustments

The two foot failure criteria is set on a global scale for the entire watershed. However, the failure elevations were manually adjusted at the wall located southwest of the intersection with Pima Road and Dixileta Drive. There is a large wall surrounding the property and is located within the Rawhide Wash corridor. There are large wall openings (Upstream: S094-20A-F, Downstream: S094-22A-G) with a wall opening height of 4-feet. Due to the importance of these structures and their appearance in the field, the wall failure elevations for the grids coinciding with these structures were increased to 6-feet to allow for 2-feet of ponding against the wall above the wall opening.

5.5.4. <u>One-Dimensional Channels (CHAN.DAT)</u>

Channel segments modeled in FLO-2D using the one-dimensional channel function include engineered channels with both natural and engineered material beds. Selection of engineered channels modeled using the 1-D channel function is based on the channel bottom width relative to the floodplain grid size. At a floodplain grid size of 20 feet, it is assumed that the flow conveyance for channels with bottom widths greater than 20 feet will be adequately represented using the floodplain grid elements. Therefore, only engineered channels with bottom widths less than 20 feet are modeled using the 1-D channel function. In total, approximately 16 miles of one-dimensional channels were modeled in the study area. The location of the 16 miles of one-dimensional channels is shown in Figure 39. The mileage per model area is listed in Table 31.

Table 31. 1-Dimensional Channel Mileage by Model Area

Model Area	1-D Channel Mileage
LT	0.0
UR	0.0
WR	0.3
TR	2.7
LR	4.1
CB	1.1
DR	7.5
R11	0.7*

*The channel mileage for R11 is included in the DR mileage since the R-11 and LR/DR sub-areas overlap. The R-11 mileage is not an additional length.

Channel parameters used in the modeling include channel alignment, cross sectional geometry, and Manning's roughness coefficients. A discussion of each of these data components is provided in the following sections.

5.5.4.1. Channel and Cross Section Alignment

One-dimensional channel cross section alignments are defined by the left and right bank grid cells. Using aerial photography and digital elevation data, grid cells were chosen to represent the left and right embankment location. The left embankment location is used in the CHAN.DAT to define the channel alignment and reference for all other channel data. Left and right embankment locations are defined in the CHANBANK.DAT and represent the alignment for each cross section.

5.5.4.2. Manning's N-Values

Manning roughness coefficients were chosen to account for vegetation and surface roughness conditions by examining aerial imagery. In general, a roughness coefficient of 0.03 to 0.04 was selected for channels with natural beds with slight vegetation. Channels lined with shotcrete were given a roughness coefficient of 0.02. The roughness coefficient for each cross section is represented in the CHAN.DAT file.

5.5.4.3. Cross Sectional Geometry

Cross sectional geometry was developed from the detailed digital topographic mapping. For engineered channels, it was assumed that the cross sectional geometry remained relatively uniform throughout the reach. Using HEC-GeoRAS and the detailed topographic data, a cross section geometry for the upstream and downstream most section was developed. The PROFILES tool was then used to interpolate the channel geometries along the length of the reach. Reaches that were not uniform throughout the reach or were longer in length, multiple cross sections were sampled throughout the reach interpolated all through the reach to better represent the true shape of the channel. Adjustments to the resulting station and elevation data were made such that the endpoint elevation of the cross section is not greater than or less than one foot that of the corresponding grid cell floodplain elevation. The resulting geometry is reformatted to the FLO-2D requirements and coded in the XSEC.DAT file.

Flow exchange between the floodplain and channel elements only occurs at the cross section endpoints. Therefore, cross sectional geometry at the upstream and/or downstream ends of each segment must be adjusted to allow this transition. This involves lowering the bank elevations and extending the cross section out onto the floodplain.

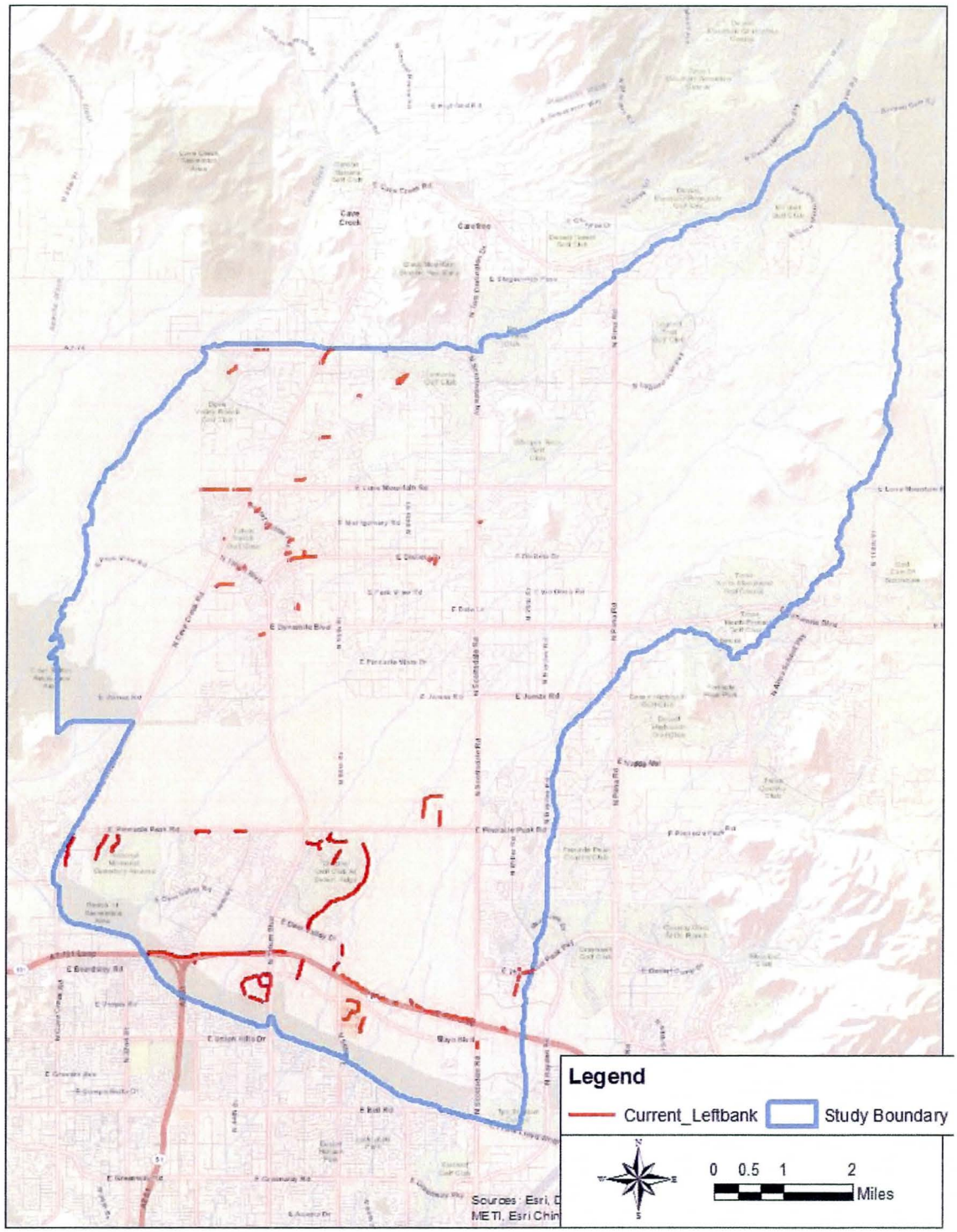

Figure 39. Location of 1D Channels

5.6. Floodway Modeling

Floodways are not modeled as part of this study.

5.7. Issues Encountered During the Study

Two primary issues arose during the FLO-2D modeling related to hydraulics, revisions to the rating curves for select hydraulic structures in the HYSTRUC.DAT file and one-dimensional channels that act more as storage features rather than actual conveyance structures. Each of these issues is discussed below.

5.7.1. Special Issues and Solutions

5.7.1.1. Rating Curve Revisions

As discussed in Section 5.5.2.4, FLO-2D will alter and revise a hydraulic structure rating curve to maintain model stability. Structures that have been revised are noted in the ERROR.CHK file and the portion of the rating curve that was revised is written to the REVISED_RATING_TABLES.OUT file. Generally, these revisions were due to elevation disparities between the structure outlet grid and the downstream grids. Since the inlet and outlet elevations of the hydraulic structures were set to actual invert elevations from the terrain dataset breakline data, see Section 5.5.2.1, the downstream grids were in many cases higher than the outlet elevation due to the grid elevation averaging. Any flow that went through a structure with the outlet in this 'sump' condition was ponded at the outlet until the flow depth reached an elevation to spread out onto the downstream grids. This inaccurate tailwater condition caused FLO-2D to revise the ratings to stabilize the structure.

The solution to this issue was to review each structure that was being revised and written to the REVISED_RATING_TABLES.OUT file. Once the structures being revised were identified, the location was reviewed to check for a topographical solution. This involved the lowering of grid element elevations to create a smooth outfall condition to the structure instead of having a sump condition. This approach was only done if the topographic data supported the lowering of grids. This would occur where the grid elevation averaging assigned elevations that were higher than the true wash bottom elevation.

In locations where the outlet was not in a sump, topographically and by FLO-2D grid element elevations, and the rating curve was being revised, the hydrograph was reviewed to check if the revision was affecting the peak of the hydrograph. If not and the hydrograph was smooth and showed no irregularities, the structure was left alone and FLO-2D was allowed to continue to revise the rating curves. If the hydrograph showed irregularities or the revisions were occurring at or near the peak, the rating curve in the HYSTRUC.DAT file was revised to incorporate the revision from FLO-2D and the HYSTRUC.DAT rating was "blended" with the revised rating section to create a smooth transition.

There are select locations where the outlets of structures are in a true sump condition based on the topographic data (e.g. retention basins or channels) and the structures were left alone and FLO-2D was allowed to continue to revise the ratings.
Section 5.5.2.4 provides more detail regarding the rating curve revisions.

5.7.1.2. Special 1D Channel Considerations

Within the Sub-watershed Lower Rawhide Wash (LR), the Loop 101 intersects the entire watershed perpendicular to the general direction of flow. Flow is transported across the freeway through a series of approximately 40 culverts within the watershed. Most of these culverts are connected by interceptor channels running parallel to the freeway to aide in the drainage. The hydraulics of these channels can become very complex. This is due to a number of factors. Examples of factors are:

- Flow does not enter the channel uniformly spatially or temporally. As shown in Figure 40, flow is shown entering the channel from the upstream and downstream ends and near the center of the channel.

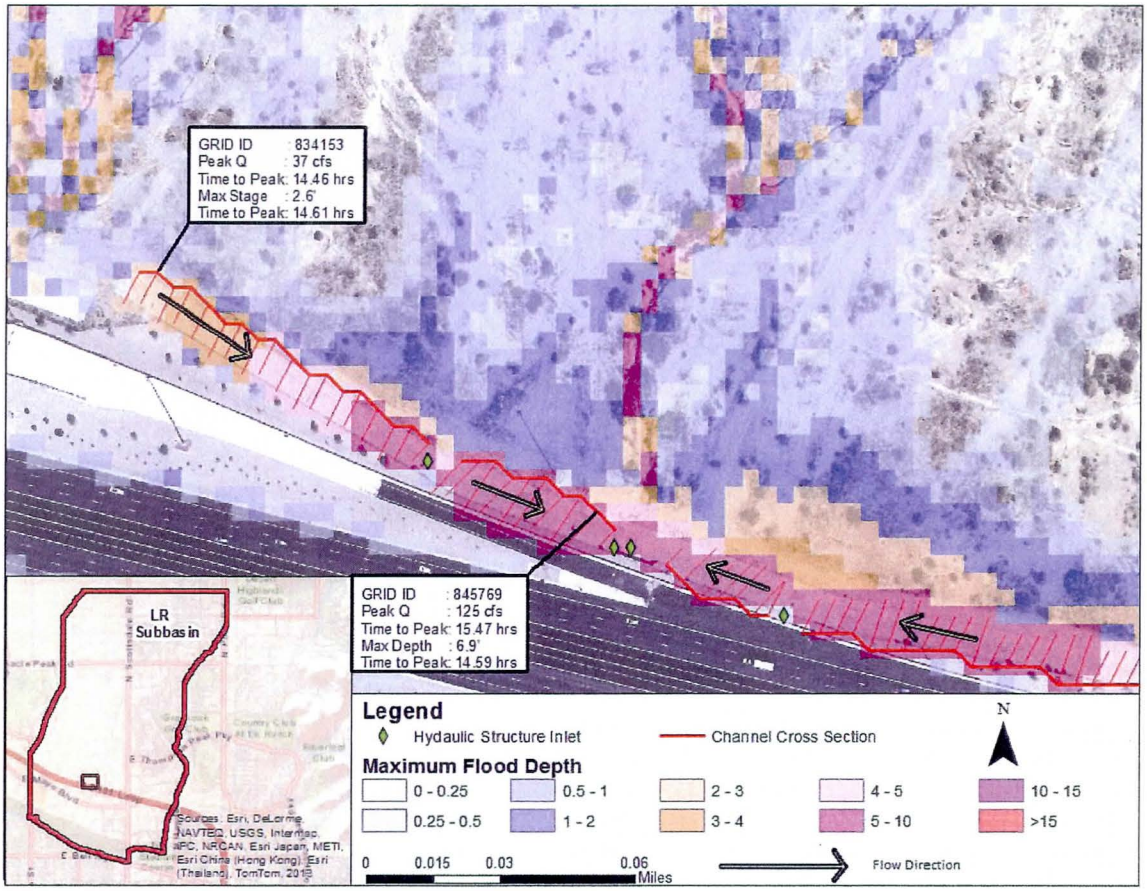

Figure 40. Just North of Loop 101

- The culverts crossing the freeway do not have sufficient capacity to convey all the flow without significant ponding. As seen in Figure 41, a high volume of flow ponds around the inlet of the hydraulic structure.

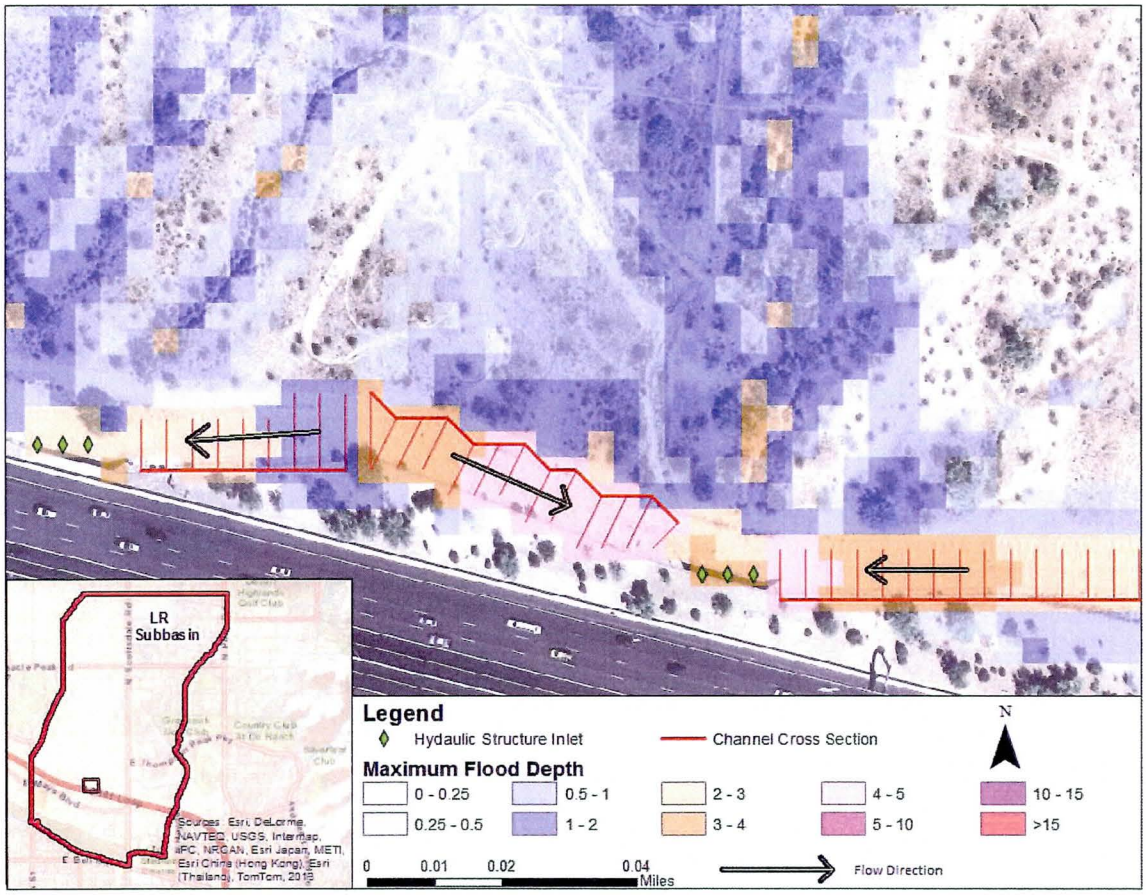

Figure 41. Just North of Loop 101

- At many locations, both factors occur simultaneously. Figure 42 illustrates an example where flow enters the channel initially from the center of the channel. However the culvert connected to the channel does not have sufficient capacity of convey the flow entering the culvert from the northeast. As a result, the flow starts in the expected flow direction within the channel, but experiences a change in momentum direction when the water ponding at the culvert builds within the channel.

Figure 43 demonstrates these changes within the hydrograph. These changes in momentum and flow direction can appear to result in numeric instabilities when examining the hydrographs for the channel cross section. However, the model is acting appropriately for this situation.

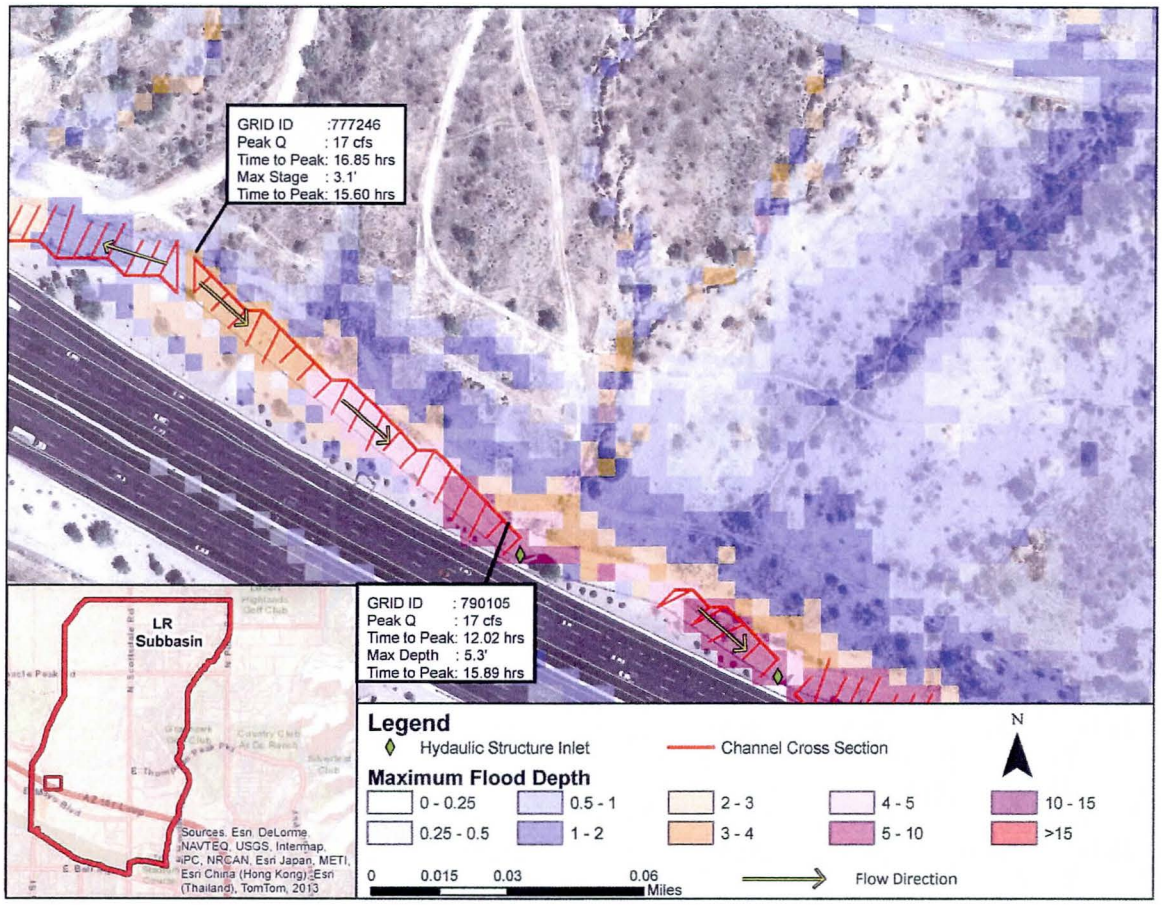

Figure 42. Just North of Loop 101

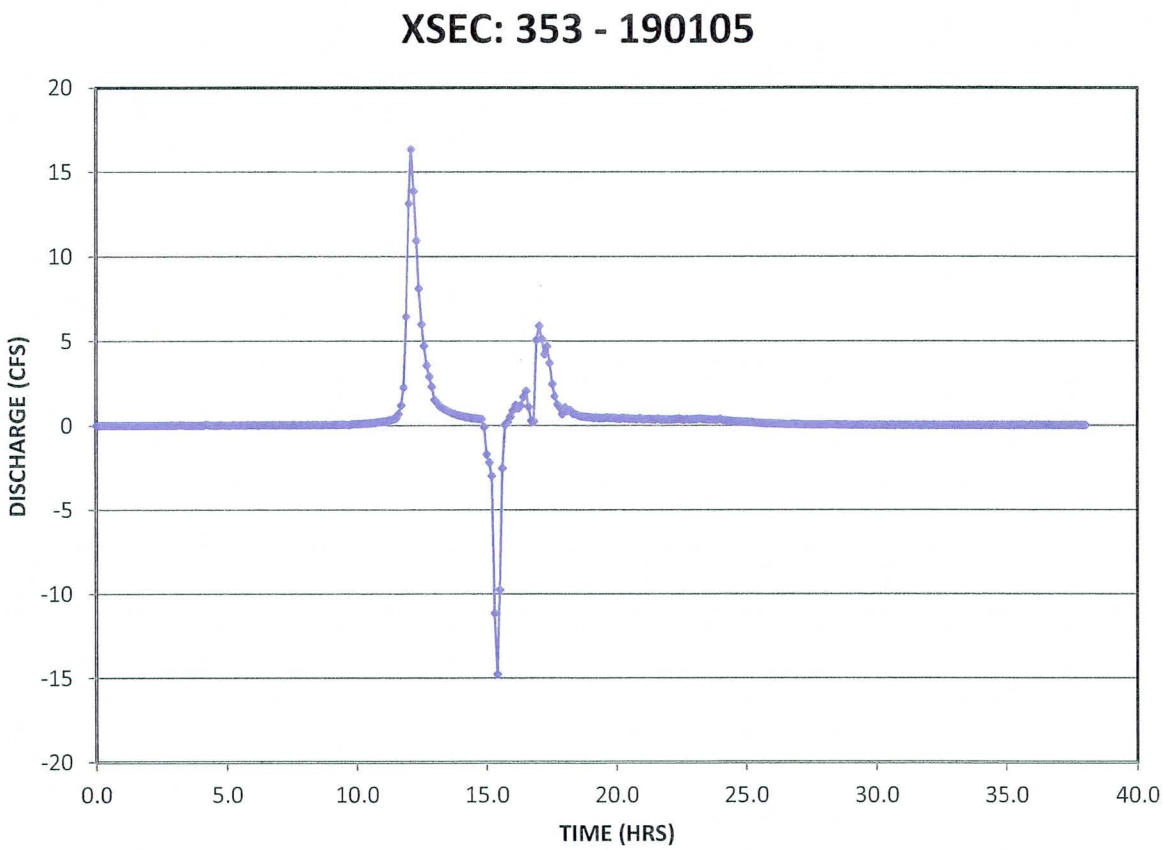

Figure 43. Hydrograph Channel Segment 353

Many of the channel hydrographs have two peaks, one for the local flow peak from local watershed rainfall and another peak from the off-site flow of the upstream watershed. Typically, the local peak of the hydrograph is smooth and occurring around hour 12.1. The off-site flow comes in typically around hours 15 to 16 and flow spreads into the channel and begins the ponding previously noted. An example of this is shown in the 1D channel located west of Tatum Boulevard and north of the L101, see Figure 44. There is a large hydraulic structure that conveys upstream flow into the 1D channel. This structure creates instabilities in the second peak of the channel hydrograph as flow enters the channel and begins to back up and slow down the conveyance of channel flow as shown in Figure 45 and Figure 46 . The hydrograph plot of Channel Segment #519, Figure 45, illustrates the instability. As the flood wave moves downstream and acts more like conveyance, the channel hydrograph stabilizes and the hydrograph second peak becomes smoother, Figure 46.

Figure 44. Channel Segment near Tatum Blvd and L101

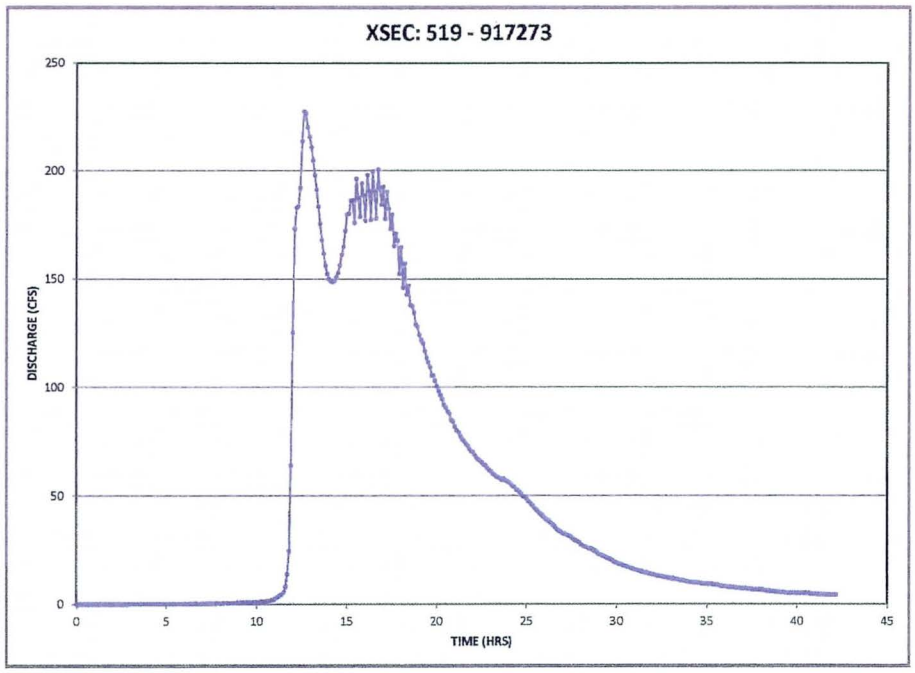

Figure 45. Hydrograph Channel Segment 519

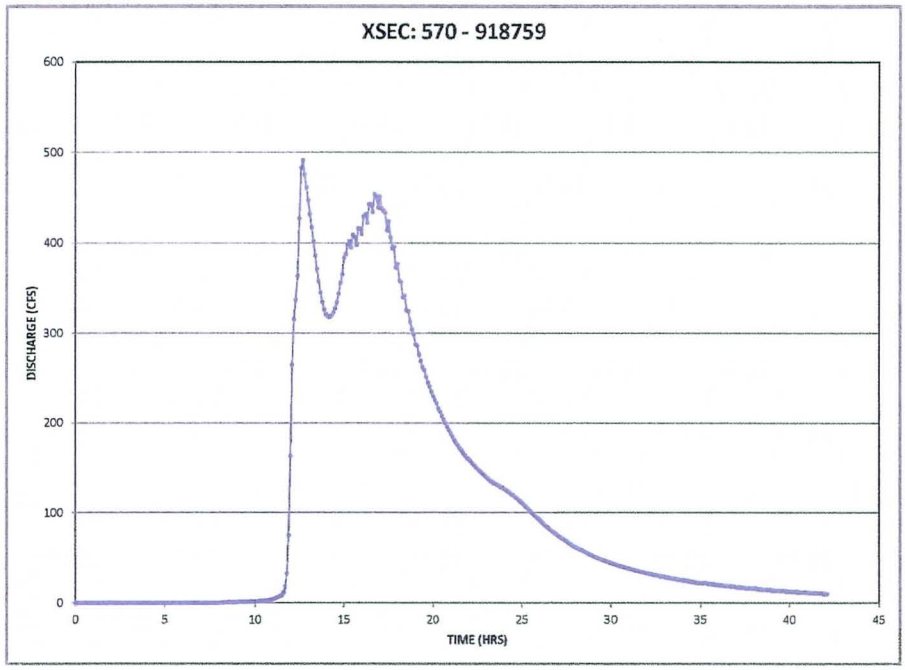

Figure 46. Hydrograph Channel Segment 570

5.7.2. Modeling Warning and Error Messages

FLO-2D computes the hydraulics and hydrology concurrently in the same model; therefore, the modeling warning messages from the Hydrology Section 4.8 apply to the hydraulics as well. There are no additional warnings or errors to list in this section.

5.8. Calibration

The only FLO-2D model calibration that was conducted was the infiltration calibration to achieve a reasonable percentage of runoff from each FLO-2D sub-model. A detailed discussion of this provided in the Hydrology portion of this report in Sections 4.2.6.1 and 4.9.

5.9. Final Results

Final results are shown on the following exhibits in the Appendix:

- Maximum Depth Results (100-Year)
- Maximum Velocity Results (100-Year)
- Peak Discharge Results (100-Year)

5.9.1. Hydraulic Analysis Results

In FLO-2D, the hydraulic results are summarized on a grid-by-grid basis. It is therefore not considered practical to tabulate and summarize the results of each grid since the PPW ADMS is comprised of roughly 7-million grid elements. The model output has been rasterized and is displayed on large scale exhibits in the appendix for depths and velocities. The model output is located with the digital data and is available there for review on a grid-by-grid basis.

5.9.2. Verification or Comparison of Results

Although there have been previous floodplain delineation studies completed in the study area, a verification of hydraulic results was not done since the discharges are different between the PPW ADMS and the previous floodplain delineation studies. In general, when comparing the FLO-2D results with the aerial photography and existing ground topography, the depth and velocity results match well with the ground data. The deeper more concentrated flows are confined to the deeper sandy wash corridors shown in the aerials and topography with shallow overland flow in the overbank areas.

6. EROSION, SEDIMENT TRANSPORT, & GEOMORPHIC ANALYSIS

6.1. Method Description

Erosion, Sediment Transport, and Geomorphic analysis was not included in the scope of work for this work assignment. All modeling was performed assuming a fixed bed analysis. A memorandum was prepared under an initial work assignment to the project contract (Work Assignment #1) that reviewed and summarized previous studies and historical documents and reports that address the geomorphic landforms within the study area, particularly whether the landforms exhibit characteristics of alluvial fans (See PPW ADMS Task 6.1 Alluvial Fan Landform Verification Memorandum). A copy of the memorandum is located in the appendix.