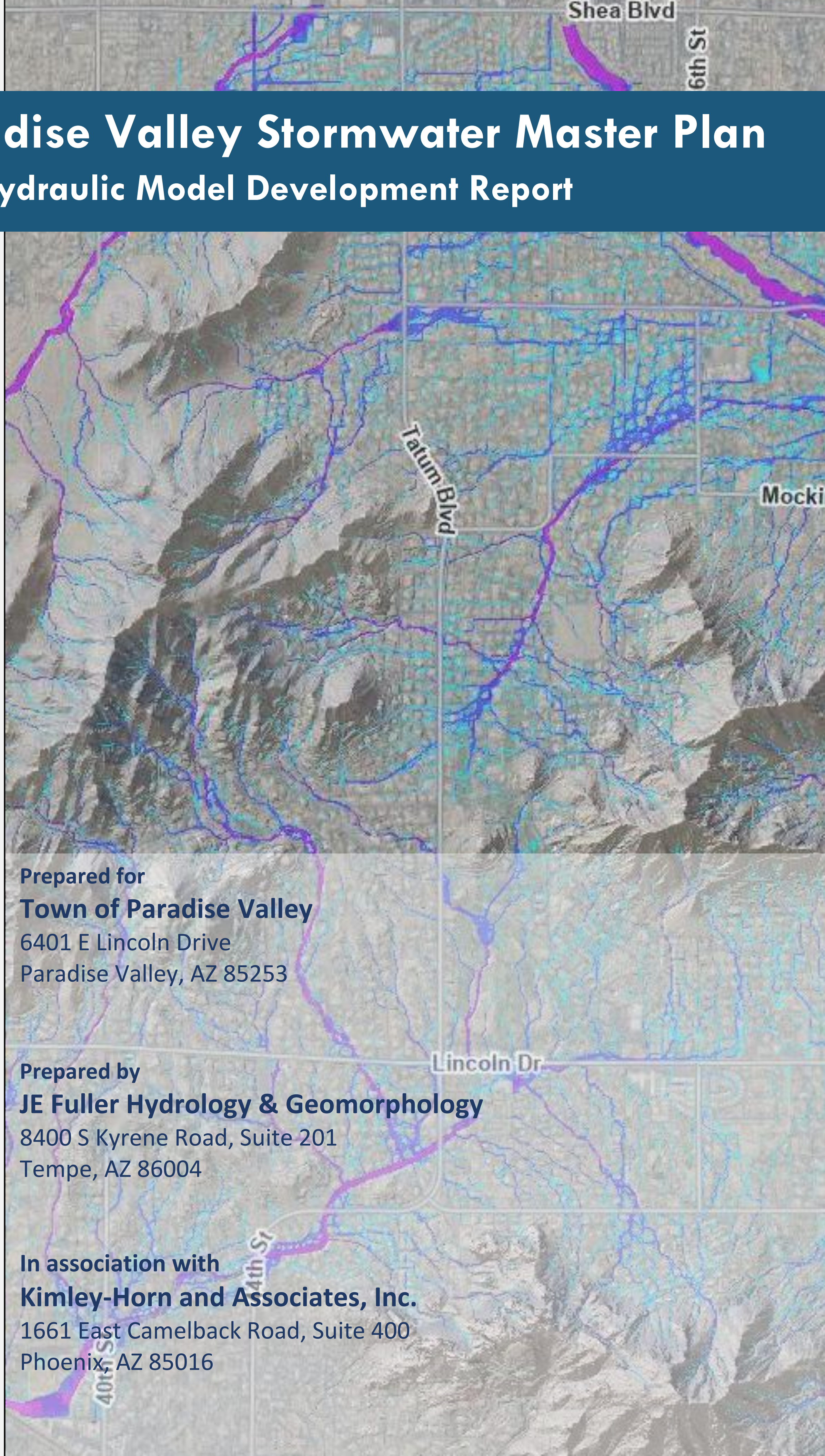
Town of Paradise Valley Stormwater Master Plan

Hydrologic and Hydraulic Model Development Report

April 2025



Prepared for

Town of Paradise Valley
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Prepared by

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In association with

Kimley-Horn and Associates, Inc.
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Hydrologic and Hydraulic Model Development Report

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Prepared for:

Town of Paradise Valley
Contact: Shar Johnson, MBA, PE
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Registered Professional Engineer (Civil)
CERTIFICATE NO.
63381
PETER MICHAEL
ACTON
Date Signed 4/8/2025
ARIZONA, U.S.A.
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Abbreviations

ACDC	Arizona Canal Diversion Channel
ac-ft	acre-feet
ADMP	Area Drainage Master Plan
ADMS	Area Drainage Master Study
ADMS/P	Area Drainage Master Study & Plan
ALERT	Automated Local Evaluation in Real Time
FCDMC	Flood Control District of Maricopa County
HEC-RAS	Hydrologic Engineering Center River Analysis System
H&H	Hydrologic and Hydraulic
IBW	Indian Bend Wash
MIBW	Middle Indian Bend Wash
NOAA	National Oceanic and Atmospheric Administration
PRISM	Parameter-elevation Regressions on Independent Slopes Model
PVSWMP	Paradise Valley Stormwater Master Plan
SFC	Surface Feature Characterization
USGS	United States Geological Survey

1 INTRODUCTION

This document outlines the development of the hydrologic and hydraulic (H&H) components of the Town of Paradise Valley (Town) Storm Water Master Plan (SWMP). These components comprise the entirety of the existing condition H&H model development.

1.1 Study Purpose

The purpose of this study is to develop a Town-wide H&H model using the completed Flood Control District of Maricopa County (FCDMC) FLO-2D models as the starting point. The goal was to develop an integrated model that depicts the entirety of the Town of Paradise Valley (Town) limits. This model will then be leveraged to evaluate existing condition drainage issues throughout the Town, and it will further be used to evaluate the effectiveness of various conceptual flood mitigation strategies.

1.2 Study Location

The study area fully encompasses the 15.4 mi^2 boundary of the Town of Paradise Valley, Arizona. **Figure 1-1** depicts the location of the Town as it relates to the Phoenix Metropolitan area.

The study area is generally bounded by Shea Boulevard to the north and Scottsdale Road to the East. The western and southern boundaries are generally are bounded by the Piestewa Peak mountainous area and Camelback Mountain, respectively. Several notable peaks reside within the Town boundary itself, one example being Mummy Mountain.

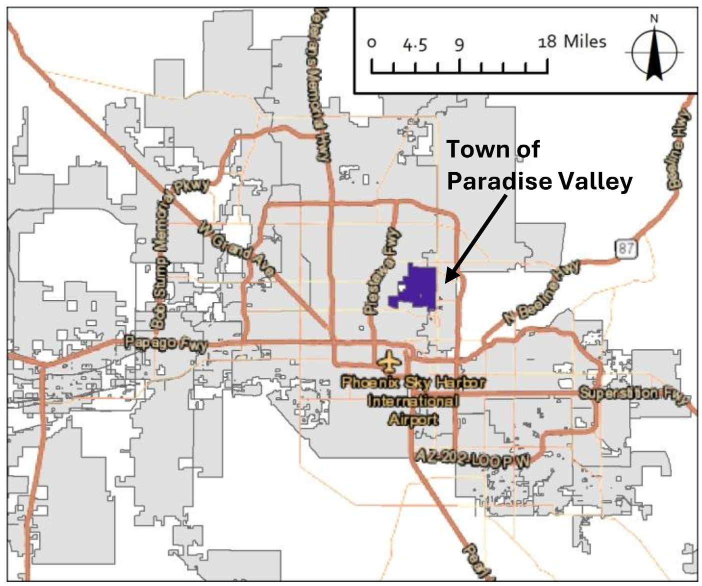


Figure 1-1. Study Location.

1.3 Consultant Team

The consultant team performing the overall SWMP study is led by Kimley Horn and Associates, Inc. (KHA) under Contract # CON-24-015-ENG. As part of the overall study, JE Fuller Hydrology & Geomorphology, Inc. (JE Fuller) performed the H&H model development tasks. Contact information for the consultant team is shown below in **Table 1-1**.

Table 1-1. Contact Information.

Consultant	Name	Role	Email	Phone
KHA	Geoff Brownell, PE	Project Manager	geoffrey.brownell@kimley-horn.com	602.944.5500
JE Fuller	Peter Acton, PE	Project Manager	peter@jefuller.com	480.222.5701

2 DATA COLLECTION AND MAPPING

This section outlines the data collection performed for this study.

2.1 Digital Data Collection

The location of the Town boundaries resides within the intersection of five Area Drainage Master Studies/Plans (ADMS/P) performed by the FCDMC. All five studies were performed using the FLO-2D modeling software. While the summation of the boundaries of the five models completely covers the Town boundaries, no singular study comprised more than half of the Town. Therefore, new model boundaries were developed, and input data for this new modeling relied heavily on the datasets used in the five ADMS studies. Data pertaining to each study was requested from the District by JE Fuller and the received data included both the entirety of the FLO-2D model input and output files as well as relevant geospatial data. The five studies are listed below in **Table 2-1**.

Table 2-1. ADMS/Ps Referenced in this Study.

Study Name	Abbreviation	Dates			Prime Consultant	Cell Size
		Topographic Data Capture	Model Development	Completed		[ft]
East Shea Corridor (Wilson, 2023)	E Shea	2007 - 2016	2016	2024	HELM-Wilson & Co., Inc.	15
Phoenix Metro (Wood Patel, 2021)	Metro	2018	2020	On-going	Wood Patel, Inc.	20
Cudia City Wash (Michael Baker, 2020)	Cudia	2015	2019	2020	Michael Baker International	15
Middle Indian Bend Wash (Kimley-Horn, 2019)	MIBW	2014	2015	2019	KHA	20
Lower Indian Bend Wash (Gavin & Barker, 2017)	LIBW	2007	2013	2017	Gavin & Barker, Inc.	20

Data leveraged from these five ADMS/Ps include inflow hydrographs, surface feature characterization (SFC) shapefiles (including building footprints), storm drain infrastructure, one-dimensional channels, and hydraulic structure (i.e., culverts).

A comprehensive soil coverage dataset was downloaded from the District website in March 2024, and precipitation datasets were also obtained from the District. This soil dataset represents the implementation of new Green and Ampt (GA) parameters that were developed by the FCDMC in late 2023. None of the five ADMS/Ps used this newer dataset.

2.2 Topographic Mapping

While much of the input data obtained from the previous ADMS modeling efforts were included in the present study, newer and more refined topography spanning the entirety of the new modeling domain was available. This dataset, obtained through the United States Geological Survey (USGS) 3D Elevation Program (3DEP) (Sanborn, 2021). The effective date of this data source is November 11th, 2020. This topographic dataset was used for the entirety of the current modeling effort.

2.3 Survey Data Collection

Survey data was provided by KHA to verify and obtain hydraulic structure properties.

2.4 Digital Projection Information

Geographical Information System (GIS) files were developed using the following projection information:

- Vertical Datum: The North American Vertical Datum of 1988 (NAVD88)
- Horizontal Datum: NAD 1983 HARN State Plane Arizona Central (WKID 2868)
- Units: International Feet

3 MODEL DEVELOPMENT

Hydrologic and hydraulic modeling for this study has been completed with the use of the FLO-2D PRO modeling software. This is a volume-conserving, two-dimensional (2D) flood routing model. The model routes flow (rainfall runoff and inflow hydrographs) over a grid comprised of square elements based upon topography (defined by the elevation of each grid element) and roughness parameters. This 2D modeling approach is highly suited for simulating the complex and distributary flow prevalent within the watersheds of this study. The FLO-2D model also incorporates significant storm drains, culverts, walls, and channels within the modeling area.

The choice of this modeling software for this study was also influenced by the existing regional modeling in the area (**Table 2-1**) that had all utilized FLO-2D, thereby simplifying the development of the present model.

The FLO-2D PRO version used in this study is build number 23.10.25 with an executable date of November 9th, 2023. The QGIS plugin version used to develop some model input files was released on August 17th, 2021, and is version 0.10.32.

3.1 Method Overview

The analysis was completed using FCDMC guidance and recommendations for model parameter estimation and development (FCDMC, 2016; FCDMC, 2020; FCDMC, 2021[a]; FCDMC, 2021[b]) as well as two-dimensional modeling techniques appropriate for the area. The QGIS plugin for developing FLO-2D model inputs was used to create spatially-varied input files for topography, infiltration, precipitation, and Manning's *n*. Other input datasets were developed using in-house custom scripting.

This study included developing datasets for the 2-Year, 10-Year, and 100-Year recurrence intervals. Both the 6-Hour and 24-Hour durations were modeled (for the 100-Year event), and the controlling duration was selected based on an evaluation of the results. The 2-Year and 10-Year events were then modeled with the controlling duration only.

3.2 Model Domain and Sub-Models

The goal of this modeling effort was to develop a Town-Wide FLO-2D model (also termed PV model in this document) that will be used to develop and assess various flood reduction measures throughout the Town. That said, the entirety of the Town boundary needed to be incorporated into the model domain. Further, a cell size of 10 feet was desired, as this finer resolution (as compared to previous FLO-2D studies listed in **Table 2-1**) as it provides a more resolved and realistic depiction of flow paths and depths. While FLO-2D can be used to model extremely large areas with very fine detail, there is an upper limit to the number of cells in a model before model runtimes become exponentially long. This issue can be easily circumvented by delineating the study area into sub-models that run independently of each other.

The model area was delineated into two separate sub-models, termed "NORTH" and "SOUTH". These delineations are simply model constructs that make it easier and facilitate running larger models. In this paradigm, sub-models that are upstream must be executed first, and the outflow from the upstream model becomes inflow into the downstream model. For the PV model, the SOUTH model is upstream of the NORTH model. Statistics for each sub-model is shown in

Table 3-1, and **Figure 3-1** below depicts the two sub-model boundaries that comprise the overall PV model.

Table 3-1. Sub-model Statistics.

Sub-Model	Cell Size (ft)	# of Cells	Area (mi^2)
NORTH	10	3,143,855	11.28
SOUTH	10	2,943,125	10.56
Total		6,086,980	21.83

Model boundaries were drawn to include as much drainage area as possible within the current modeling area, as this minimizes the need for inflow hydrographs developed using other modeling methods (e.g., earlier version of the GA parameters) keeping in line with the study goal of developing a Town-wide comprehensive model. The result is that the PV modeling area is larger than the Town boundary itself, as it includes a significant amount of upstream tributary area. The boundary was also drawn to facilitate the inclusion of existing storm drain networks and channels and to allow for accurate headwater and tailwater calculations for culverts. Multiple iterations of delineations and review of results were performed to fine tune the model boundaries.

The Tatum Wash drainage area was included in the NORTH sub-model domain for several reasons. First, initial modeling indicates distributary flow as the wash transitions from undeveloped desert into more urbanized settings, with some fraction of the distributary flow crossing into the Town along the northwestern boundary of the Town. Secondly, model calibration (not included in this submittal) leverages both precipitation datasets and stage data collected at the Tatum basin. Including this area in the NORTH sub-model simplified the calibration effort and it provided more accurate inflow hydrographs.

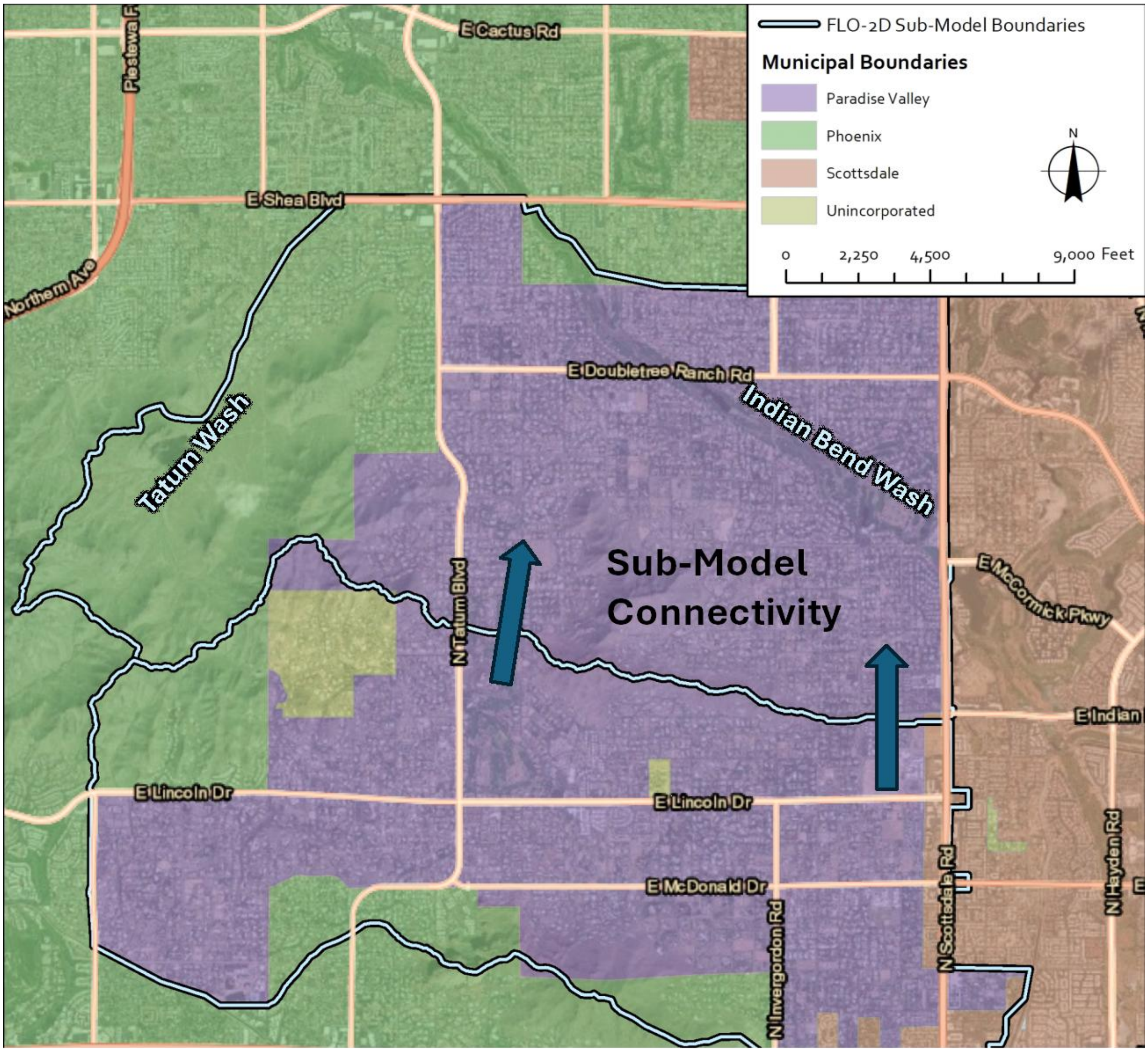


Figure 3-1. Sub-Model Boundaries and Connectivity.

3.3 Off-Site Model Inflows

The previous models were leveraged to account for inflows into the PV model. While the need for inflows from other models was minimized, the northern boundary of the PV model required offsite inflow to be included. The off-site models used to parameterize this inflow included both the MIBW and East Shea models. **Figure 3-2** below shows the spatial extents of where the offsite inflows are included in the context of the PV model boundary as well as the boundaries of the five off-site models. Several storm drain inflows from contributing studies were also identified and included.

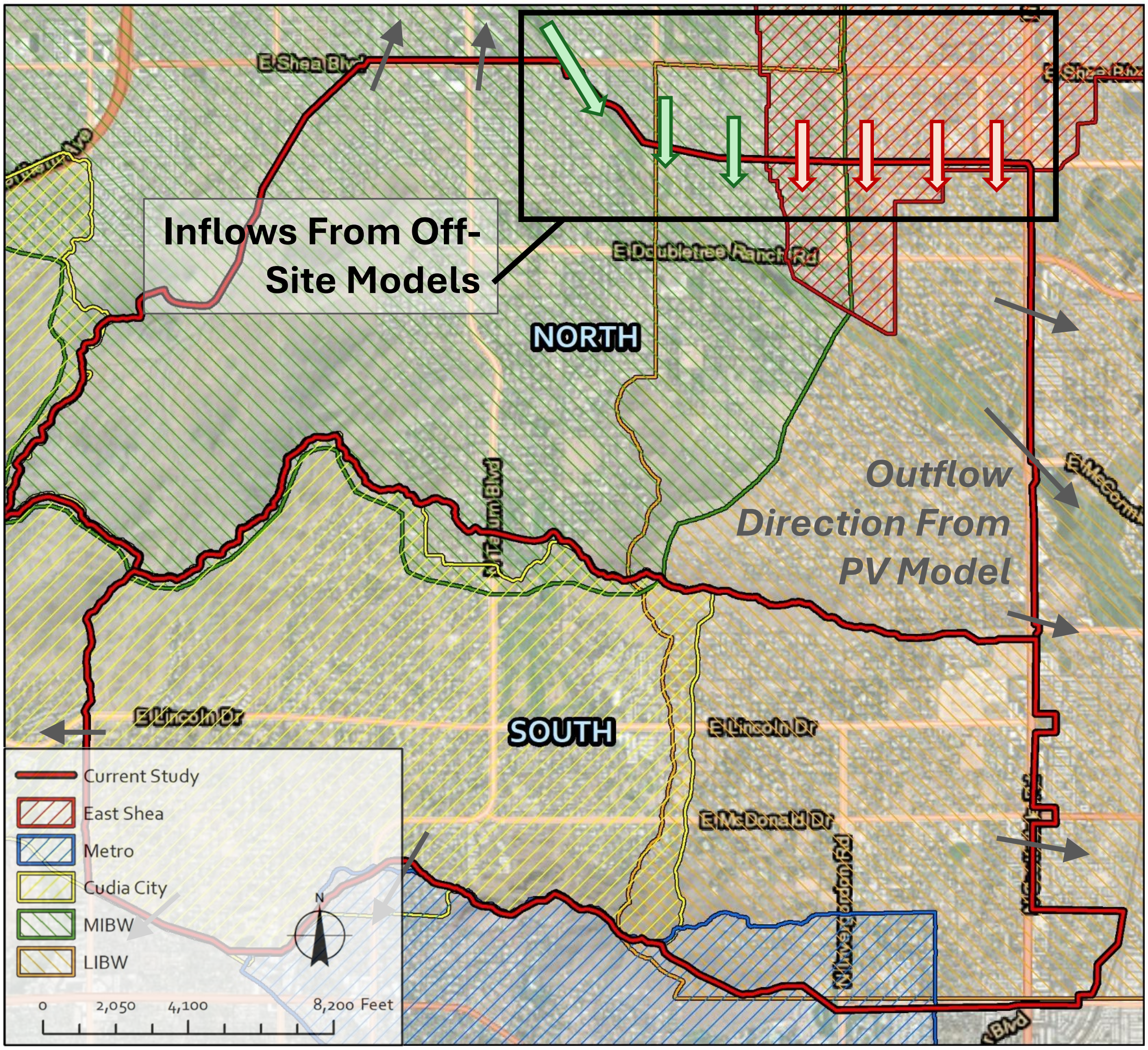


Figure 3-2. Offsite Model Boundaries and Inflows to Current Study.

The inflow hydrographs from the off-site models were obtained by using the original model files from each study (obtained from the FCDMC) and re-parameterizing the RAIN.DAT input files to develop models representing the 2-, 10-, and 100-year recurrence intervals for both the 6-hour and 24-hour durations. No other changes were made to the off-site model inflow files. For the MIBW model this required running two off-site models (MIBW-North and MIBW-South), while the East Shea Model only required one sub-model to be re-parameterized, as it is hydrologically detached from the other three sub-models. Since both studies overlap far into the PV model, new outflow nodes were placed in each off-site model domain directly on top of the upstream PV boundary to obtain hydrographs for each cell. Cells from the off-site models mapped to PV inflow cells to translate the outflow hydrographs from the offsite models into inflow hydrographs for the PV model. **Figure 3-3** depicts an example of how mapping was performed between an off-site model (with a larger grid size) and the PV model.

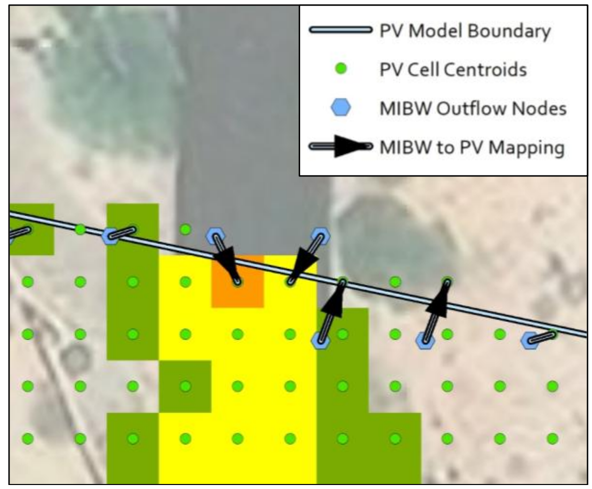


Figure 3-3. Off-site Model Mapping.

3.4 Surface Feature Characterization

The surface feature characterization (SFC) dataset defines the spatial land use for the model area. This polygon shapefile defines the extent of each land use category, and this dataset is used to develop FLO-2D input files pertaining to infiltration, blocked obstructions, and roughness parameters.

SFC datasets were generated as part of each study listed in **Table 2-1**, and these five datasets were all provided by the FCDMC. Each dataset was then merged to create one comprehensive, Town-wide SFC dataset. For areas overlapping SFC coverages the newest dataset was used. Given the varying dates of the development of each contributing SFC dataset, a close examination of the Town-wide SFC shapefile was performed against aerial datasets (dated 2024) to look for changes in land use since the underlying SFC development. Manual changes were made as necessary to update the SFC based on this newer aerial dataset.

The resulting, updated SFC dataset is shown below on **Figure 3-4**.

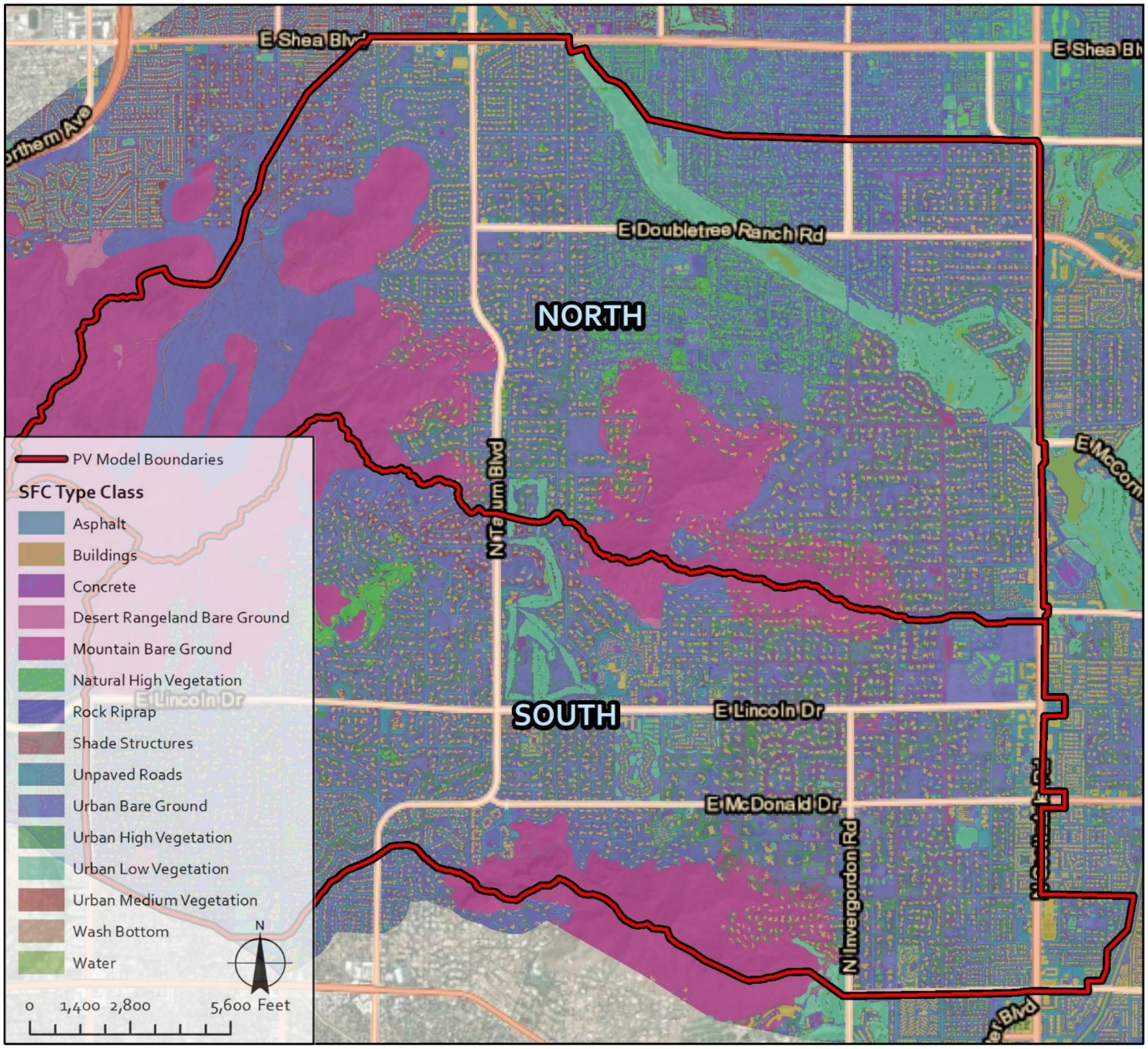


Figure 3-4. Merged and Updated SFC Dataset.

3.5 Development of Model Input Files

This section highlights the processes used to develop the FLO-2D input files.

3.5.1 Topography

3.5.1.1 Base Grid Elevation Assignment

The underlying topography for the model was developed by sampling the 10-foot model grid against the more resolved, 1-foot bare earth raster. Processes outlined in FCDMC (2021[b]) were used along with the QGIS plugin, as required by the District. A hill shade view of the topography in the study area is shown on **Figure 3-5** below.

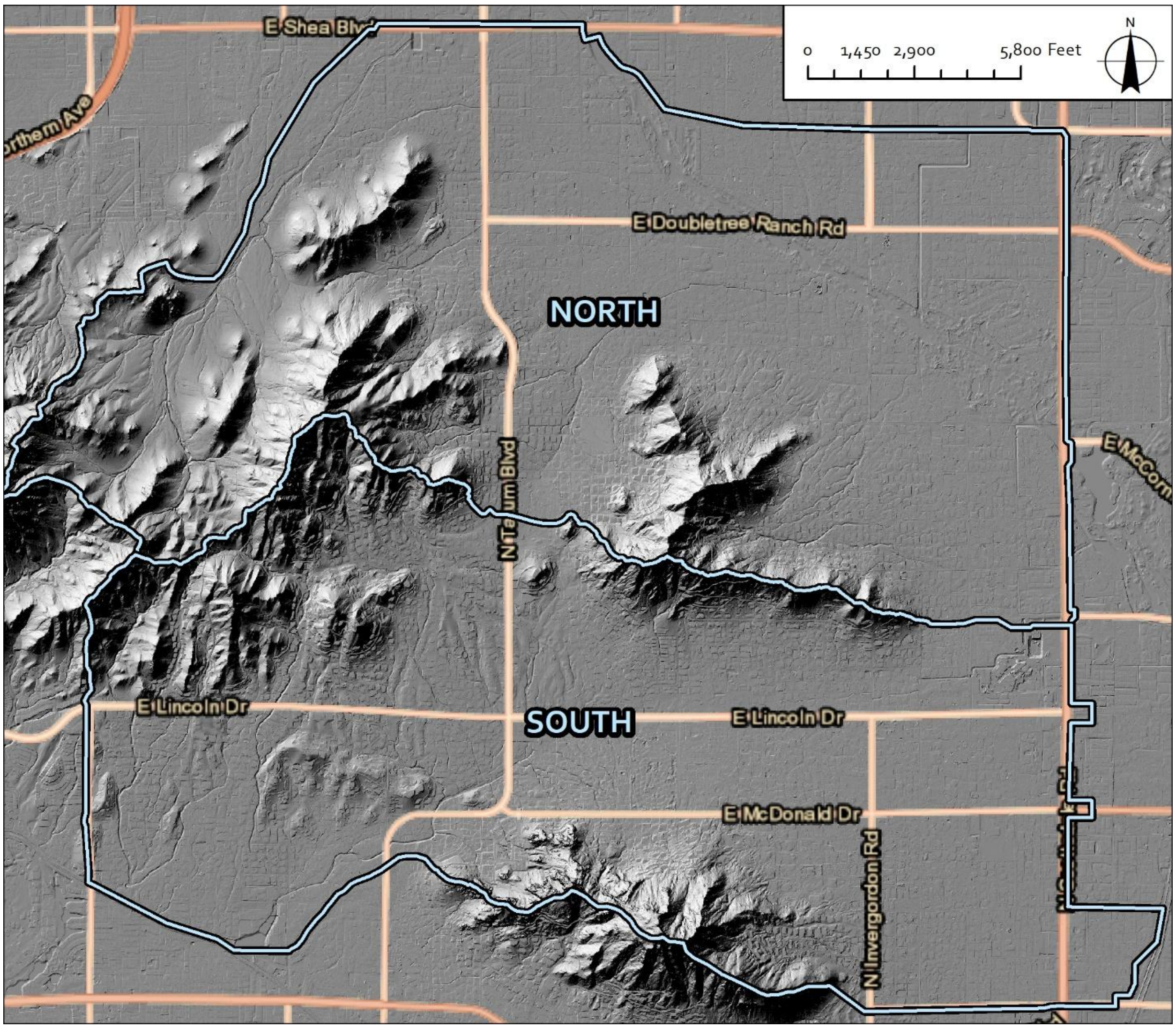


Figure 3-5. Hill Shade View of Study Area.

3.5.1.2 Elevation Adjustments

Elevation adjustments were made using a polygon shapefile to override elevations assigned in the rasterization of the source topography. This included establishing elevations for culvert and storm drain inverts along with changes to improve model stability. This shapefile is included in the digital deliverable.

3.5.2 Roughness

Roughness values in the model, represented by Manning's n values, were based on the SFC dataset discussed earlier in this report. Two spatially-varied input files were used in this model representing a depth-varying approach to assigning roughness. Both an n-value (MANNINGS_N.DAT) and an n-value corresponding to a shallow depth (SHALLOWN_SPATIAL.DAT) were used in this study. This differentiation allows FLO-2D to compute a time and depth-varied roughness value based upon the depth of water. In general, the n-value parameter corresponds to the roughness at a depth of three feet, and the shallow n-value applies to a depth of 0.5 feet. A logarithmic function is then used to compute roughness values in between these depths. The FLO-2D Data Input Manual can be referred to for more detailed information on this interpolation process.

Roughness value assignments by SFC category are listed below in **Table 3-2**.

Table 3-2. Roughness Values by SFC Category.

Class ID	Type Class	n	Shallow n
0	Natural High Vegetation	0.065	0.20
3	Urban High Vegetation	0.065	0.20
4	Urban Medium Vegetation	0.055	0.15
5	Urban Low Vegetation	0.045	0.12
6	Mountain Bare Ground	0.050	0.40
8	Desert Rangeland Bare Ground	0.040	0.25
9	Urban Bare Ground	0.035	0.13
12	Wash Bottom	0.035	0.15
13	Concrete	0.016	0.10
14	Asphalt	0.020	0.10
15	Buildings	0.024	0.12
16	Shade Structures	0.035	0.10
17	Water	0.040	0.10
21	Unpaved Roads	0.026	0.10
23	Rock Riprap	0.065	0.25

Increases in n-values in cases of deep ponding is applied in the development of FLO-2D modeling to improve model stability. Adjustments to n-values in ponded locations were made using criteria shown in **Table 3-3** below.

Table 3-3. Manning's n-value Adjustment for Ponded Depth.

Ponded Depth (ft)	Manning's n-value
5 - 8	0.08
8 - 10	0.1
10 - 15	0.2
Greater than 16	0.3

3.5.3 Precipitation

The RAIN.DAT input file defines the rainfall temporal and spatial distribution for the FLO-2D model. Two RAIN.DAT files were created to represent two design storms. Given the large scale of the modeling area, the 24-hour event may control in larger washes and channels, and the 6-hour event may control for smaller tributary watersheds throughout the modeling area. Both the 6-hour and 24-hour storm durations were run for the 100-year storm at the preliminary modeling phase, and a single duration was selected for use to perform the detailed modeling.

The rainfall depths were taken from the NOAA Atlas 14, Precipitation-Frequency Atlas of the United States, Volume 1: Semiarid Southwest (Arizona, Southeast California, Nevada, New Mexico, Utah) as provided on the FCDMC website for public download, and the temporal distributions were taken from the FCDMC Hydrology Manual. The SCS Type II pattern was used for the 24-hour duration, and the Maricopa County Pattern No. 1 was used for the 6-hour duration. The maximum and minimum rainfall depths for the study area are shown below in **Table 3-4**. The 24-hour and 6-hour distributions are also shown below on **Figure 3-6.**

Table 3-4. Precipitation Depths.

Event	Maximum	Minimum
	[in]	[in]
2-Year 6-Hour	1.176	1.121
10-Year 6-Hour	1.766	1.689
100-Year 6-Hour	2.691	2.581
100-Year 24-Hour	3.766	3.526

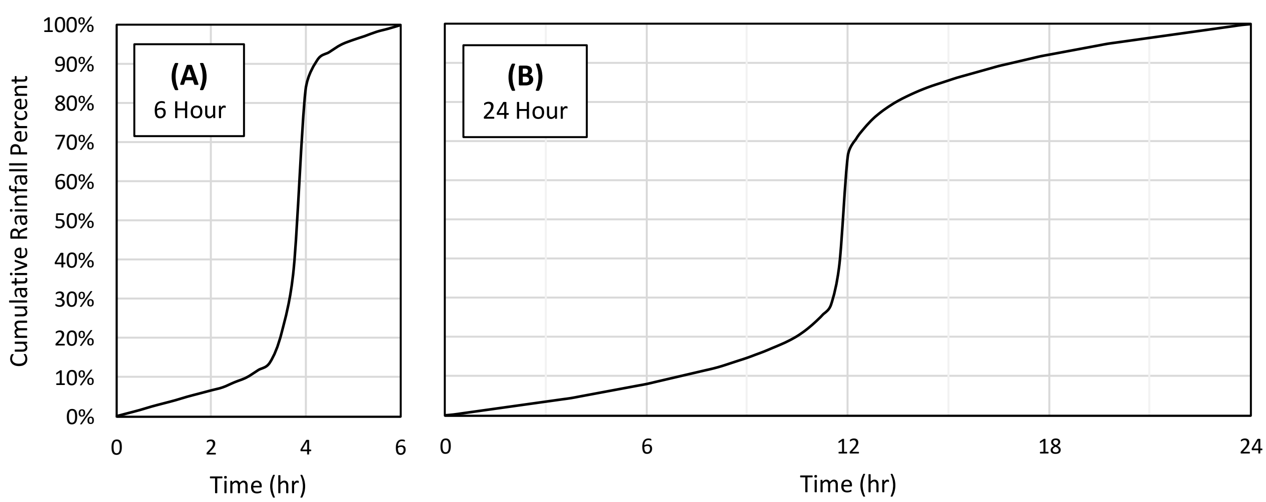


Figure 3-6. (A) Maricopa County 6-Hour Local Storm Temporal Distribution (Pattern No. 1) and (B) SCS Type II 24-Hour Temporal Distribution.

The RAIN.DAT model input files were developed using the QGIS plugin and the rasterized precipitation datasets obtained from the FCDMC. **Figure 3-7** below depicts the spatial distribution of the NOAA Atlas 14 precipitation statistics for the four modeled events. **Section 4.3** below highlights the selection of the controlling duration.

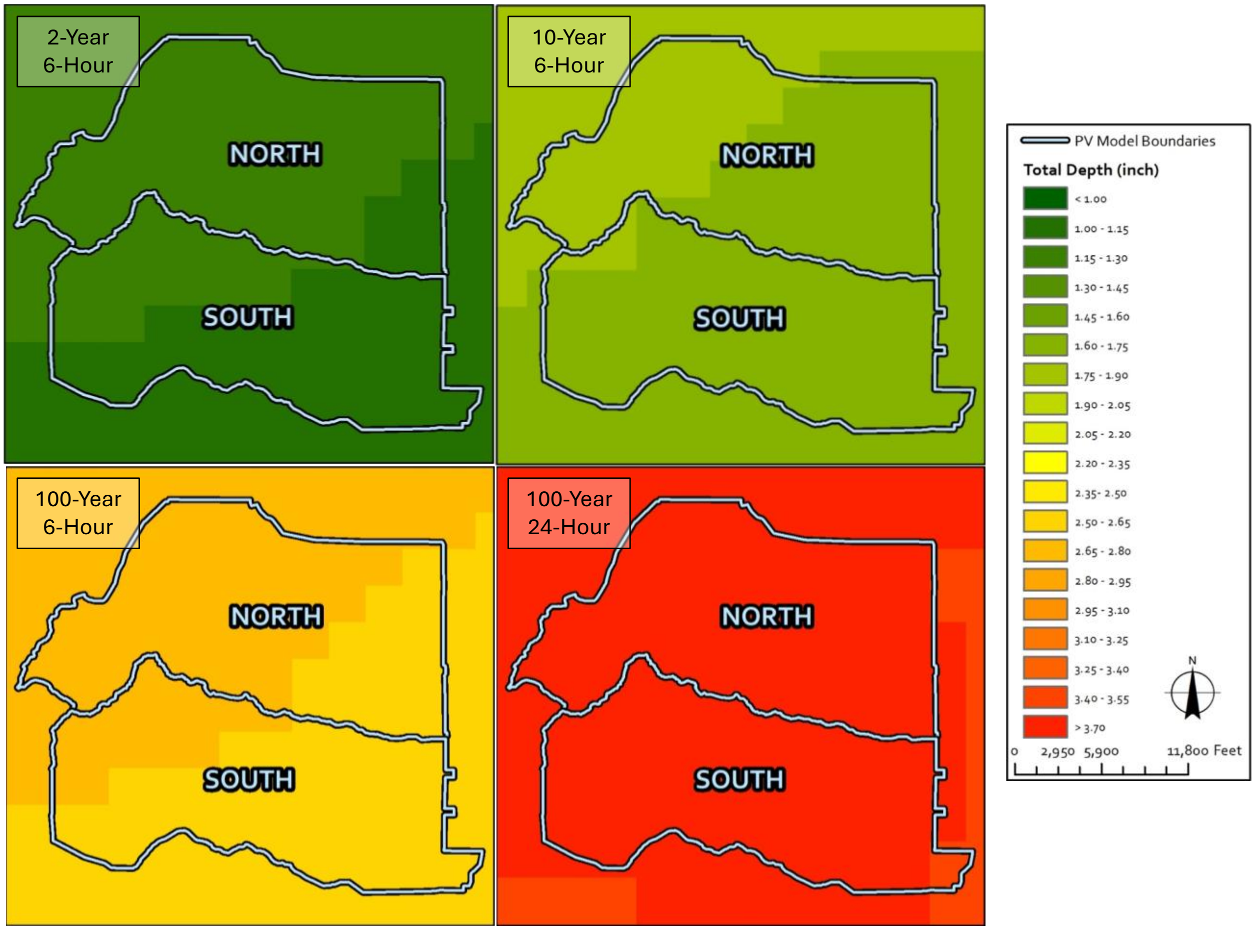


Figure 3-7. NOAA Atlas 14 Precipitation Datasets.

3.5.4 Infiltration

The INFIL.DAT file contains the infiltration parameters used in FLO-2D to calculate rainfall losses. The Green and Ampt method was used for this study as recommended by the District's Hydrology Manual. This method requires a number of inputs to be estimated from the land use and soil types within the study area.

Recent revisions to the District Hydrology Manual have simplified the approach to parameterizing the Green and Ampt infiltration model. Soils datasets provided by the District contain pre-computed Green-Ampt parameters, thereby eliminating the need to compute this dataset. Soil properties provided in this dataset used in the INFIL.DAT parameterization include hydraulic conductivity (XKSAT), rock outcrop (ROCKOUT), moisture deficit (DTHETA) for dry, normal, and saturated conditions, as well as capillary suction head (PSIF). Values by soil unit are shown below on in **Table 3-5**, and this coverage is shown on **Figure 3-8** below. The limiting depth variable was globally set at 4 inches for this submittal based upon District guidance (FCDMC, 2021[b]), and this value may be adjusted based upon a future model calibration phase. The second dataset used in developing INFIL.DAT is the SFC delineations. Properties of each land use used in the INFIL.DAT parameterization include initial abstraction (IA), percent impervious (RTIMP), development type (Type) and initial saturation condition (InitSat). Values by SFC category are shown below in **Table 3-6**.

The QGIS plugin directly computes the INFIL.DAT file for each sub-model simply by pointing the software to the two FCDMC shapefiles (SFC and soils).

Table 3-5. Soil Unit Infiltration Values.

Soil ID	Description	Rock Outcrop	XKSAT (2006)	PSIF (2006)	DTheta, Dry	DTheta, Normal	Limiting Depth
		[%]	[in / hr]	[in]			[ft]
AZ645109	Schenco-Rock outcrop complex 25 to 60 percent slopes	25	0.069	16.539	0.266	0.135	0.333
AZ64550	Estrella loams	0	0.254	14.129	0.298	0.167	0.333
AZ645113	Tremant gravelly loams	0	0.196	14.727	0.299	0.171	0.333
AZ64568	Gunsight-Cipriano complex 1 to 7 percent slopes	0	0.14	9.438	0.279	0.174	0.333
AZ64555	Gilman loams	0	0.121	16.899	0.263	0.132	0.333
AZ64598	Pinamt-Tremant complex 1 to 10 percent slopes	0	0.12	11.336	0.269	0.158	0.333
AZ655Es	Estrella loam	0	0.156	14.967	0.277	0.147	0.333
AZ655PvC	Pinamt very gravelly loam 3 to 5 percent slopes	0	0.072	16.7	0.267	0.137	0.333
AZ655Ro	Rock land	70	0.01	18.18	0.195	0.063	0.333
AZ655Va	Valencia sandy loam	0	0.65	4.603	0.315	0.235	0.333
AZ655LaA	Laveen loam 0 to 1 percent slopes	0	0.137	16.7	0.267	0.137	0.333
AZ655Ru	Rough broken land	70	0.01	18.18	0.195	0.063	0.333
AZ655CeC	Cavelt gravelly loam 1 to 5 percent slopes	0	0.125	15.714	0.276	0.149	0.333
AZ655Mv	Mohall loam MLRA 40	0	0.152	15.372	0.274	0.142	0.333
AZ655AoB	Antho gravelly sandy loam 1 to 3 percent slopes	0	0.577	4.631	0.331	0.248	0.333
AZ655LaB	Laveen loam 1 to 3 percent slopes	0	0.137	16.7	0.267	0.137	0.333
AZ655TrB	Tremant gravelly loam 1 to 3 percent slopes	0	0.087	10.015	0.234	0.133	0.333
AZ655Gm	Gilman loam	0	0.121	16.899	0.263	0.132	0.333
AZ655RiB	Rillito gravelly loam 1 to 3 percent slopes	0	0.219	14.461	0.303	0.176	0.333

Table 3-6. Infiltration Properties by SFC Category.

Class ID	Type Class	IA	RTIMP	Type	InitSat
		[in]	[%]		
0	Natural High Vegetation	0.10	0	Natural	Dry
3	Urban High Vegetation	0.10	0	Urban	Normal
4	Urban Medium Vegetation	0.10	0	Urban	Normal
5	Urban Low Vegetation	0.10	0	Urban	Normal
6	Mountain Bare Ground	0.25	0	Natural	Dry
8	Desert Rangeland Bare Ground	0.35	0	Natural	Dry
9	Urban Bare Ground	0.20	0	Urban	Dry
12	Wash Bottom	0.10	0	Natural	Dry
13	Concrete	0.05	98	Urban	Normal

Class ID	Type Class	IA [in]	RTIMP [%]	Type	InitSat
14	Asphalt	0.05	95	Urban	Normal
15	Buildings	0.05	95	Urban	Normal
16	Shade Structures	0.05	98	Urban	Normal
17	Water	0.00	100	Urban	Saturated
21	Unpaved Roads	0.10	50	Urban	Dry
23	Rock Riprap	0.25	95	Natural	Dry

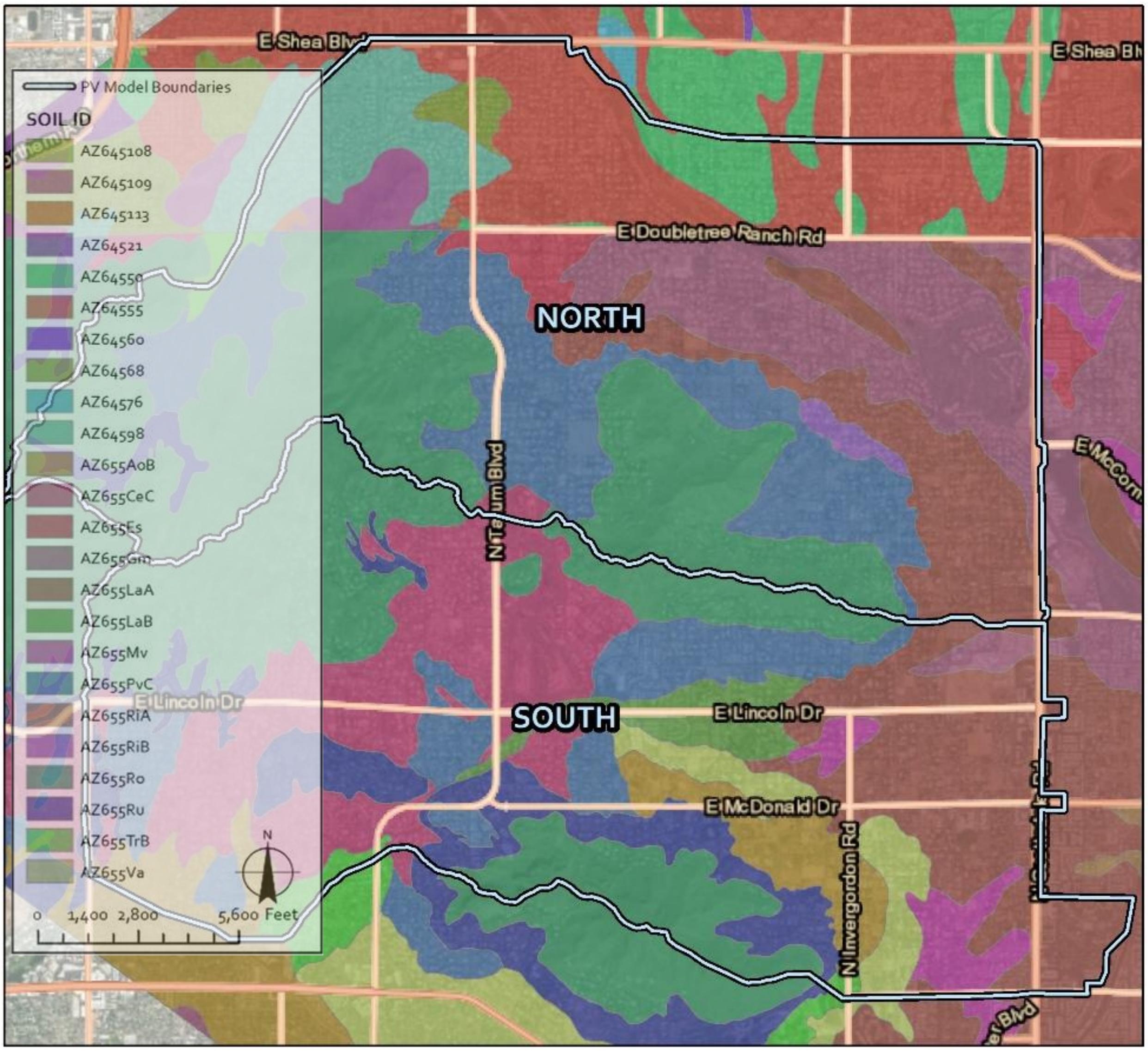


Figure 3-8. Soil Survey in Study Area.

3.5.5 Inflow and Outflow

Inflow hydrographs were used to transfer flow from an upstream sub-model to a downstream sub-model (namely, from the SOUTH sub-model to the NORTH sub-model). The inflow hydrographs are automatically written by FLO-2D during model execution of the upstream sub-models. The OUTCHAR field of the upstream OUTFLOW.DAT file was altered in order to capture flow moving out of the upstream model as well as organize the outflows into INFLOW.DAT files for the downstream sub-model. FLO-2D allows for a single sub-model to branch into up to nine separate sub-models, and there is no limit on how many sub-models can be collected into a single downstream sub-model.

3.5.6 Floodplain Cross-Sections

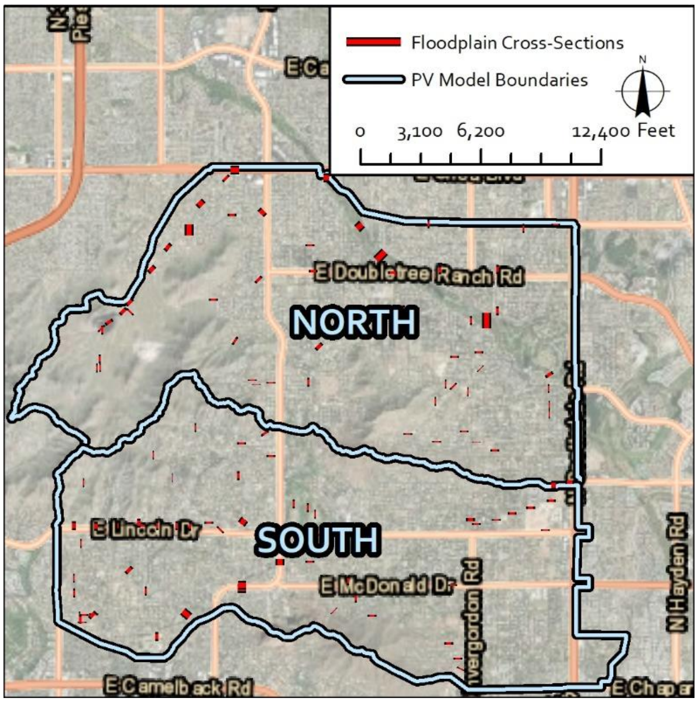


Figure 3-9. Floodplain Cross-Section Locations.

Floodplain cross-sections were developed and included in the FPXSEC.DAT file to query flow hydrographs and peak discharges from the FLO-2D model at key locations within the study area. A total of 121 floodplain cross-sections have been placed throughout the two model domains, and **Figure 3-9** visualizes the locations and density. Cross-sections were placed at locations of major flow concentration, flow transfer location between sub-models, areas of interest to the Town, and areas to assist in identifying model stability. Shapefiles containing information found in HYCROSS.OUT are provide for each sub-model including the peak discharge, volume, and id for each floodplain cross-section.

3.5.7 Blocked Obstructions

Blocked obstructions were included in the model to simulate the hydraulic effects of buildings on routing using the areal reduction factor (ARF) routine in FLO-2D. Building footprints were extracted from the general surface feature characterization shapefile from which a global area-weighted 10-foot blocked obstruction raster was created. This raster was then used to compute the percentage of area obstructed by buildings and assigned to area reduction factors (ARF) for each grid in both sub-models. Width Reduction Factors (WRFs) were not incorporated in the model development.

Further, the FLO-2D model will provide a revised ARF.DAT file (termed 'ARF.BAC') that rounds up ARF values to 1.0 when the prescribed value for a given cell is greater than 0.85. This is done automatically by the model to improve model stability. The revised .BAC file was then used in subsequent model runs. Also, where model boundaries cross ARF cells (i.e., buildings), outflow cells were removed to prevent conflicts between ARF, OUTFLOW, and INFLOW cells. However, if the ARF value for a given cell was less than 0.15, the ARF cell was removed, and the OUTFLOW cell was retained.

3.5.8 Property Walls

Walls and wall openings were not defined in this modeling effort, and the LEVEE.DAT input file was not included.

3.5.9 Culverts

Culverts were modeled using the HYSTRUC.DAT input file. A single shapefile (provided in the appendix) provides locations and attributes for all hydraulic structures in both sub models. Data from this shapefile was then directly used to automatically generate the HYSTRUC.DAT shapefiles for both sub models. This file is particularly useful when reviewing and modifying the hydraulic structures. A total of 241 culverts were included in this model with 280 separate hydraulic structure objects (**Figure 3-10**). This results where multiple objects may be used to simulate a single physical culvert, particularly when the culvert is wider than one FLO-2D model cell.

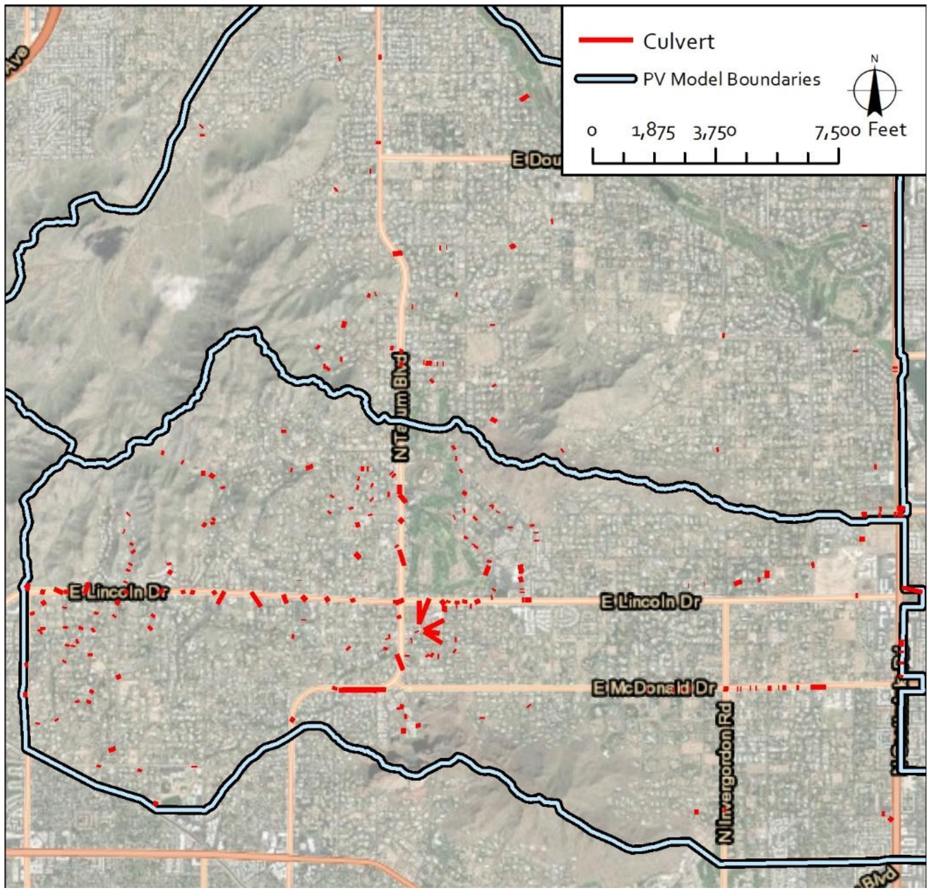


Figure 3-10. Hydraulic Structure Locations.

Hydraulic structures were included in each of the five contributing studies, and a proprietary process was developed to bring this data into the current study. In general, hydraulic structures from each study were mapped from local coordinates (i.e., cell centroids) to global coordinates (i.e., northing and easting) and then re-mapped to local coordinates of the current study. Most culverts imported this way were parameterized using rating tables, however some leveraged the generalized culvert equations. No modifications to imported culverts were made in this study with the exception of several structures where additional hydraulic structure objects were added to wider culverts to address the smaller cell size of this study. A significant portion of the total culvert count was developed in this study, and in all cases, new structures added to the model utilized the generalized culvert equations. Sources of data included the Town GIS database, aerial imagery, and field survey. Elevations for inlets and outlets were adjusted as necessary to reflect the elevations found in the high-resolution topographic dataset.

3.5.10 Storm Drains

3.5.10.1 Input Data Development

Storm drain model components were developed based on the storm drain systems modeled in previous studies (see **Table 2-1**), with additional storm drain systems along Indian Bend Road incorporated in this study due to their hydraulic significance. The storm drain systems were incorporated into the FLO-2D model through its SWMM module. The SWMM model input parameters for nodes (e.g., inlet type, inlet size, inlet maximum depth) and conduits (e.g., Manning's n-values, lengths, shape) were adopted directly from the previous studies for any storm drain systems modeled in those studies. The SWMM model input is documented in the SWMM.INP, SWMMFLORT.DAT, and SWMMOUTF.DAT files.

Since the previous studies were completed at different times and the SWMM module has undergone significant revisions over time, some minor modifications were made to the SWMM components in each ADMS before they could be combined into the final model. For example, the conduit offset input parameter in the MIBW model used an actual elevation rather than the offset from the node invert that was used in the other ADMSs. This was adjusted to use the offset to be consistent with other systems. Another example is that the original LIBW storm drains used sub catchments to denote inlets that were connected to the FLO-2D grid surface. Sub catchments became unnecessary in later versions of the SWMM module, so these were removed from the LIBW systems that were imported to the full PV SWMP model.

Additional systems along Indian Bend Road and Lincoln Drive that were not included in the previous ADMSs was developed based on data provided by KHA.

As a result, a total of 372 inlets and approximately 15 miles of storm drain conduits were modeled using SWMM and are shown in **Figure 3-11**. Detailed information on SWMM components (number of inlets, manholes, and outfalls and total length of pipes) by sub-models are summarized in **Table 3-7**.

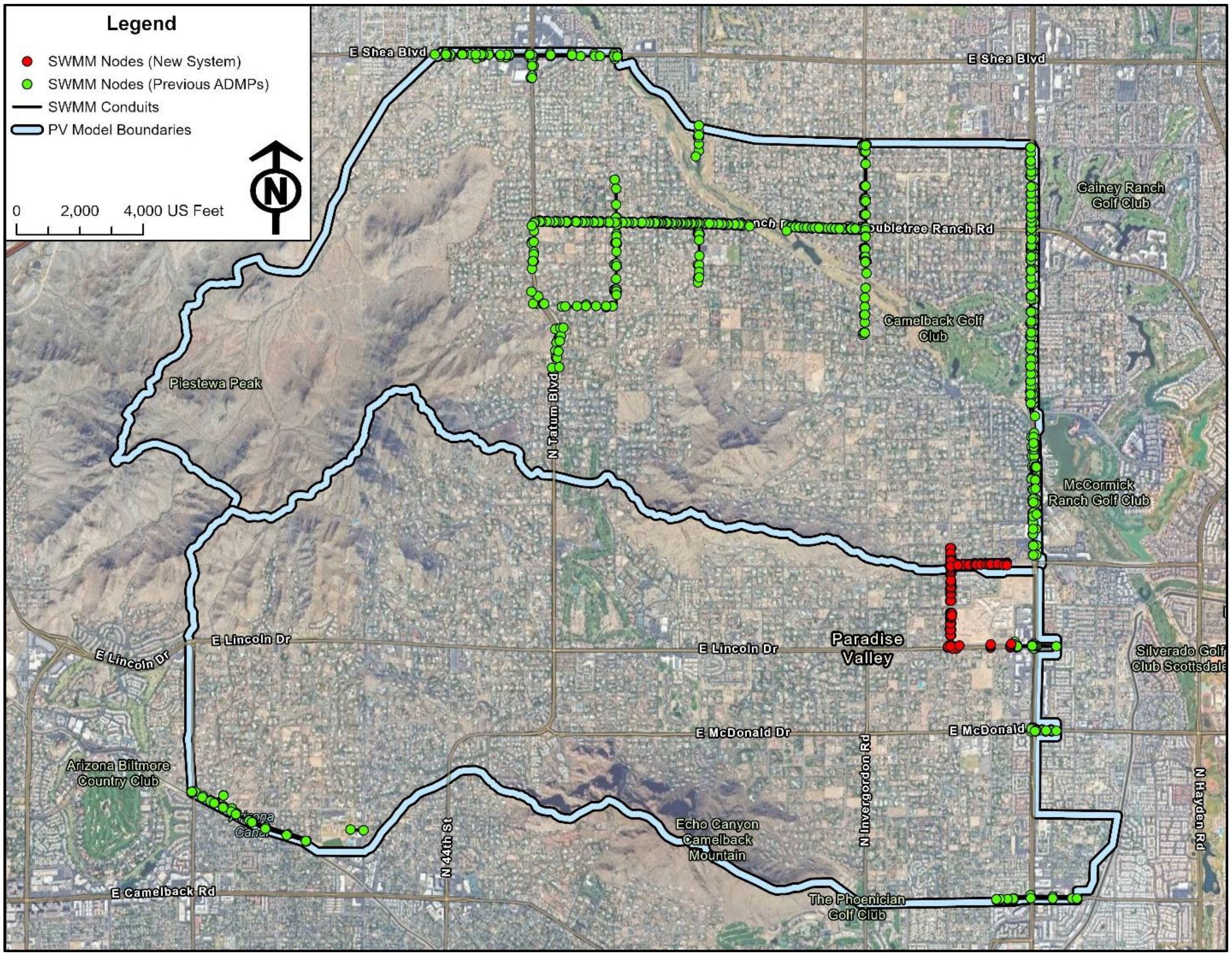


Figure 3-11. A Map of SWMM Components

Table 3-7. SWMM Component by FLO-2D Sub-Models

Sub-Model	Number of Inlets	Number of Manholes	Number of Outfalls	Total Length of Pipes (Mile)
North	296	6	14	11.6
South	76	20	11	2.9

3.5.10.2 Inlets

The curb opening inlets were modeled as Type 2 inlets (Curb Opening Inlet with Sag) with an 8-inch curb height, the minimum curb height described in Uniform Standard Details for Public Construction by Maricopa Association of Governments (2023) and a 0.5-ft of sag width. The sizes of inlets were obtained from the as-built drawings. If no information on inlet sizes is available in the as-built drawings, measurements were taken using Google street view and ArcMap. For the curb-opening inlets, the weir coefficient of 2.3 was used as suggested in the FLO-2D Storm Drain Manual (FLO-2D Software, Inc., 2021).

Grate inlets are modeled as Type 3 inlets, which are defined as grate (gutter) inlets. For the FLO-2D model inputs, the perimeter and area for the grates are based on as-built drawings. It should be noted that since these grates are not against a curb and are typically within depressed swales, all four sides of the grate configuration were included in the grate perimeter. The weir coefficient of 3.0 was used as recommended in the FLO-2D Storm Drain Manual for the grate inlets.

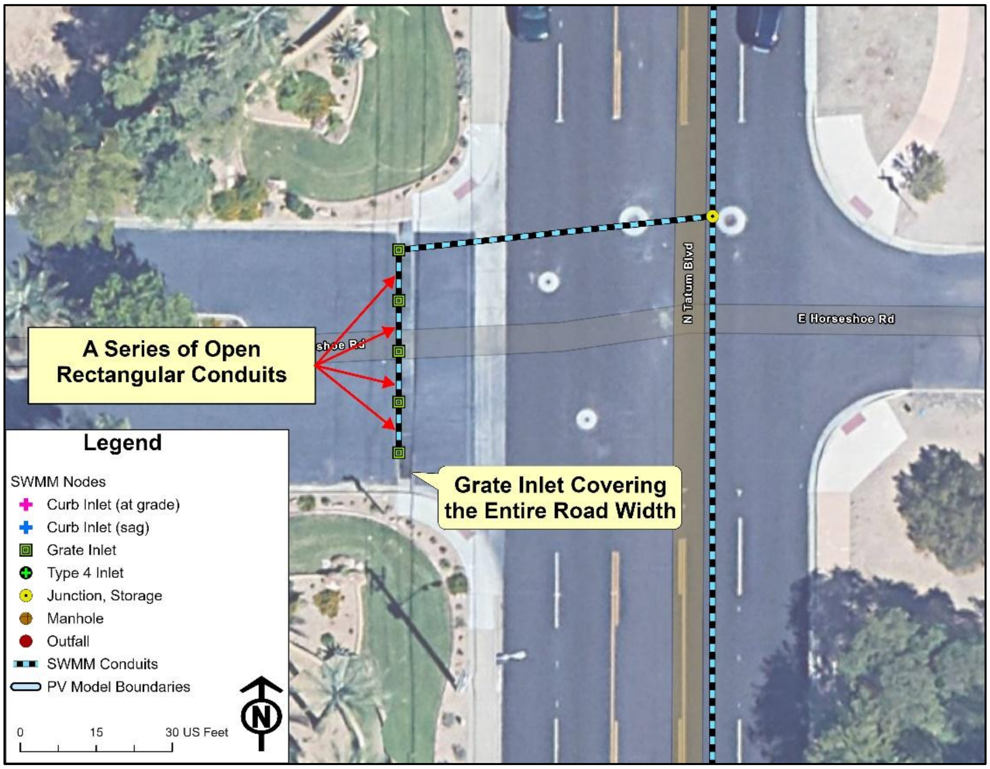


Figure 3-12. Grate Inlet Modelled with a Series of Open Rectangular Conduits at East Horseshoe Road and North Tatum boulevard

Several grate inlets could not be accurately modeled using the 10-ft FLO-2D grid because their lengths were much greater than the grid size, as shown in **Figure 3-12**. To address this, the length of the grate inlet was divided by the number of grid cells it intersected, and this adjusted length was assigned to each grid cell. The grate inlets were then represented as a series of open rectangular conduits, with depths matching the maximum depth of the grate inlets and lengths equal to the grid size of 10 ft.

Manholes can act as inlets in FLO-2D once the manhole lid is popped. While most manholes are called out as having a solid lid in the plans, several manholes are called out as having grated lids. These grated lids have a surcharge depth set to 0.01-ft so that flow can enter/exit with minimal surcharge. The remaining manholes with solid lids have a surcharge depth of 0.69-ft which is based on a manhole lid weight of 210-lbs per MAG detail 423-2, manhole diameter of 2.5-ft, and the surcharge depth equation per the FLO-2D Storm Drain Reference Manual.

Junctions were modeled in FLO-2D SWMM where lateral pipes connect directly to the storm drain main. These locations are simply junctions and do not receive or deliver flow to the FLO-2D surface grid.

Vertical opening inlets with a culvert entrance can be modeled using an inflow discharge defined by a rating table and are modeled as Type 4 inlets, which are characterized as inlets with variable geometry. A rating curve for each inlet must be supplied for Type 4 inlets. The rating tables for these inlets were created using the inlet control equation described in Hydraulic Design of Highway Culverts report by U.S. Department of Transportation Federal Highway Administration (Schall, Thompson, Zerges, Kilgore, & Morris, 2012).

Combination inlets usually consist of a curb opening inlet with a grate inlet, although other combinations are possible. Within the study area, multiple combination inlets were observed and modeled as Type 4 rating table in the FLO-2D model. The rating tables for the combination grate/curb inlets were based on calculations with the weir and orifice equations similar to how a typical curb (or grate) inlet is calculated in the FLO-2D SWMM component. The maximum weir depth for when the weir equation is calculated based on the geometry of the inlet. Similarly, the minimum depth for when the orifice equation controls is calculated based on the inlet geometry, and the discharges between these two depths are calculated based on linear interpolation. The grate and curb discharges are calculated separately, and the two sets of discharges are combined into one rating table which is applied to the combination inlet as a Type 4 inlet in the FLO-2D SWMM model.

3.5.10.3 Conduits

Information on the storm drain conduits (e.g., diameter, shape, length, material, and invert elevations) was obtained from the previous studies and incorporated into the existing FLO-2D SWMM.INP input file. Per the recommendations in the FLO-2D Storm Drain Manual, it is advised to have a minimum pipe length equal to or greater than the size of the FLO-2D grid element side. This precaution helps prevent excessive volume conservation errors. Consequently, a minimum pipe length of 10 ft was implemented for this purpose in this study.

The additional storm drain system along Indian Bend Road primarily consists of rubber-gasketed reinforced concrete pipes (RGRCP), with a few corrugated metal squash pipes. For these conduits, a Manning's n value of 0.012 was used.

3.5.10.4 Outfalls

Outfall in the FLO-2D SWMM component refers to a point where main storm drain flow is discharged from the stormwater network back to the FLO-2D grid surface. Outfalls are located in natural/manmade channels, reservoirs, basins, or underground storage. In the study area, several underground outfalls that have a connection to drywells and the ground surface through a grate were identified. The FLO-2D cell elevation was adjusted to match the invert elevation of the storm drain outfall. Therefore, the storm drain discharged to the adjusted cell, the water ponded in that cell and overflowed into surrounding cells once the ponded depth reached a sufficient level.

3.5.11 One Dimensional Channels

The one-dimensional (1D) Channel option was applied to locations deemed hydraulically significant or where channel capacity could not be adequately captured by the FLO-2D grid cell size and elevation interpolation. The modeled segments include 3,600 feet in the North model, 2,700 feet in the South model, with a total of approximately 6,000 feet in length.

The 1D channel geometry incorporates natural cross-sections sampled from topographic data (**Section 2.2**). These channels were modeled as 1D channels in earlier studies, and the Manning's n values for the channel segments were derived from those studies. The north channel segment is an engineered, concrete-lined channel with Manning's n values ranging from 0.016 to 0.03. In contrast, the south channel segment is a natural channel, where Manning's n values range from 0.02 to 0.076. The 1D channels are shown in **Figure 3-13**.

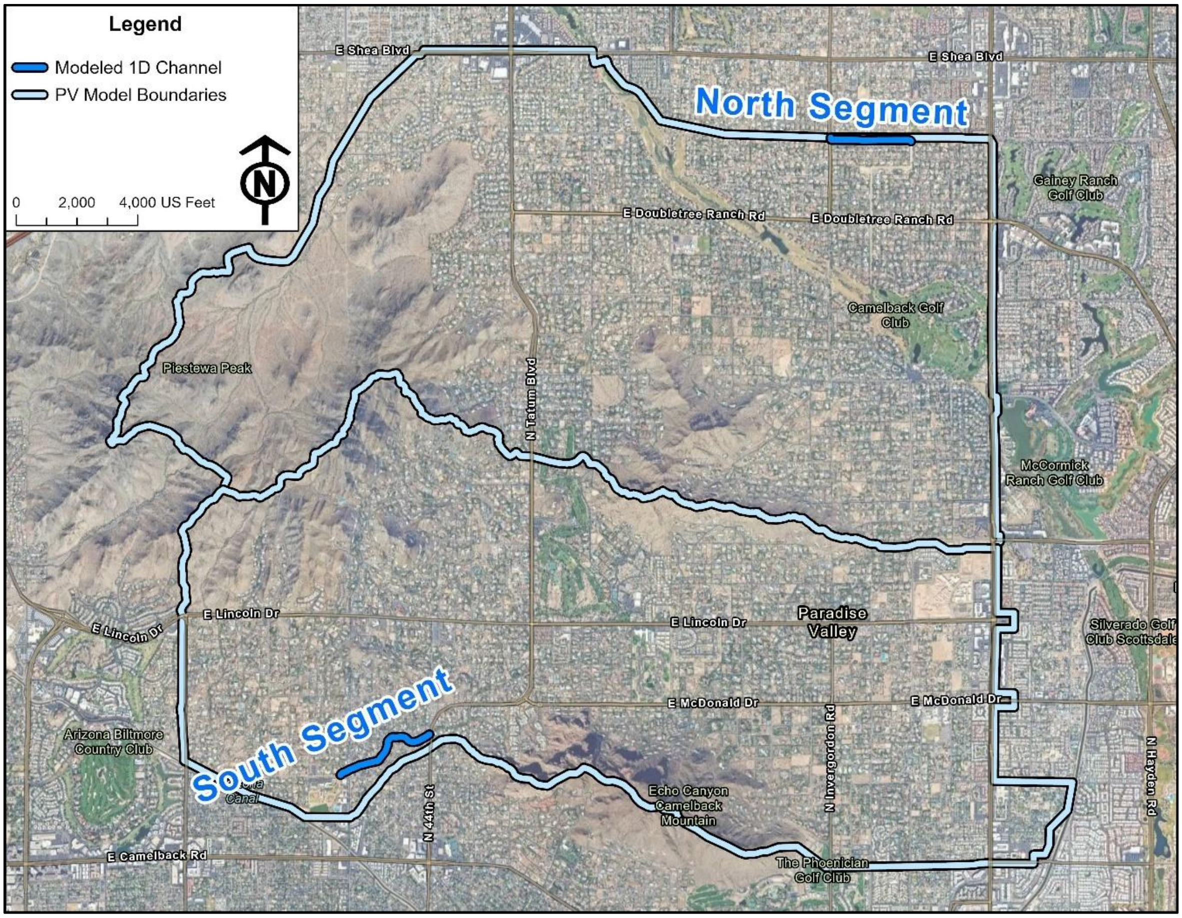


Figure 3-13. 1D Channels

3.5.12 Model Control Files

A model runtime of 12 and 18 hours was used for the 6-hour and 24-hour durations, respectively. This allowed for adequate time to capture the peak of the hydrograph as observed in HYCROSS.OUT. Further iterations and refinement of this modeling may require a longer simulation time. A global limiting Froude number (FROUDL) of 0.95 was set in CONT.DAT. A TOLGLOBAL value of 0.004 feet was used per District guidelines, and this value was subtracted from the initial abstraction value in INFIL.DAT. The floodplain Courant number was set to 0.60, and the TIMEACCEL parameter was set to 0.1. Both DEPTOL and WAVEMAX were set to 0.0.

3.5.13 Special Modeling Considerations and Solutions

There are no special modeling considerations in the model at this stage of model development.

3.6 Model Warnings and Error Messages

The following warnings and error messages are reported in the FLO-2D output file ERROR.CHK. Some messages are repeated multiple times if the applicable situation occurs multiple times during a simulation. For example, the warning message appears quite often that the downstream water surface is higher than the upstream water surface for a hydraulic structure. Each warning is only discussed once in the list below. The warnings that were reported for both final sub-model runs include the following:

- 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 imperceptible in the model results.
- *** THERE ARE DRY OUTFLOW NODES FOR THE FOLLOWING DOWNSTREAM GRID SYSTEM: 1*** GRID CELL: XXXXXX ***
    - 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
- WARNING: THE HYDRAULIC STRUCTURE OUTFLOW NODE: XXXXXXX IS REPEATED MORE THAN ONCE WITHOUT ASSIGNING A D-LINE CONVEYANCE CAPACITY LIMITATION. EITHER REVISE THE OUTFLOW NODES OR ADD A D-LINE
    - This is a valid assumption, as in these cases there are adjacent outlets.
- NOTE: THE HYDRAULIC STRUCTURE LOCATED AT THE END OF THE CHANNEL SEGMENT ELIMINATES THE CHANNEL ELEMENT: 2219312 FROM HAVING OUTFLOW TO THE FLOODPLAIN AT END OF THE CHANNEL SEGMENT. ALL THE CHANNEL FLOW WILL ENTER THE CULVERT
    - This is understood to be a limitation of the model, and it was assumed that the structure was appropriately sized for the channel. This approach mimics how the previous study parameterized this location.
- REVIEW THE EVACUATEDCHAN.OUT FILE FOR COMPLETE EVACUATION OF VOLUME IN THE LISTED CHANNEL ELEMENTS - IMPROVE ROUTING STABILITY BY REDUCING THE OUTFLOW FROM THE CHANNEL ELEMENT
    - The EVACUATEDCHAN.OUT file only lists one node, and no obvious channel instabilities were detected.
- NOTE: THE FOLLOWING INTERIOR CHANNEL ELEMENT ELEVATIONS WERE RESET TO THE CHANNEL BED ELEVATION FOR THE UPSTREAM OR DOWNSTREAM END OF THE CHANNEL SEGMENT TO EXCHANGE FLOW BETWEEN THE FLOODPLAIN AND CHANNEL TERMINUS. THE FLOODPLAIN GRID ELEMENT ELEVATIONS UPSTREAM OR DOWNSTREAM OF THE END OF THE CHANNEL SHOULD ALSO BE ADJUSTED TO ALLOW FLOW INTO THE CHANNEL OR FROM THE CHANNEL BASED ON THE CHANNEL BED ELEVATION (SEE PROFILES PROGRAM FOR BED ELEVATION):
    - Checks were made to ensure a smooth transition between the floodplain the channel.
- WARNING: THE FOLLOWING HYDRAULIC STRUCTURES HAVE ARF VALUES IN EITHER THE INLET OR OUTLET ELEMENTS: (THIS WOULD ONLY BE A PROBLEM IF THE REMAINING SURFACE AREA WAS RELATIVELY SMALL (< 50%).)
    - This error was printed in the CHK file, but no nodes were identified.

4 MODEL RESULTS

4.1 Overview

Runoff in the study area generally flows to the northeast in the NORTH model, except for the northeastern corner on the north side of IBW, where runoff flows to the south into IBW. The western half of the SOUTH sub-model flows to the southwest towards the Arizona Canal Diversion Channel (ACDC), while the eastern half flows toward IBW. The valley floor is marked by extensive amounts of shallow, distributary flow.

4.2 Hydrologic and Hydraulic Results

The digital deliverable includes rasterized datasets for model inputs and outputs, and a brief overview of the general flow paths can be seen in the 100-year 6-hour results on **Figure 4-1** below.

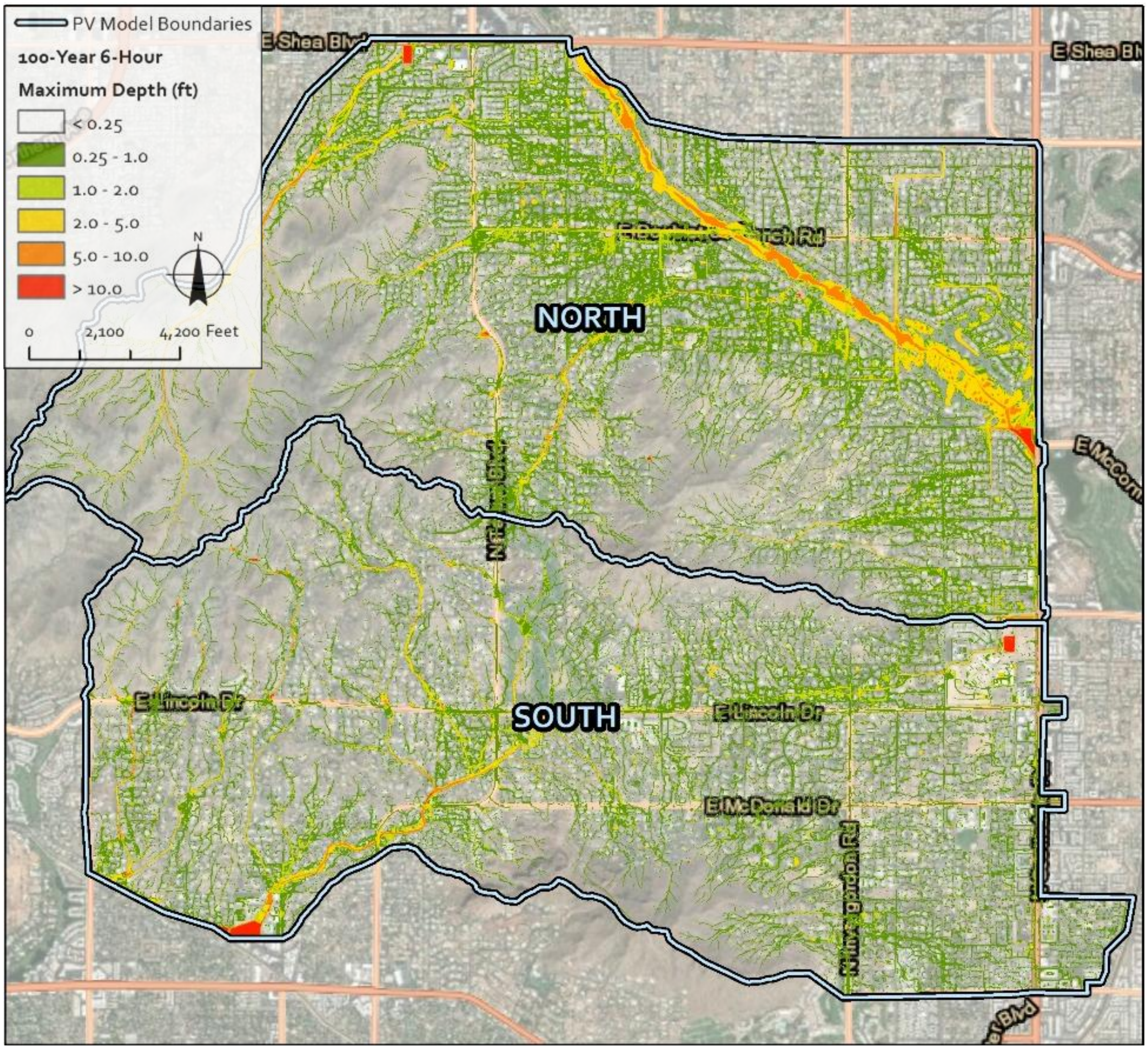


Figure 4-1. Preliminary Maximum Depth Results for the 100-Year 6-Hour Event.

4.3 Controlling Duration

An analysis was performed to determine the controlling duration to use for the remainder of this study. Both the 6-hour and 24-hour durations were simulated at the 100-Year recurrence interval, and a comparison of some results for peak discharge from these simulations was performed to determine which duration generally controls. It should be noted that this comparison was made early in the study, prior to the development of more detailed modeling features (e.g., culverts, storm drains, etc.). The comparisons of discharges were generally made upstream of significant hydraulic features.

Peak discharges from identical locations and from both durations were plotted against each other on **Figure 4-2** below. Values that fall below the 1:1 indicate points where the 6-hour duration yields a higher flow rate, and values that fall above the line represent points where the 24-hour result is greater. The comparison points span a wide range of small to large watersheds in the study area, and it also includes several points along Indian Bend Wash. Nearly all comparison points indicate the 6-hour event yields a higher peak flow rate. The comparison on larger, axial watercourses (namely Indian Bend Wash) suggests a 24-hour controlling duration, and this is expected given that the watershed size is much larger than the other comparison points. Given that the purpose of the overall SWMP study is to identify drainage solutions throughout the Town (i.e., focusing on smaller drainage areas and less on IBW), the 6-hour duration was selected as the controlling duration. The 2-year and 10-year recurrence intervals were then analyzed solely with this duration.

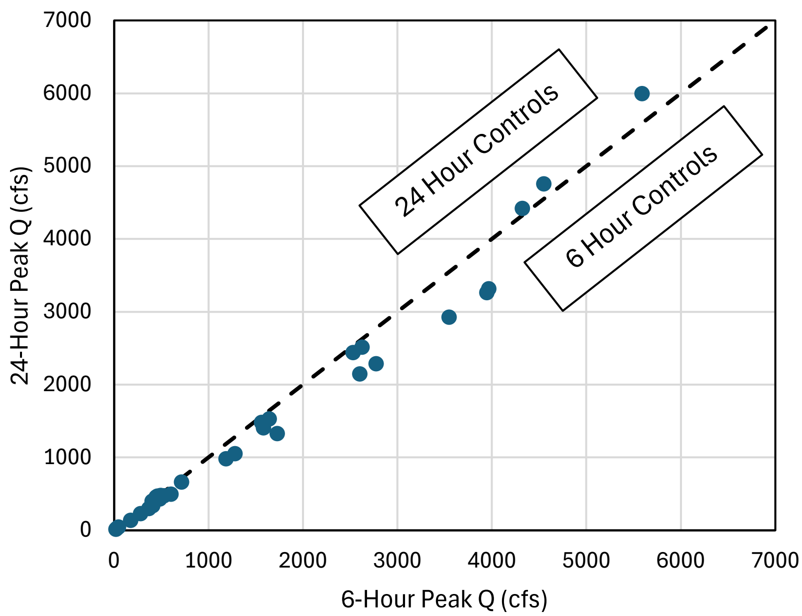


Figure 4-2. 6-Hour vs. 24-Hour Peak Discharge Comparison.

4.4 Infiltration Loss Summary

The loss due to infiltration of the rainfall volume is summarized in **Table 4-1**. As expected, the loss percentage due to infiltration is higher for lower-magnitude events. The loss rates seen below are considered reasonable based upon experience in similar watersheds.

Table 4-1. Infiltration Loss Summary.

Duration	Recurrence Interval	Sub-Model	Maximum Point Precipitation [in]	Precipitation Volume [ac-ft]	Inflow Volume [ac-ft]	Infiltration and Interception Volume [ac-ft]	Loss Percentage (of Precipitation Only)
6-Hour	100-Year	SOUTH	2.691	1,483	0	332	22.4%
		NORTH	2.691	1,598	1,809	409	25.6%
	10-Year	SOUTH	1.766	972	0	327	33.6%
		NORTH	1.766	1,049	811	400	38.1%
	2-Year	SOUTH	1.176	647	0	308	47.6%
		NORTH	1.176	698	437	370	53.0%

5 MODEL CALIBRATION AND VERIFICATION

This section documents the calibration efforts that were performed. Thereafter the final results are placed in the context of other studies and hydrologic approaches.

5.1 Calibration

The model calibration effort focused on gage data that was collected by the District at the Tatum Basin Inflow (ID # 58007) flood warning stream gage. This gaging station is located just upstream of the Tatum Basin within Tatum Wash, and it was installed in 1994 and has been continuously active and maintained since. This station consists of a pressure transducer sensor mounted to the vertical concrete channel wall, and the station uses the ALERT2 telemetry protocol for real-time data reporting. The contributing drainage area at this location is 2.17 mi^2. This gaging location is shown in **Figure 5-1**.

Figure 5-1. Tatum Basin Inflow Gage. Source: FCDMC ALERT Website.

The location of the gage in relation to the contributing drainage area and the study model boundaries is shown below on **Figure 5-4**.

5.1.1 Historical Flow Events

As of December 2024, a total of thirty-six flow events have been recorded by this gaging station ranging in peak discharges between 8 cfs and 1,688 cfs. Three of these events were chosen to include in the calibration effort for this study:

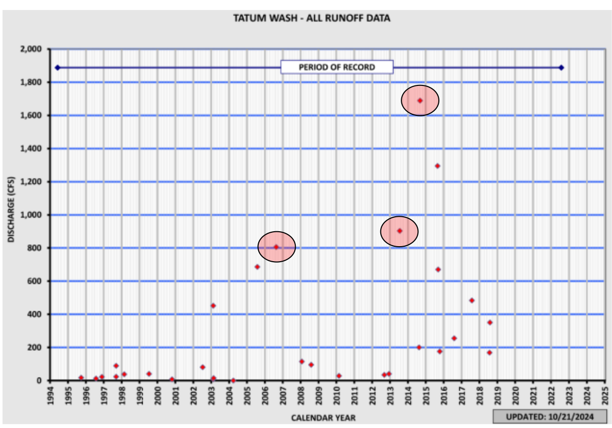


Figure 5-2. Runoff Event History. Source: FCDMC ALERT Website.

- August 24, 2006 808 cfs
- July 21, 2013 903 cfs
- September 8, 2014 1,688 cfs

Figure 5-2 to the left shows the magnitude of peak flow rates of thirty-six events, and the three used in this study are circled. The 2014 event was a notable flooding event in the Paradise Valley area, with extreme runoff resulting in significant Town-wide flooding issues. The other two events were selected as they represent lower-magnitude runoff events that may better depict hydrologic conditions used in the sizing of stormwater infrastructure. The precipitation recurrence interval for each of the three events was determined using NOAA Atlas 14 precipitation data for the calibration watershed (statistics obtained with coordinates of 33.5547°, -112.0052°). Spatially-distributed precipitation datasets on a five-minute interval for each event were provided by the District. Four locations were selected within the calibration watershed that generally span the extent of the basin to extract precipitation hyetographs. This data was then plotted for each event, averaged across the four locations, and the approximate precipitation duration was determined (**Figure 5-3**). Using the average depth and duration, a precipitation recurrence interval was then calculated for the three events (**Table 5-1**).

Both the 2006 and 2013 events were similar in magnitude and duration and were less than a 10-year event. The 2014 event exhibited significantly higher precipitation and subsequently was calculated as a 750-year precipitation event for this basin. Lastly, it should be noted that precipitation recurrence interval is not necessarily tied to runoff recurrence interval, as carrying hydrologic methods (e.g., loss methods, routing options, software, etc.) can yield significantly different flow rates.

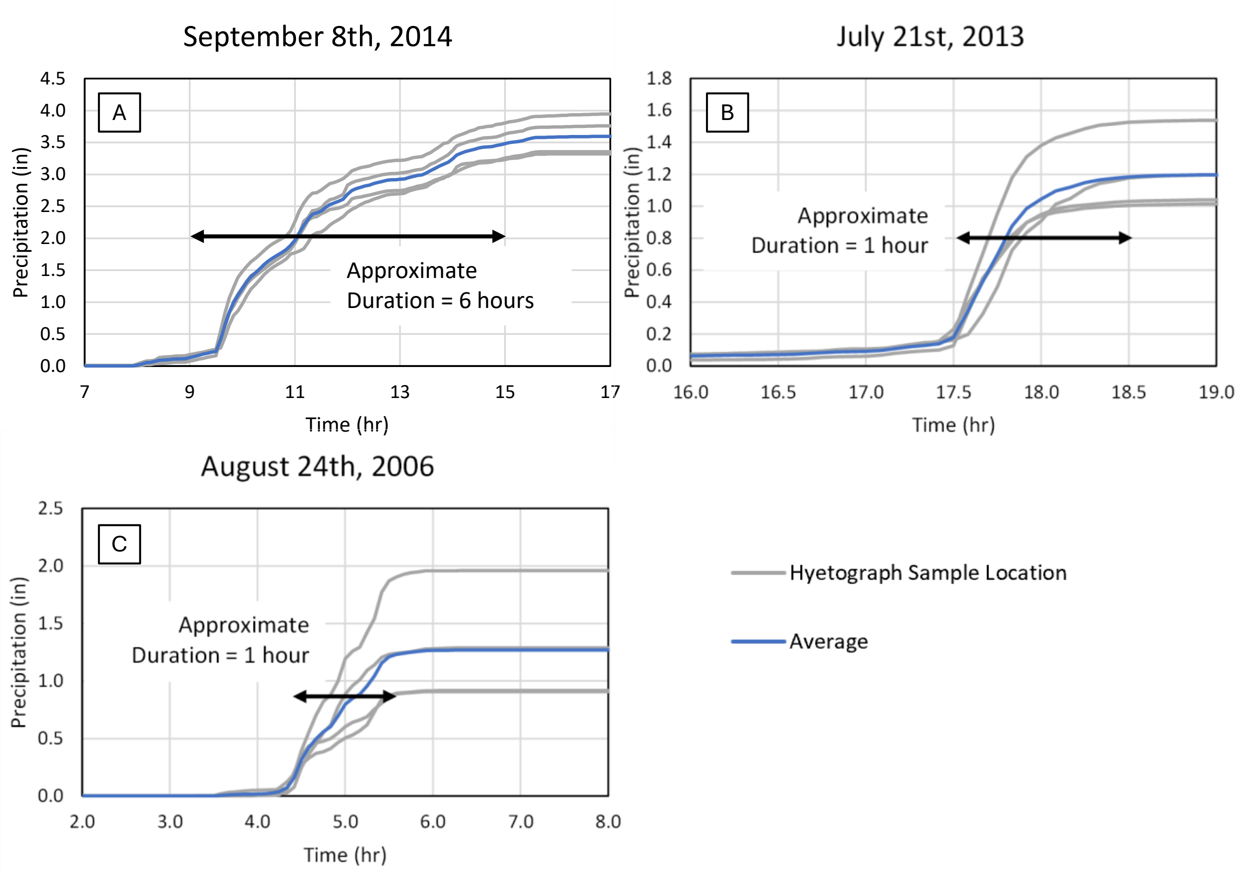


Figure 5-3. Event Hyetographs for (A) September 8th, 2014, (B) July 21st, 2013, and (C) August 24th, 2006.

Table 5-1. Precipitation Recurrence Interval of Calibration Events.

Event	Average Total Depth (in)[a]	Approximate Duration (hour)	Precipitation Recurrence Interval (year)	Maximum Measured Flow Rate (cfs)
August 24th, 2006	1.149	1	6.6	808
July 21st, 2013	1.127	1	6.1	903
September 8th, 2014	3.540	6	750	1,688
[a]*Averaged across Tatum Wash watershed*				

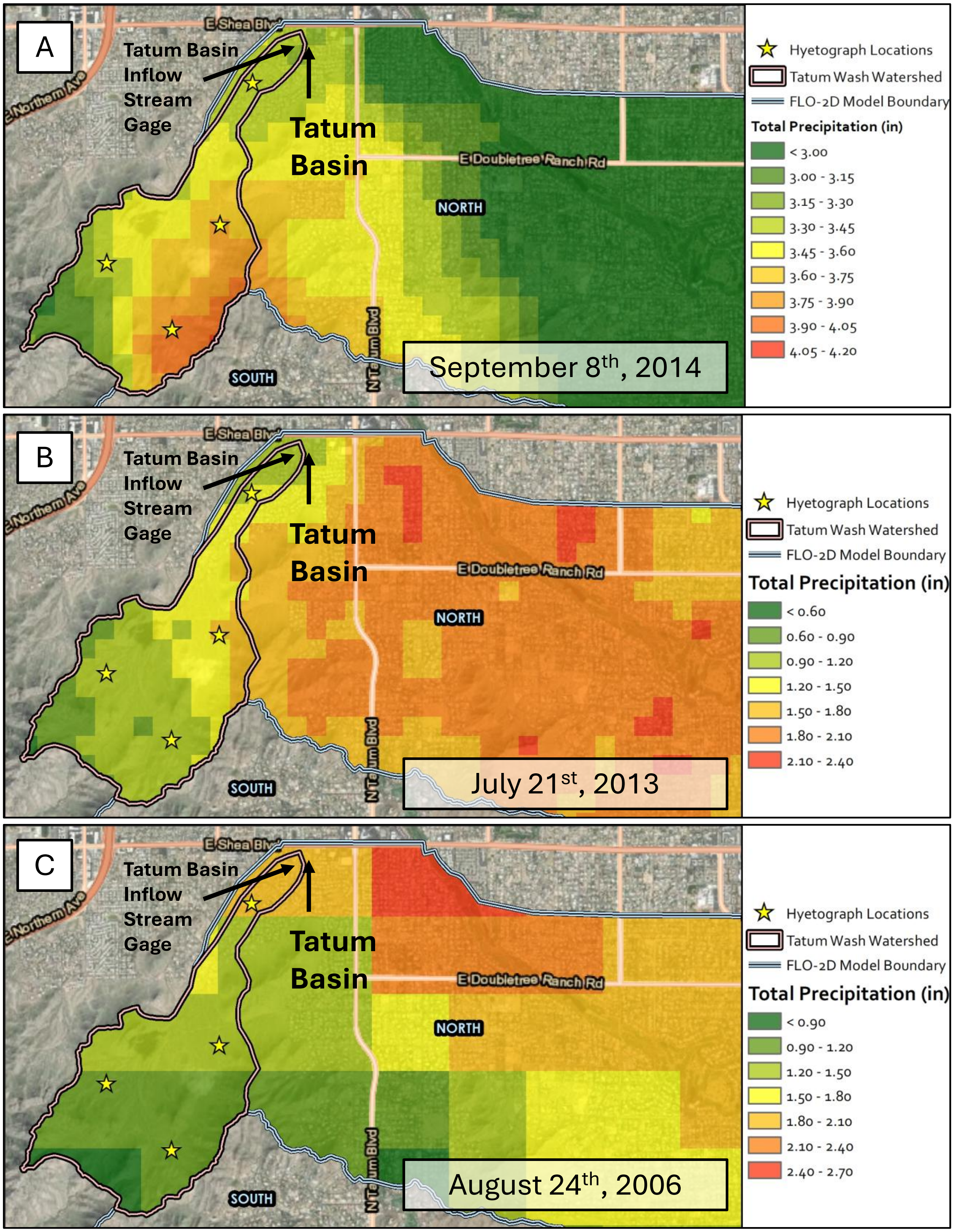


Figure 5-4. Calibration Precipitation Events. (A) September 8th, 2014, (B) July 21st, 2023, and (C) August 24th, 2006.

5.1.2 Calibration Process

The input data provided by the District was processed into RAINCELL.DAT FLO-2D input files using the QGIS pre-processor application. This input file describes precipitation input in both spatially (cell by cell) and temporal dimensions. During the development and testing of this input file it was noted that the District-approved FLO-2D executable (Build 23.10.25, Compiled on 11/9/2023) was not correctly reading the RAINCELL.DAT or RAINCELL.HDF file. JE Fuller coordinated with the District to receive guidance on how to proceed. An interim executable described as a re-compiled version of the same source code (Compiled on 11/5/2024) as the approved version was provided to JE Fuller, however the District noted that this executable exhibited other instabilities un-related to this specific application. A comparison between the 2023 and 2024 executables yielded the same peak discharge and timing at Tatum Basin. Given this, the following work plan was developed between JE Fuller and the District for this study:

1. Compare base results for the 2023 and 2024 executable.
2. Utilize the 2024 executable for calibration runs, including adjusting input parameters.
3. Using the input parameterization from the final, selected calibration scenario, re-run the design storm models (i.e., 2-, 10, and 100-year events) with the approved 2023 version.

Hydrographs at the gaging location just upstream of the inlet to Tatum Basin were compared between the actual, gage data and the base model results. The three events are compared below on **Figure 5-5**. A similar trend is seen in all three scenarios, where the modeled peak flow rate is less than the gage data and the flood wave arrival time is longer than the gage data as well. Further, the modeled volume for the 2014 event appeared to be larger as well.

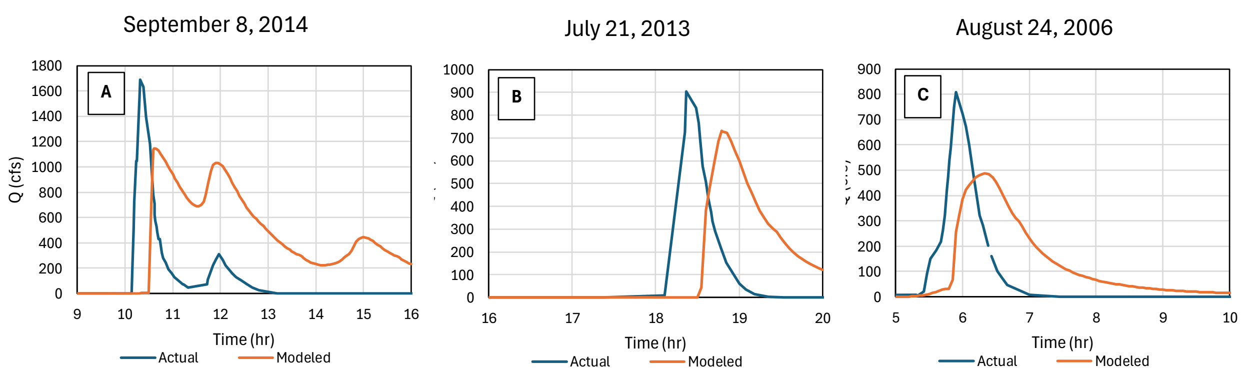


Figure 5-5. Initial Hydrograph Comparisons for the (A) September 8, 2014, (B) July 21, 2013, and (C) August 24, 2006 events.

5.1.2.1 *Scenarios*

A range of scenarios were developed to understand the sensitivity of various input parameters to the modeled peak discharge, timing, and volume (**Table 5-2**) and reflect ranges of adjustments that are typically made for hydrologic studies. This exercise was performed with the 2006 event only, as it (along with the 2013 event) are closer in magnitude to what it is typically used to size drainage infrastructure as compared to the 2014 event.

In general, adjustments to infiltration (hydraulic conductivity, initial abstraction, and limiting depth) and roughness (base Manning's n and shallow Manning's n) were made. The first scenario involved simply adjusting the global porosity setting to 0.4, however it was concluded afterwards that this value should be 0.0 when using spatially varied parameterization of INFIL.DAT, and when used in this manner it simply increases the DTHETA value.

The scenarios shown below were applied to all cells in the NORTH model. Global modifications were made as this initial step in the calibration process is simply much more effeicient than modifying based upon soil type and land use classification. The purpose of this initial step was to identify which parameters can be adjusted to match the gage data and to provide a more general view of the response of the model to various adjustments. Detailed calibration adjustments then follow this preliminary step. The storm drain component was turned off for all calibration-related simulations, as the purpose of this was limited to the Tatum Wash sub-basin only, and no storm drains are hydraulically upstream from the comparison point.

Table 5-2. Calibration Scenarios for 2006 Event.

Run	Description
S1	Set global porosity to 0.40
S2	50% of base for XKSAT, liming depth = 8"
S3	50% of base for XKSAT, liming depth = 16"
S4	75% of base for XKSAT, liming depth = 16"
S5	50% of IA
S6	50% of IA, 50% for XKSAT, and limiting depth = 8"
S7	50% of Shallow n
S8	50% of Manning N
S9	50% of Shallow n and Manning N
S10	Turn off Shallow n
S11	200% of XKSAT, limiting depth = 16", 50% of IA, 33% of Manning N, turn off Shallow n
S12	67% Shallow n, 1.25 IA, 60% Manning n, limiting depth = 16"
S13	60% of Shallow n, 75% of base n values, 80% of XKSAT, and 110% of IA, limiting depth = 16"

5.1.2.2 Hydrograph Comparisons

The resulting hydrograph at the Tatum Basin inlet for all scenarios is shown below on **Figure 5-6**. The gage data is also displayed along with the 'base', or un-calibrated result. One of the closer matches is from the S1 scenario, as it matched the arrival time well and provided a notable improvement in peak discharge (in terms of matching the gage data), however as noted above this is not a realistic parameterization. It does, however, suggest that an increase in DTHETA could yield closer results. DTHETA was not adjusted in this calibration exercise. The results from the remaining scenarios demonstrate that the arrival time both the arrival time and peak discharge from the model can match the gage data, however not in a simultaneous parameterization. Further, the S11 scenario, which was the only simulation where the peak was close to the modeled peak, represents an extreme parameterization and it required very large adjustments to several variables and is far more of an adjustment that what is typically used in calibration exercises.

In all but S11, the scenario results tend to exhibit lower peak discharge and higher volume. No reasonable combination of variable adjustments yielded agreement with the gage data in terms of timing, peak discharge, and volume comparisons.

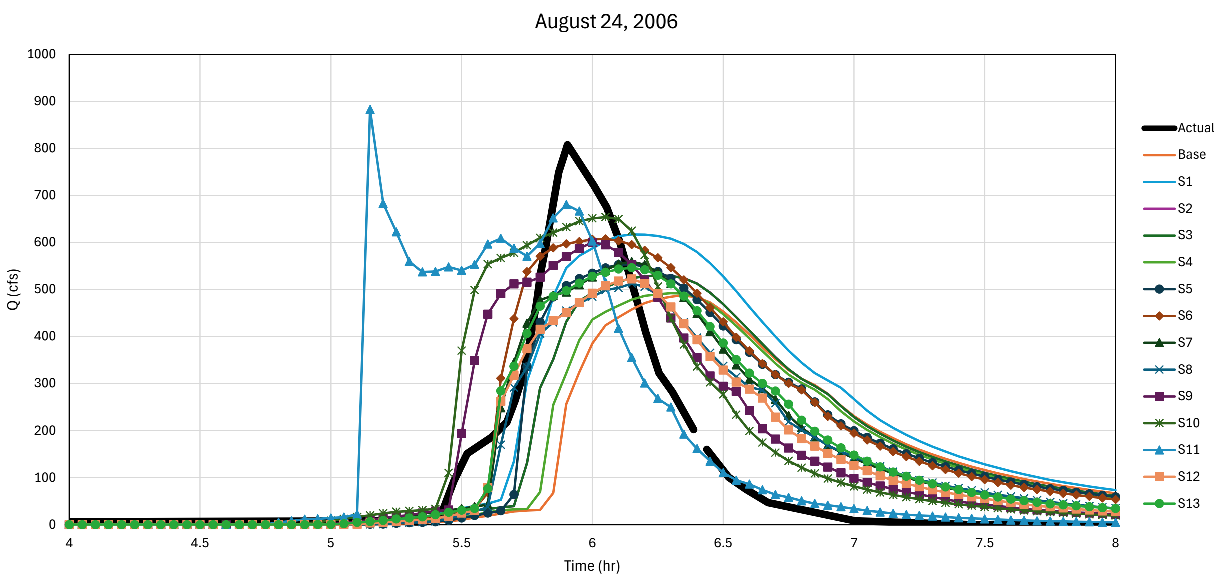


Figure 5-6. Calibration Iterations for 2006 event.

5.1.3 USGS Regression Analysis

Flow rate estimates using the USGS Regional Regression approach (USGS, 2014) were computed for the Tatum Basin. A basin area of 2.17 mi^2, mean basin elevation of 1,705 feet, and mean annual precipitation of 9.17 inches was used. Precipitation was determined using the PRISM dataset (PRISM, 2025). Regression equations for various recurrence intervals were developed in the USGS (2014) study, and results for the 2-, 10-, and 100-year recurrence intervals are presented in **Table 5-3**.

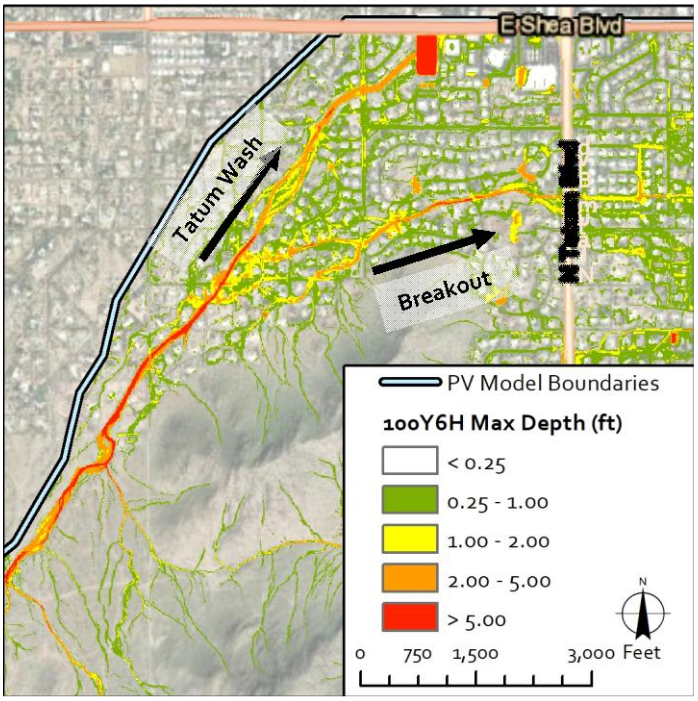


Figure 5-7. Tatum Wash Breakout.

It was observed in the preliminary (i.e., base) model results that a significant breakout to the east occurs just upstream of 44th Street (**Figure 5-7**), and the presence of this breakout in flow was corroborated by Town staff. The model results suggest that flow begins to break out between the 2-year and 10-year events. Therefore, comparisons between model and regression equation results need to include the breakout hydrograph for the modeled results.

Model results presented in **Table 5-3** include both the Tatum Basin inlet as well as the peak of the sum of the inlet and breakout hydrographs. This provides a more one-to-one comparison to the regression results. Inversely to the gage data comparison, the model results are significantly higher than the regression equations with the 100-year modeled results being nearly double of the regression equation results.

Table 5-3. USGS Regression Results.

Recurrence Interval (year)	USGS Region 3 Regression Results (cfs)	6 Hour Model Results (cfs)	
		Tatum Basin	Tatum Basin + Breakout
2	140	887	888
10	569	1,299	1,789
100	1,649	1,641	3,124

5.1.4 Field Verification

A number of field verification points were included in the MIBW ADMS/P H&H Report (Kimley-Horn, 2019). These are locations within the MIBW study where field photographs from the 2014 event were compared with model results from that study. Given that much of the MIBW study extends north of Shea Boulevard, only one comparison point from that study fell within the present study modeling domain.

Figure 5-8 below depicts the conditions and MIBW results from this event at the intersection of Tatum Wash and 44th Street as well as the base results from the present study. The flotsam line from the wash is generally visible. The WSEL profile includes flow from the south along 44th Street, and it is inferred that the transition from a horizontal water surface elevation to varying elevation denotes where the inundation from Tatum Wash intersected the roadway and thereby deposited the flotsam. A simple GIS exercise demonstrates that this location is generally in the vicinity seen in the field photograph taken shortly after the 2014 event.

While this is a qualitative comparison, it does suggest that the base FLO-2D parameterization produces reasonable results.

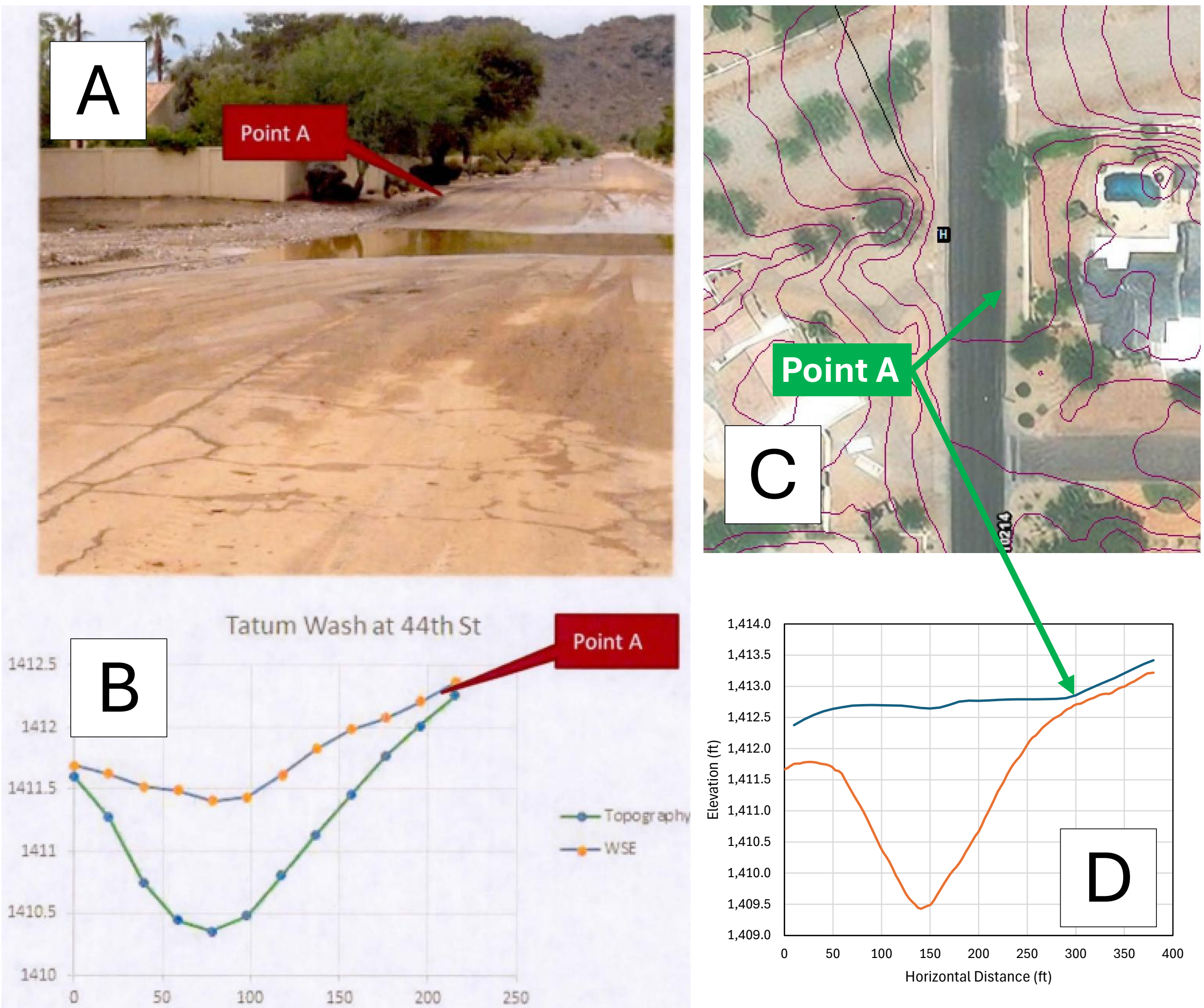


Figure 5-8. 44th Street and Tatum Wash Verification. (A) Field photograph from MIBW report, (B) WSEL profile from MIBW Report, (C) WSEL contours from current study, and (D) WSEL profile from current study.

5.1.5 Further Investigation

Given the disagreements between the gage comparison and regional regression comparisons, further investigation into these differences was performed.

5.1.5.1 FLO-2D Cell Size

To assess the effects of cell size on model results, a 5-ft grid FLO-2D model was developed. This model uses the same input values as the 10-ft grid PVSWMP FLO-2D model, and a comparison of the results is shown below on **Figure 5-10**. Even with the same input shapefiles, the 5-ft model produces a thinner, higher peak hydrograph relative to the 10-ft cell model. This result correlates well with the observed hydrographs at the Tatum Basin Inflow stream gage, where the observed hydrograph arrives faster and has a higher peak than what is obtained from the 10-ft cell with multiple variations of input parameters. This suggests that it may be difficult to fully calibrate the 10-ft model to observed events in this watershed because resolution is lost in the rasterization process relative to actual conditions in the physical world. An example of lost resolution in the Manning's n values is shown in **Figure 5-9**.

More notably, however, is that above a certain flow rate, Tatum Wash breaks out to the east. This breakout occurs over a long length along Tatum Wash (approximately 1,000 feet), and the elevation divide separating the wash from the breakout acts like a lateral weir. Slight differences (e.g., < 0.10 feet) in weir elevations can have very large impacts when considering the long length. The rasterization of the topography at smaller cell sizes closer represents the actual ground surface, and perhaps closer represents the hydraulic properties at the breakout.

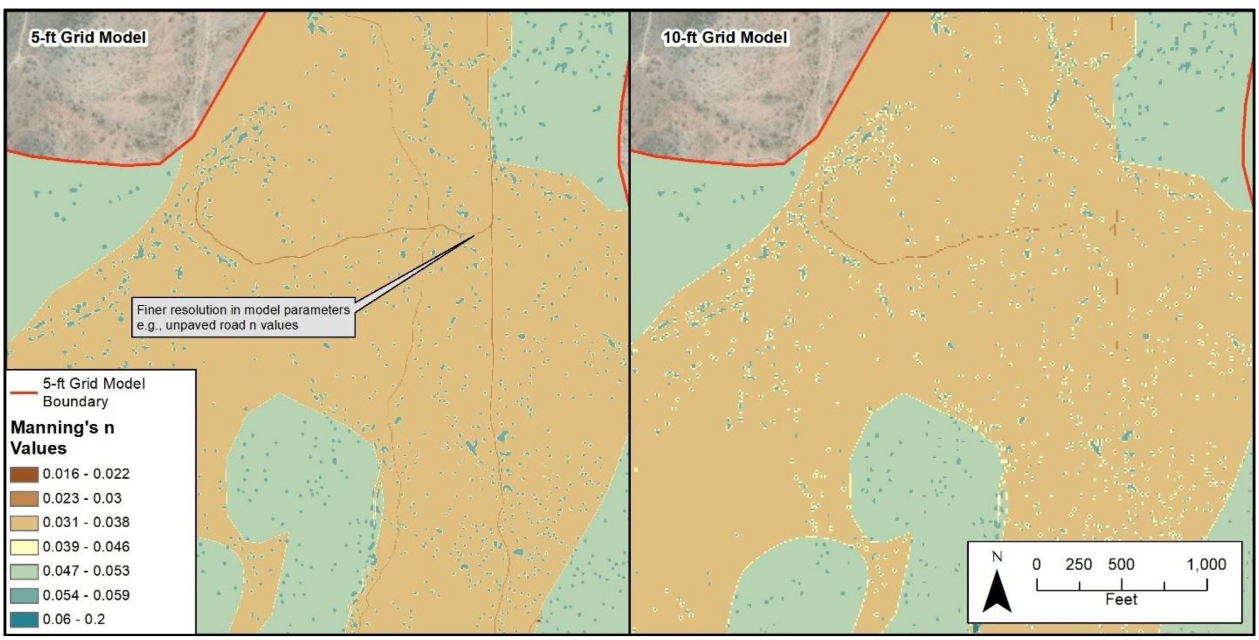


Figure 5-9. Comparison of Manning's n Values between the 5-ft Grid and 10-ft Grid FLO-2D Models.

5.1.5.2 HEC-RAS 2D

To further test model sensitivity, a HEC-RAS 2D (RAS2D) model was developed. Since the infiltration input parameters for this model differ from the FLO-2D model (e.g., initial abstraction is not an explicit parameter and transmission losses cannot be simulated) and RAS2D does not currently implement a shallow Manning's n-value, the results do not provide an exact comparison to FLO-2D. However, the results do give insight into how each of the models compute runoff. The results from the RAS2D model are compared with the FLO-2D results on **Figure 5-10**. This comparison shows that RAS2D produces a significantly higher peak, and the arrival time is shorter relative to both FLO-2D hydrographs – similar to the differences seen between modeled and measured events for the three calibration storm events. This would indicate that infiltration parameters and Manning's n values may need to be adjusted to produce results equivalent to FLO-2D.

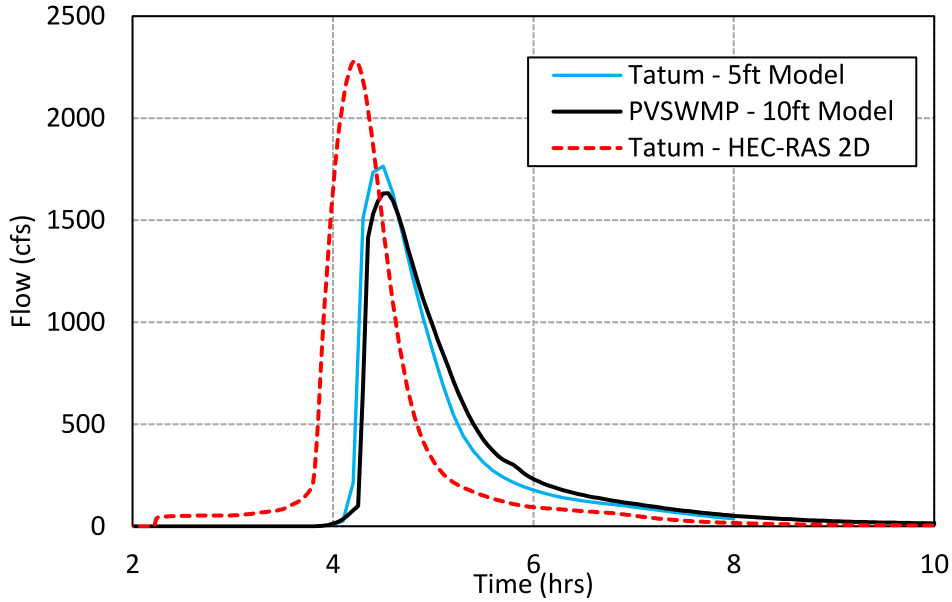


Figure 5-10. Comparison of 100-year Hydrographs at the Tatum Basin Inflow Channel.

Lastly, comparisons between these test models, the base FLO-2D model results, and the gage data were compared for the 2006 and 2014 calibration events (**Figure 5-11** and **Figure 5-12)**. Note that both test models did not use gridded rainfall data (i.e., the RAINCELL.DAT for FLO-2D) for the real event simulations, rather the rainfall input was estimated based on the data in **Figure 5-3**, the storm events where there was less variation across the watershed may be more accurate than those with more variation.

Simply reducing the cell size from 10 to 5 feet yielded a better improvement in matching the gage data than any scenario listed above (**Table 5-2**). The 5-ft hydrograph exhibits an improved arrival time, peak discharge, and the shape better matches the measured hydrograph (in terms of width). It can be easily assumed that calibration of the 5-ft model to the 2006 event would be possible and a very close match could be achieved. The arrival time of the RAS2D hydrograph is much less than the gage data, as is the peak discharge, volume, and hydrograph shape. It should, however, be noted again that there are significant differences in the parameterization of RAS2D as compared to FLO2D. Perhaps the two largest drivers of these differences are the inability to provide increased roughness for shallower depths and no mechanism to simulate transmission losses, where FLO-2D inherently computes these losses when using the Green-Ampt infiltration routine.

A similar trend was seen in comparing the 5-ft and 10-ft model to the 2014 event. The results for the 5-ft model showed slightly elevated peak flow rates and an arrival time much closer to the gage data. Both modeled scenarios, however, over-predicted volume by a significant margin.

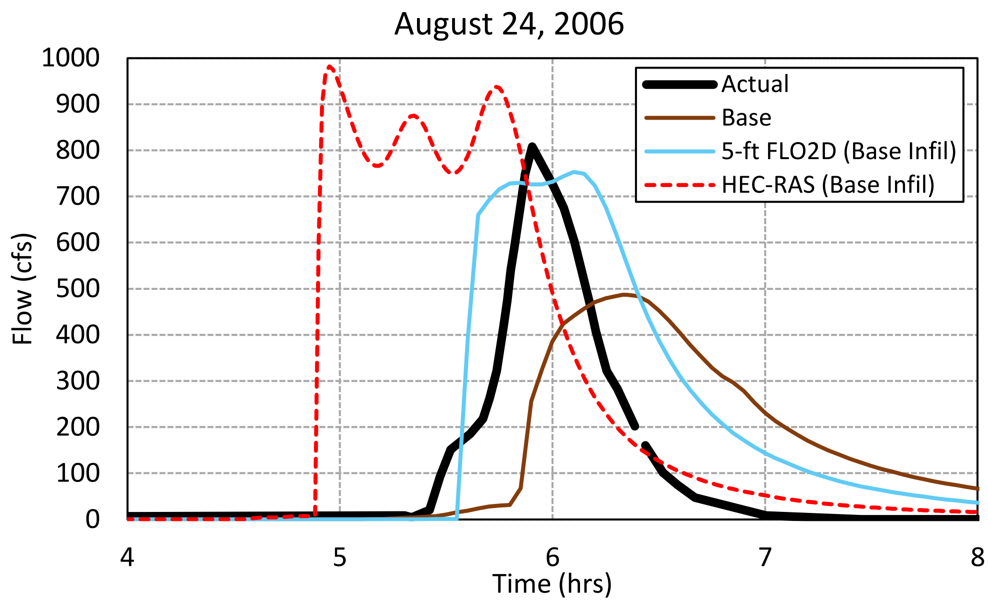


Figure 5-11. Comparison of Hydrographs at the Tatum Basin Inflow Channel for the Event of 8/24/2006.

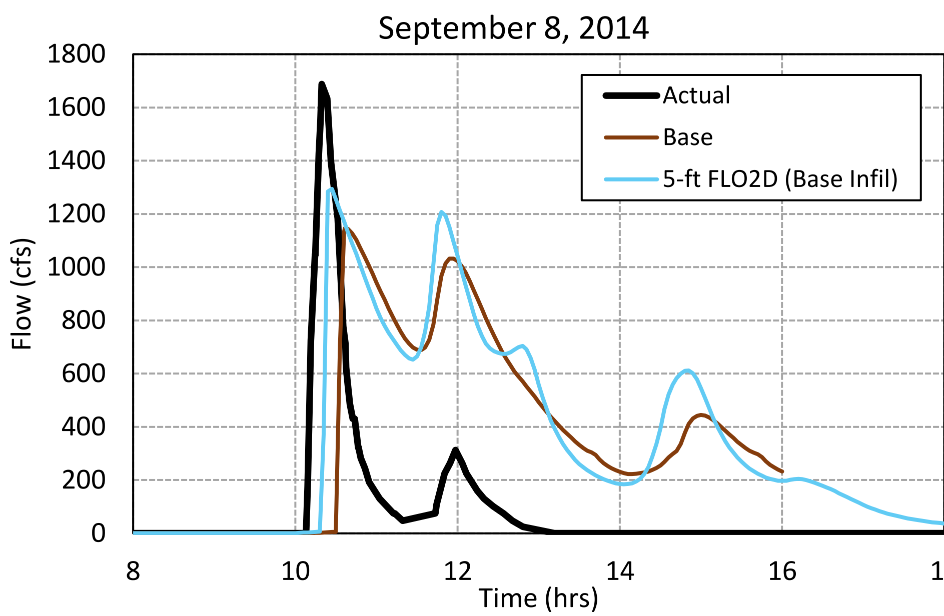


Figure 5-12. Comparison of Hydrographs at the Tatum Basin Inflow Channel for the Event of 9/8/2014.

5.2 Comparison to Previous Studies

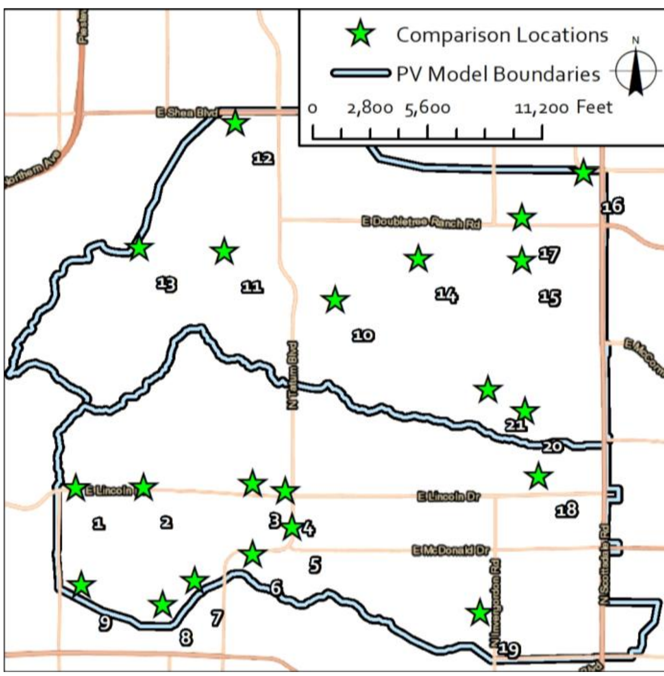
Several exercises were performed to assess the results of this study in the context of previous studies.

5.2.1 Agreement with Existing Studies

The base (i.e., un-calibrated) results of this study were also compared to the results of the contributing study to assess differences and to ensure that the results generated in this study are reasonably close to previous, and in some cases recent results.

The 100-year 6-hour results were compared to the overlapping contributing studies listed in **Table 2-1** with the exception of LIBW, where only the 24-hour event was simulated. Further, this model was not re-computed as there are no inflows into the current model from LIBW. Note that while most of the input data used in developing the PVSWMP model was taken from the Cudia City ADMS, the peak flow rates used in this comparison are from the final Echo Canyon Wash Letter of Map Revision (LOMR) (JE Fuller, 2024) FLO-2D model since this modeling was recently approved by FEMA for a floodplain revision in Echo Canyon Wash (also referred to as Cudia City Wash). The Echo Canyon Wash LOMR modeling exercise resulted in minor changes to the Cudia City Wash ADMS model throughout the LOMR process.

A total of twenty-one comparison points were made throughout the study area (**Figure 5-13**). In general, there was a very close agreement between the current study and the contributing studies. The largest discrepancies were noted when comparing to the East Shea ADMS/P results, where the current study exhibited higher peak discharges. While the average percent difference across the twenty-one comparison points is 8.9%, when weighting the difference based upon the magnitude of the flow rate this difference drops to 0.42% indicating that the base results closely align with the overlapping studies.



Comparison Point	Peak Q (cfs) Current Study	Peak Q (cfs) Comparison	Difference cfs	Difference %	Source
1	595	547	48	8.4%	Cudia
2	124	110	14	12.0%	Cudia
3	1,257	1,190	67	5.5%	Cudia
4	205	202	3	1.5%	Cudia
5	1,510	1,346	164	11.5%	Cudia
6	3,201	2,894	307	10.1%	Cudia
7	2,790	2,784	6	0.2%	Cudia
8	3,295	3,109	186	5.8%	Cudia
9	882	973	-91	9.8%	Cudia
10	1,451	1,483	-32	2.2%	MIBW
11	368	359	9	2.5%	MIBW
12	1,643	1,403	240	15.8%	MIBW
13	3,432	3,269	163	4.9%	MIBW
14	426	426	0	0.0%	MIBW
15	978	617	361	45.3%	E Shea
16	479	546	-67	13.1%	E Shea
17	981	579	402	51.5%	E Shea
18	372	347	25	7.0%	LIBW
19	284	237	47	18.0%	LIBW
20	204	221	-17	8.0%	LIBW
21	616	508	108	19.2%	LIBW

Figure 5-13. Comparison to Previous Study Results.

5.2.2 Off-Site Inflows

The off-site models (i.e., MIBW and E Shea) were re-computed using a variety of recurrence interval and duration combinations. Given that these models were previously calibrated, inflow from these models were examined within the PV model to ensure agreement of peak flow rate values between the off-site model results and what is shown in the PV model results. This is especially important, as the off-site models were re-processed using a much newer FLO-2D executable version.

The primary comparison point is on Indian Bend Wash and Shea Boulevard, at the northern boundary of the PV model. The original study reported a 100-year 6-hour peak discharge of 4,254 cfs, and the equivalent peak discharge at a nearby floodplain cross-section in the PV model was 4,420 cfs. This confirms that the inflow from the MIBW model was accurately translated into the PV model.

5.3 Conclusion

The calibration scenarios were not applied to the design storm events, and the base model parameterization was determined to best reflect hydrologic and hydraulic conditions in the study area. This determination was based on the following factors and conclusions:

- Attempts to calibrate the model to the 2006 storm event could not replicate the gage data, and the modeled discharges were less than the measured discharges. The inverse relationship was found when comparing to the USGS regression equation results, where the modeled results were significantly higher than the regression estimates. Therefore, any attempts to push the model to match one dataset would pull it further from the other, and both datasets (i.e., gage data and regression equation results) are valid and suitable for calibration.
- Further investigations suggest that observed discrepancies may result from basal factors, including model cell size and even model software.
- There is close agreement with the base parameterization results with the final contributing study results. This is particularly important, as it provides a seamless transition to adjacent studies. Further, regulatory flow rates have already been established within the Town limits using the contributing studies, and it is important that this Town-wide model yield similar results.

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