

# Stagecoach Area Drainage Master Plan Technical Support Data Notebook



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# Stagecoach ADMP

## Technical Support Data Notebook

September 2, 2024



Prepared for:

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## Appendices

**Appendix A** – Concept Design Technical Memorandum, Concept Design Sheets, Construction Cost Estimates, and Life-Cycle Cost Estimates

**Appendix B** – Digital Data Submittal

# 1 INTRODUCTION

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## 1.1 PROJECT PURPOSE

The Stagecoach Area Drainage Master Plan (ADMP) was developed to meet three primary objectives.

- Evaluate and identify flooding and sedimentation hazards in the study area. This involves data collection, reviewing previous studies, gathering information from public agencies and residents, conducting hydrologic and hydraulic modeling, performing geomorphic assessments, and conducting field surveys.
- Develop a series of alternatives to partially or wholly mitigate the identified hazards.
- Coordinate with stakeholders and conduct public outreach through a series of public meetings to inform about the existing hazards and present the mitigation alternatives.

Each major task is described in the subsequent sections and includes a description of the technical approach, analysis results, interpretation of results, and applicability to the overall project purpose. The results of this study can be used as a planning tool and as input to the design of potential future drainage infrastructure and flood mitigation measures that are appropriate for the physical environment for both existing and future development.

## 1.2 PROJECT LOCATION

The Stagecoach ADMP watershed is approximately 71 square miles and is located on the southeastern slopes of the Virginia and Flowery Ranges, approximately 22 miles northeast of Carson City (Figure 1-1). The watershed is located within Lyon and Storey Counties about 10 miles northeast of the adjacent Dayton area. The primary focus area of the ADMP is the lower watershed area downstream of the mountains, also shown on Figure 1-1.

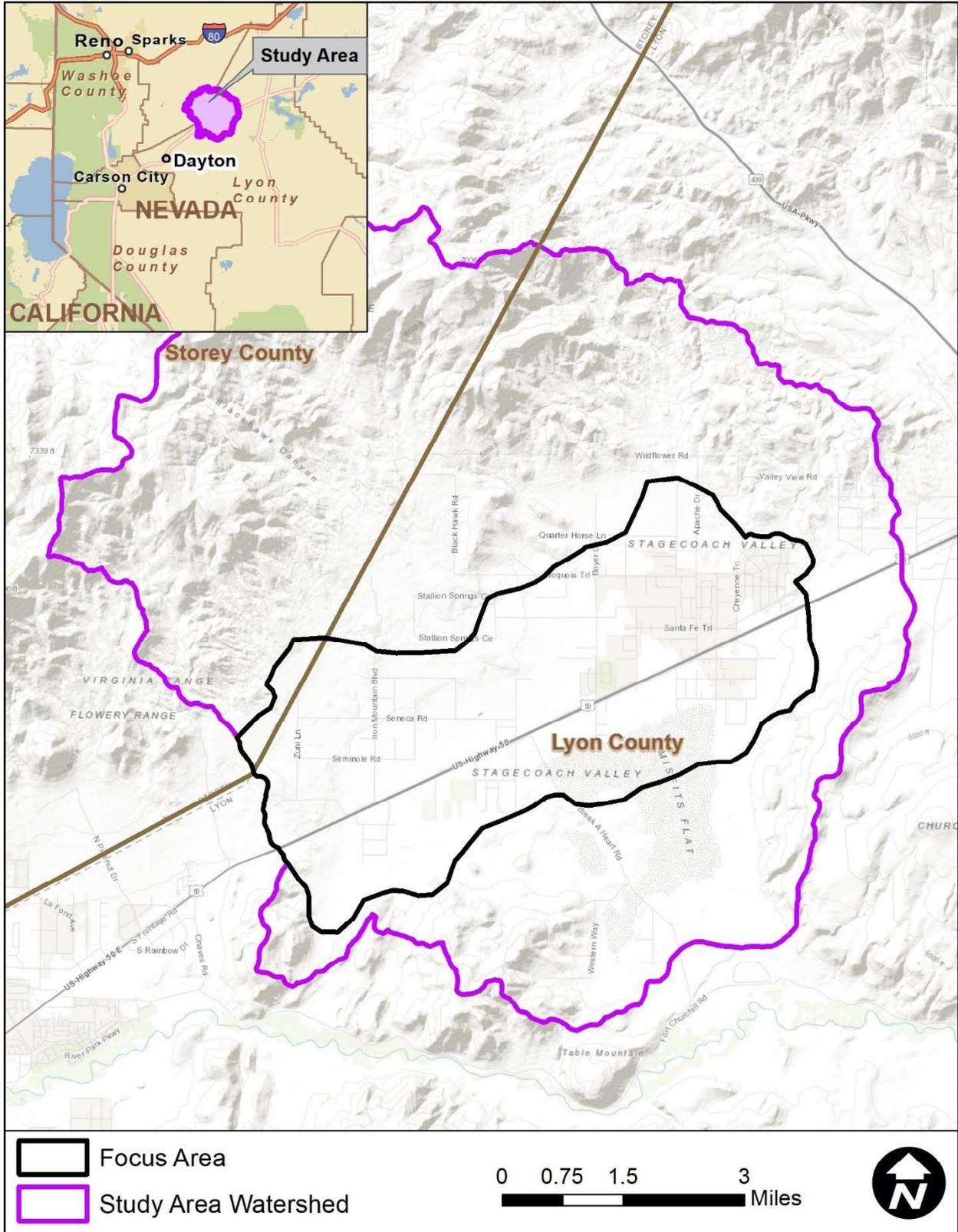


Figure 1-1. Vicinity Map

### 1.3 PREVIOUS STUDIES AND INITIAL DATA COLLECTION

At the outset of the study, multiple types of data and reports relevant to the ADMP area were collected and reviewed. These included drainage reports for local subdivisions, flood insurance studies (FIS), National Resources Conservation Service (NRCS) soils data, Nevada Department of Transportation (NDOT) infiltration parameters and culvert data, historical aerial photography, National Oceanic and Atmospheric Administration (NOAA) rainfall data, and geologic reports.

#### 1.3.1 Flood Insurance Studies

Federal Emergency Management Agency (FEMA) Flood Insurance Studies (FIS) for Lyon County were collected and reviewed for historical flooding records and regulatory discharge estimates for watercourses in the study area. There are no named watercourses within the study area.

##### 1.3.1.1 Effective FEMA Floodplain Mapping

As of the date of this study, the Misfits Flat Playa is the only basin within the study area with a FEMA regulatory floodplain (Figure 1-2). Table 1-1 lists the descriptions for each flood zone shown in the Figure. Like FIS data, FEMA floodplain mapping provides a base-level comparison of flood risk for the hydraulic modeling results from this study.

Table 1-1. FEMA Flood Zones

Flood Zone	Definition	Flooding Type	Recurrence Interval
A	No base flood elevation is provided	Riverine	1% chance
AE	Base flood elevation (BFE) is provided	Riverine	1% chance
AE with Floodway	BFE and Floodway is provided	Riverine	1% chance
X (0.2 PCT ANNUAL CHANCE)	Flooding outside the SFHA	Riverine, Other	0.2% chance

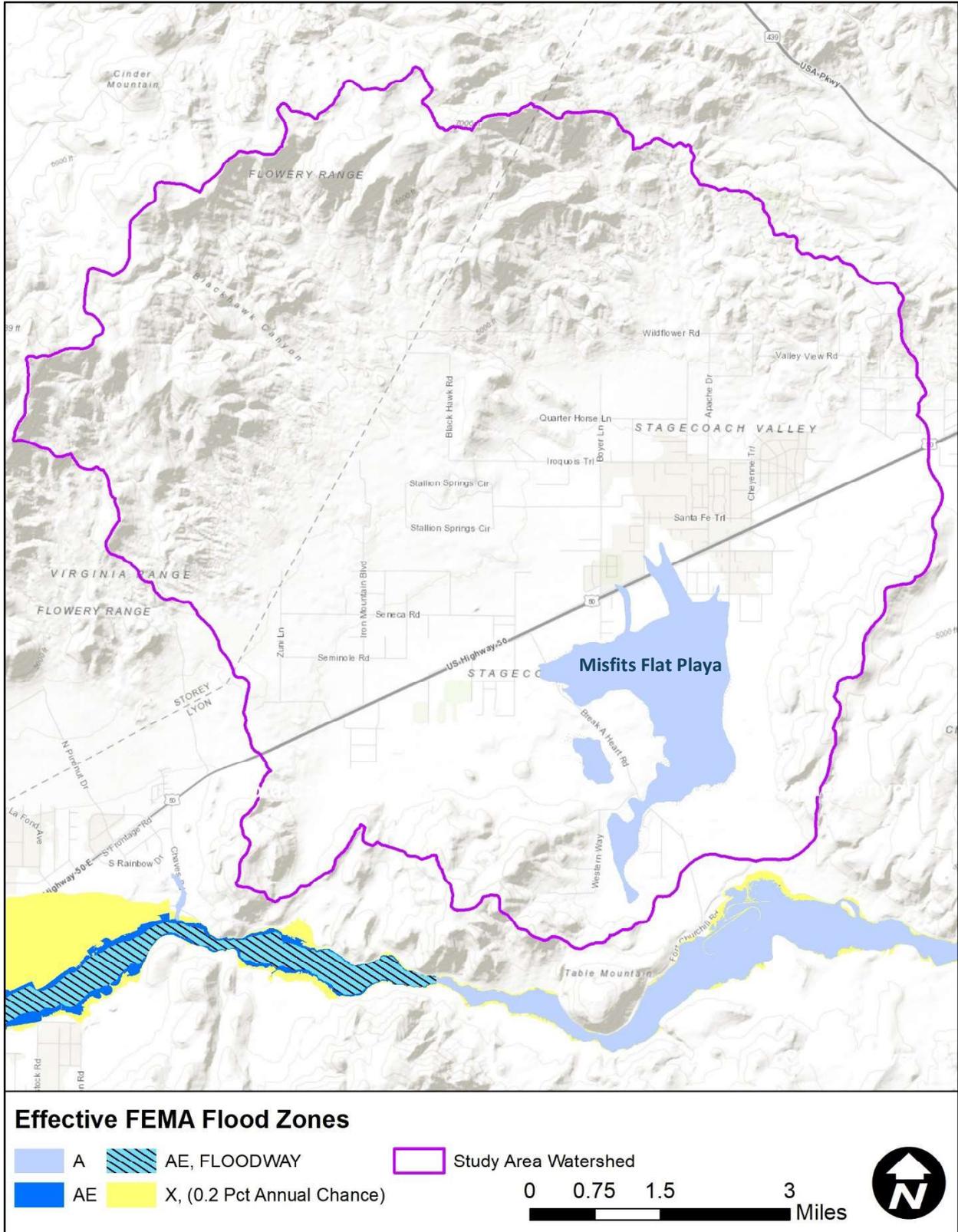


Figure 1-2. Effective FEMA Floodplains

## 1.4 GEOLOGIC SETTING

The study is comprised of the following general geologic landforms: Mountains, Piedmont (Active and Inactive Alluvial Fans), and Playas. Each of these landforms post a different type of flood risk, thus differentiating the landforms is critical to understanding the over risk to existing and future development in Stagecoach.

### 1.4.1 Mountain Landforms

Offsite stormwater impacting Stagecoach north of highway 50 originates from the Flowery Range, and south of highway 50 from the Pine Nut Mountain foothills to the west and Churchill Butte from the east. The highest elevation within the watershed is 7,341 feet and the lowest elevation is 4,247 feet (a difference of 3,094 feet). Stormwater from the mountain landforms collects in tributary drainage channels and flows to the piedmont landforms where it transitions to a distributary pattern. Because flow in the mountain landforms is concentrated in tributary channel networks, the depths and velocities are generally higher than within the other landforms.

### 1.4.2 Piedmont Landforms

A piedmont is a broad, gently sloping, and low relief plain located between mountain ranges and axial drainages and are part of an erosion-depositional system where sediment eroded from mountains is transported by a stream across the piedmont to a valley where it is deposited, or to an axial stream where it is transported out of the valley. Piedmont slopes generally range from less than 1 percent near the valley floors to more than 10 percent near the mountains. Typical piedmonts consist of pediments and relict fans on the upper slopes adjacent to the mountains and alluvial plains on the lower slopes adjacent to the valley floors or base level streams. Active alluvial fans (fans that are presently aggrading and eroding) can occur anywhere on the piedmont. Lower portions of many piedmonts consist of alluvial plains, low-relief aprons of mostly fine-grained deposits with small, discontinuous channel networks. Many piedmonts are formed by the lateral coalescence of separate alluvial fans into a landform called a bajada.

Piedmonts often have areas of tributary and distributary stream channels. Floodwater enters the piedmont in channels from the tributary mountain streams and as overland flow along the mountain front and from rainfall directly on the piedmont surface.

#### 1.4.2.1 Alluvial Fan Landforms

An alluvial fan landform is a geologic sedimentary deposit, or landform, located at a topographic break such as the base of a mountain front, escarpment, or valley side, that is composed of streamflow and/or debris flow sediments and generally has the shape of a fan. The portions of alluvial fans that have been subject to net depositional processes in recent geologic time are referred to as active alluvial fans. Active alluvial fan landforms pose unique flooding hazards that are not present on non-fan landforms such as rivers/streams, whose flooding hazards are commonly defined through Federal Emergency Management Agency (FEMA) regulatory floodplains. Flowpath on active alluvial fan landforms may change during flood events (avulsions), or over time resulting from multiple flow events. The complexity and unpredictable flow patterns on fan surfaces can extend far beyond established channels and associated hazard zones designated based on riverine flood modeling, thus, properly identifying alluvial fan landforms is a critical step in understanding flood risk to downstream development and should be considered during mitigation decisions. Inactive alluvial fan landforms are geologically older and are no longer subject to active fan flooding processes. As they age, inactive alluvial fans develop their own

internal tributary drainage networks that exhibit riverine flooding characteristics. Manmade mitigation structures such as dams, debris basins, retention basins near the fan apex, or channelization from the apex downstream across the piedmont can also result in an alluvial fan transition from active to inactive. Examples of alluvial fan landforms are shown in Figure 1-3 and Figure 1-4.

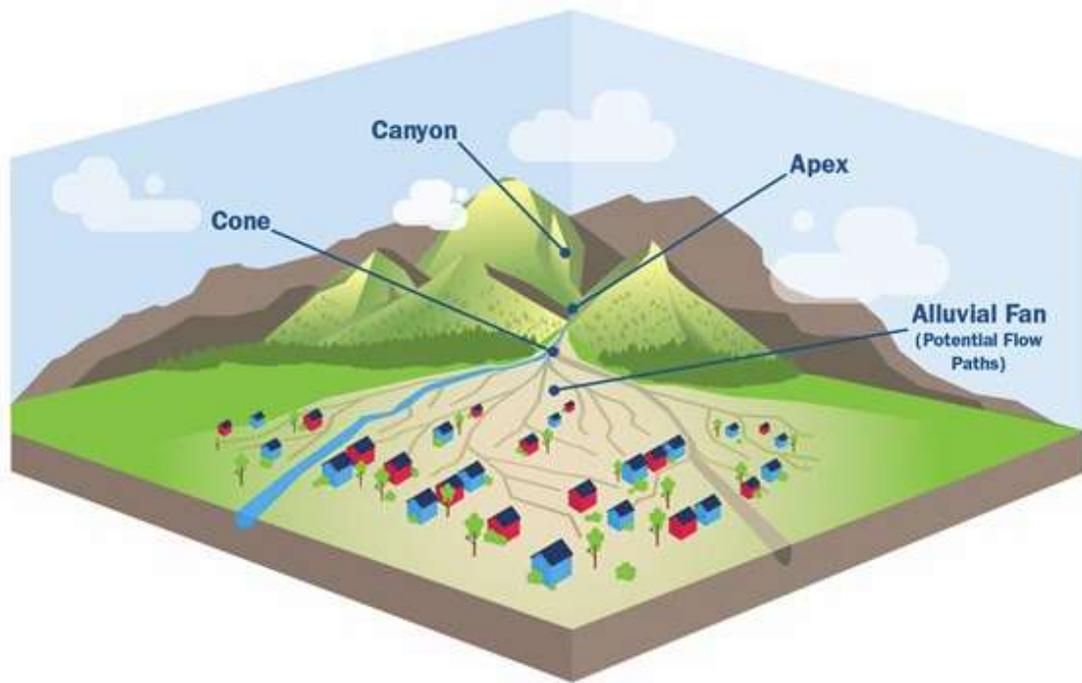


Figure 1-3. Alluvial fan landform example (source: FEMA)

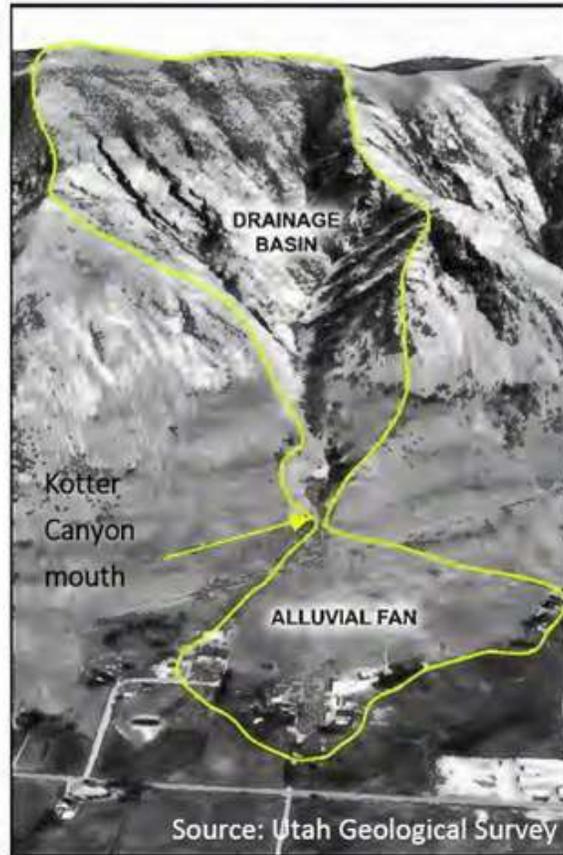


Figure 1-4. Oblique view of an alluvial fan landform near Brigham City, UT

### 1.4.3 Playa Landforms

A playa is defined as a dry, barren area found within a closed basin watershed and comprised of clay, silt, or sand with soluble salts. A playa may be dry for extended periods or may contain water perennially (playa lake). The Stagecoach watershed is a closed-basin system which means that all stormwater drains to a playa and not to an axial stream which normally carries flows downstream out of the watershed. There are two main playas within the study area, the larger is to the east and is named Misfit Flats Plays. The smaller, western playa is unnamed.

A delineation of the landforms identified in the study area is shown in Figure 1-5.

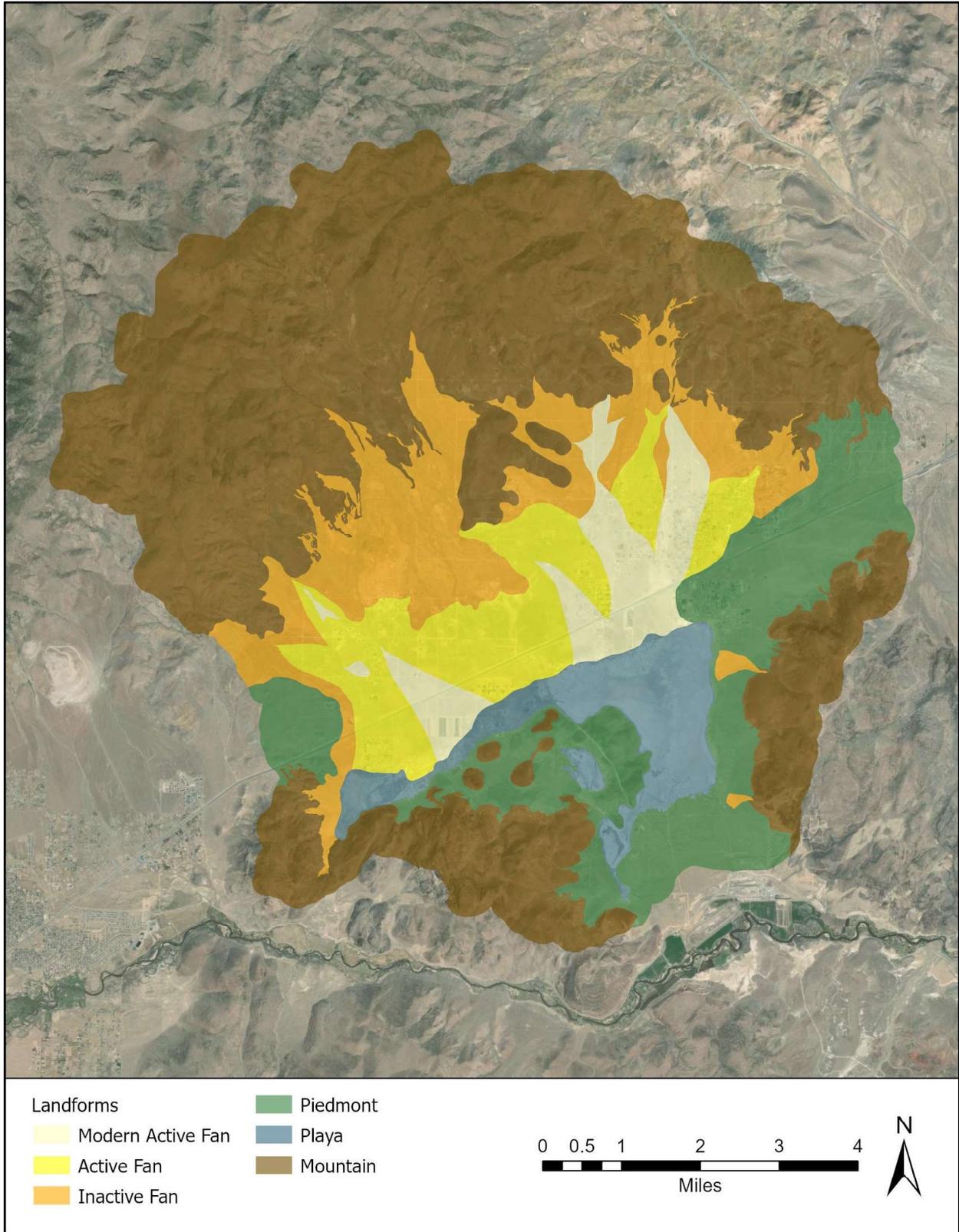


Figure 1-5. Study area landforms

#### 1.4.3.1 U.S. Army Corps of Engineers Alluvial Fan Mapping

In December 2017, the U.S. Army Corps of Engineers (USACE), Sacramento District, published a study titled *Alluvial Fan Mapping for the Carson River Watershed Methodology* (Floyd, 2017) which included the Stagecoach ADMP study area. The purpose of the mapping study was to classify the relative risk of alluvial fan landforms within the Carson River Watershed. Alluvial fan landforms were identified and assigned a risk ranking based on the following categories:

- Appearance of active or inactive
- Existence of disturbances
- Presence of infrastructure

Within each category, a series of risk factors were examined. For example, the Active/Inactive category included four risk factors:

- Soil Development
- Alluvium
- Unconfined Flow
- Incised Channels

The risk factors were assigned a relative score and summed to derive an overall hazard ranking by watershed. Figure 1-6 from the report depicts the distribution of relative risk rankings by watershed. Figure 1-7 shows the identified alluvial fan landforms within the DVADMP study area and their assigned risk.

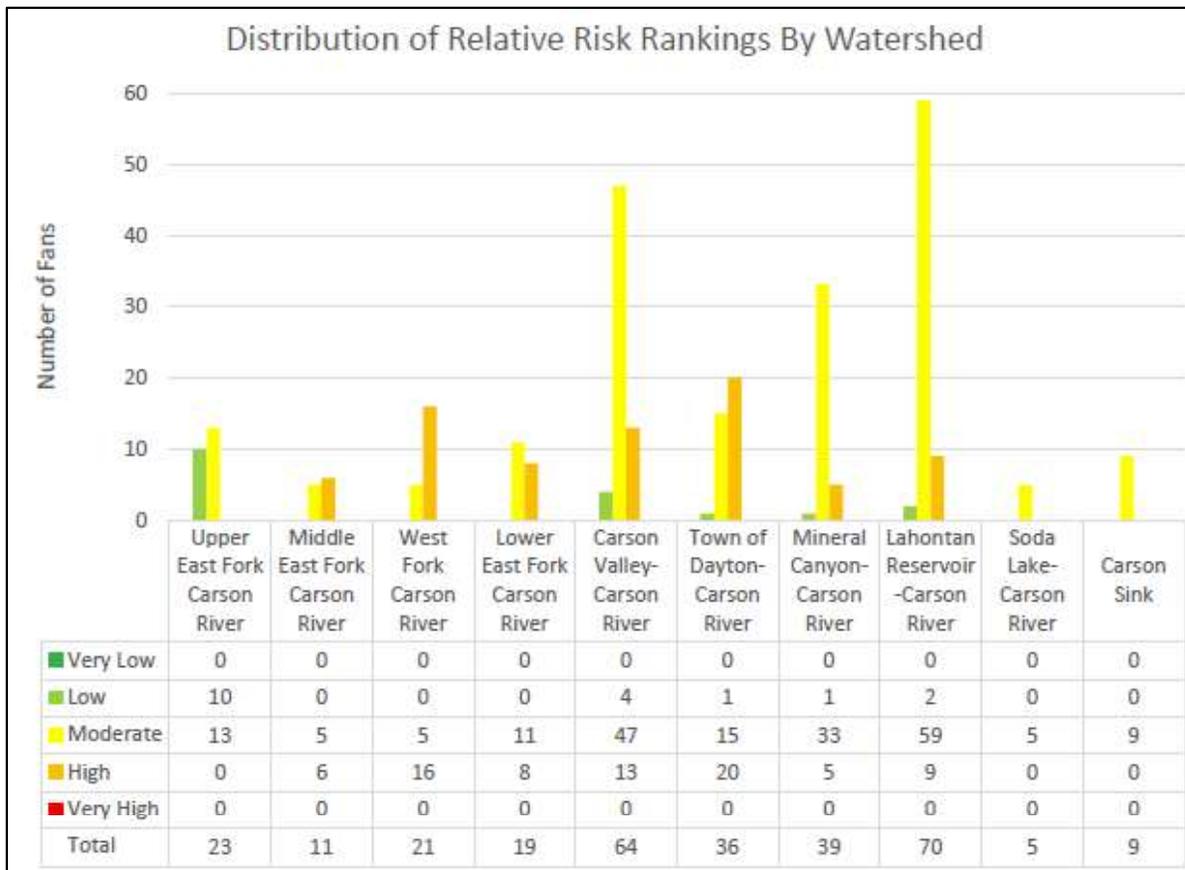


Figure 1-6. Distribution of relative risk rankings by watershed, from Floyd (2017)

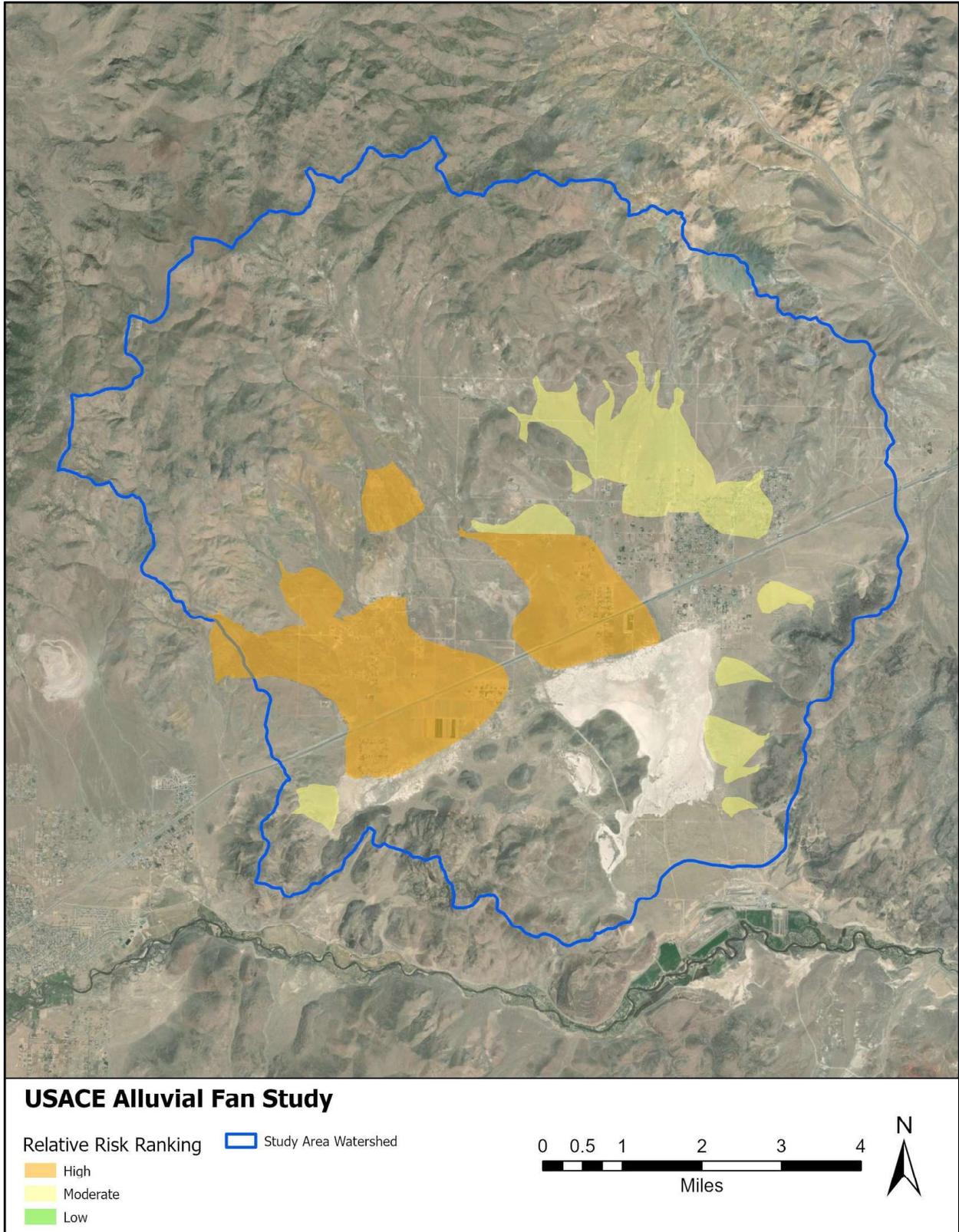


Figure 1-7. USACE alluvial fan risk ranking

#### 1.4.4 Geologic Mapping

The Nevada Bureau of Mines and Geology (NBMG) has published two geologic maps that cover the study area. Table 1-2 lists the published maps. Detailed surficial mapping at 1:24,000 scale is only available for a small portion of the eastern area (Figure 1-8). Descriptions of the map units are listed below. In summary, the mapping confirms that much of the study area is comprised is geologically young alluvial fan landforms.

Table 1-2. Published geologic maps

Geologic Map	Source	Scale	Published Year
Preliminary Geologic Map of the Stockton Flat Well Quadrangle, Lyon and Storey Counties, Nevada	NBMG	1:24,000	2007
Geologic Map of the Carson City 30x60 Minute Quadrangle, Nevada	NBMP	1:100,000	1999

##### 1:24,000 Map Alluvial Fan Units

- Qfy – Young fan deposits
- Qfy1 – Late to idle Holocene fan deposits
- Qfy2 – Early Holocene to Latest Pleistocene fan deposits
- Qf – Fan deposits (undifferentiated)

##### 1:100,000 Map Alluvial Fan Units

- Qfy – Younger alluvial fan deposits



#### 1.4.5 Historical Flowpath Assessment

Understanding the historical evolution of a geomorphic system is critical to understanding present-day processes and predicting future trends. Natural systems can take hundreds of thousands of years to develop, and their morphology is a direct reflection of this long-development period. Anthropogenic changes to a natural system often result in abrupt changes that can be managed for a brief period, but quite often the disturbed system will trend back to its natural condition, despite efforts to change and maintain it.

A historical flowpath assessment was conducted for the study area to assess the natural flowpaths of the study watercourses with the goal that understanding the natural flowpaths will aid in understanding the current flooding patterns and potential future flooding trends.

##### *1.4.5.1 Aerial Photography*

Historical aerial photography from 1948 and 1953 (earliest years available) was collected and semi-rectified using GIS software tools. The natural flowpaths for the project watercourses were identified and delineated from the photography within the piedmont landform areas. Figure 1-9 shows the historical aerial photography, and Figure 1-10 shows the modern aerial photography (2021) for the ADMP focus area. The historical photographs pre-date much of the development within the focus area and show the landforms in a (mostly) natural condition. The locations of the main flowpaths for the major drainage channels were interpreted and delineated from the historical photographs to compare with the present-day locations (Figure 1-11).

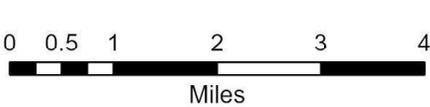


Figure 1-9. 1948 aerial photography



**2021 Aerial Photography**

 Study Area Watershed



*Figure 1-10. 2021 aerial photography*



Figure 1-11. Major flowpath comparison (1948-2021)

#### 1.4.5.2 Summary

The most significant changes in flowpath alignment since 1948 have occurred due to manmade channel realignments associated with development. One of the most notable is the alluvial fan apex located immediately west of Coleman Lane. The natural flowpath of the fan was located further east than at present. It appears that a channel realignment project occurred to prevent flow from inundating the developed area west of Boyer Lane. Figure 1-12 shows a comparison of the flowpaths within this area. Both the historical and modern aerial photography indicate evidence of distributary and active alluvial fan drainage patterns throughout the project focus area, but many of the main drainage channels have remained laterally stable for at least the past 70+ years. This suggests that there may not have been a flood event of sufficient magnitude since at least 1948 to cause major channel avulsions. The major watercourses were investigated during the field verification phase of the ADMP and were not found to be incised or laterally confined within the lower project focus area. In other words, there are no physical constraints that should have prevented historical channel avulsions since 1948, which further suggests that there hasn't been a flood event with sufficient energy to cause a major channel avulsion.

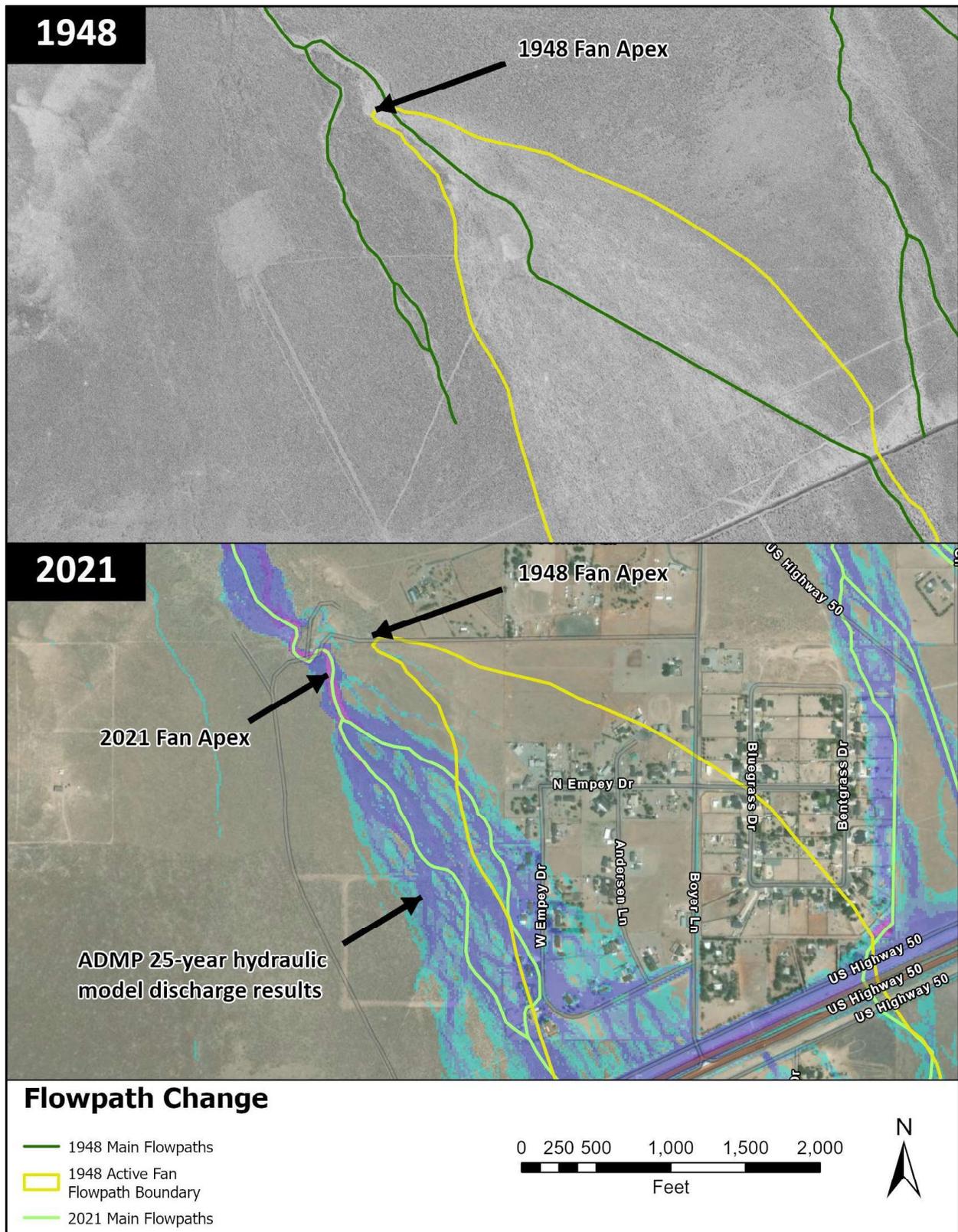


Figure 1-12. Flowpath change example

## 2 HYDROLOGY AND HYDRAULICS

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### 2.1 METHOD DESCRIPTION

The procedures used in the hydrology and hydraulics (H&H) modeling for the for the ADMP are as follows:

- Infiltration was simulated using the physically-based Green and Ampt (G&A) infiltration model using the NDOT infiltration parameters.
- Rainfall depths were based on the NOAA Atlas 14 (NOAA14) precipitation estimates.
- The hyetograph for the 6-hour storm was based on the balanced mass curve, while the hyetographs for the 24-hour storms were based on the Nevada Department of Transportation (NDOT) 90th percentile maximum intensity with smoothing by the generalized logistic equation (GLE) (NDOT, 2015).

The rainfall depths and hyetographs were chosen to maintain consistency with the adjacent Dayton and South Dayton ADMPs. The G&A infiltration method was chosen because it is more physically based than the curve number methodology, and the associated input parameters for model were recently developed for the entire state of Nevada (JEF, 2020).

All modeling, both hydrologic and hydraulic, was done using the FLO-2D Pro software<sup>1</sup> package, Build No. 21.08.23 with an executable dated September 17, 2021. This version has been used for multiple area drainage master studies and has functioned adequately. FLO-2D was selected for this ADMP due to the following: 1) to maintain method consistency with other ADMPs and drainage studies in the area (Manhard, 2012; JEF, 2019; JEF, 2020b) and 2) FLO-2D is a combined rainfall-runoff model (i.e., both hydrologic and hydraulic processes are simulated within the model).

Finally, flow path uncertainty scenarios (Section 2.6) were also developed to account for the shifting of channels over time that can occur on alluvial fans (see Section 1.4). These results were not used in the preliminary design of the concept alternatives since 1) the basins were placed at locations where the flow generally coalesces into a localized inflow, and 2) the major flowpaths have only show minor changes (see Figure 1-11) in the upper watershed since 1948.

### 2.2 MODEL DEVELOPMENT

#### 2.2.1 Spatial Reference System

All data that was generated for the ADMP used the following horizontal and vertical projections:

- Vertical Datum: The North American Vertical Datum of 1988 (NAVD 88)
- Horizontal Datum: Nevada Coordinate System, West Zone, NAD83 (WKID 3423)
- Units of Measurement: US Survey Feet

#### 2.2.2 Model Domains and Grid Size

Since the study area is over 70 square miles, two domains were used to model the area – the Upstream model and the Focus Area model. The spatial location of modeling domain boundary in relation to the

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<sup>1</sup> <https://www.flo-2d.com/>

focus area is shown in Figure 2-1, while the grid size and the number of cells in each domain are shown in Table 2-1.

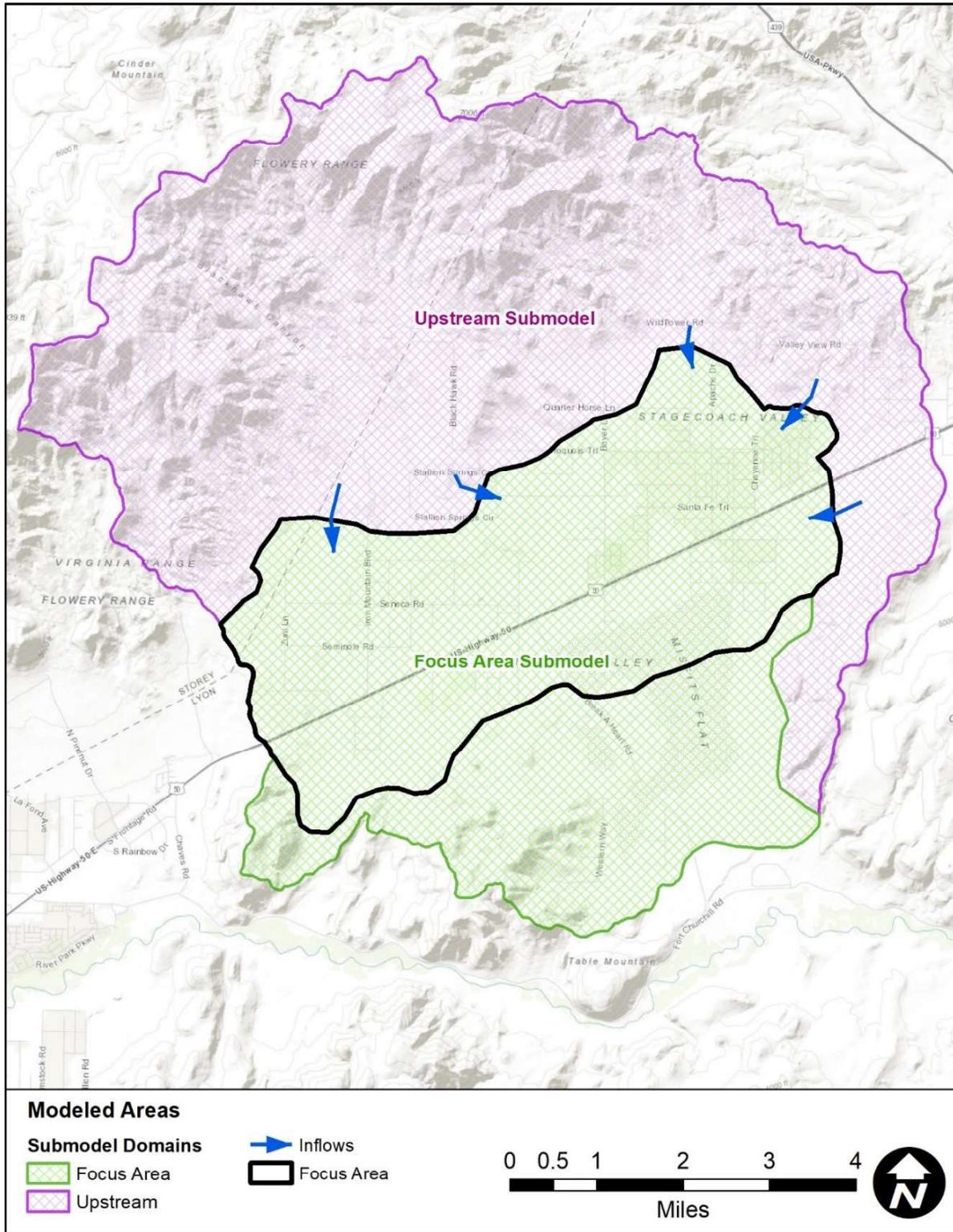


Figure 2-1. Location of submodel domains

A 30-ft cell size was used in the Upstream model because a fine resolution is not necessary in the mountains that are the predominant feature in this model. A smaller 15-ft cell size was used in the Focus Area model to resolve the smaller features (e.g., roadside ditches and culverts) while maintaining a size that could 1) contain the entire Misfits Flats Playa in one domain and 2) keep maximum depths in the Playa less than the grid size.

*Table 2-1. FLO-2D Model Domain Areas and Number of Grid Cells*

<b>Model</b>	<b>Grid Size</b>	<b>Domain Area (sq. miles)</b>	<b>Number of Grid Cells</b>
<b>Upstream</b>	30-ft	39.9	1,235,462
<b>Focus Area</b>	15-ft	31.4	3,884,409

### 2.2.3 Grid Cell Elevations

The limits of the topographic mapping sources used in the FLO-2D modeling are shown in relation to the model domains on Figure 2-2. There were three mapping sources:

- 2020 QL2 LiDAR
- 2017 QL1 LiDAR
- 2017 QL2 LiDAR

All three mapping sources are discussed in detail below.

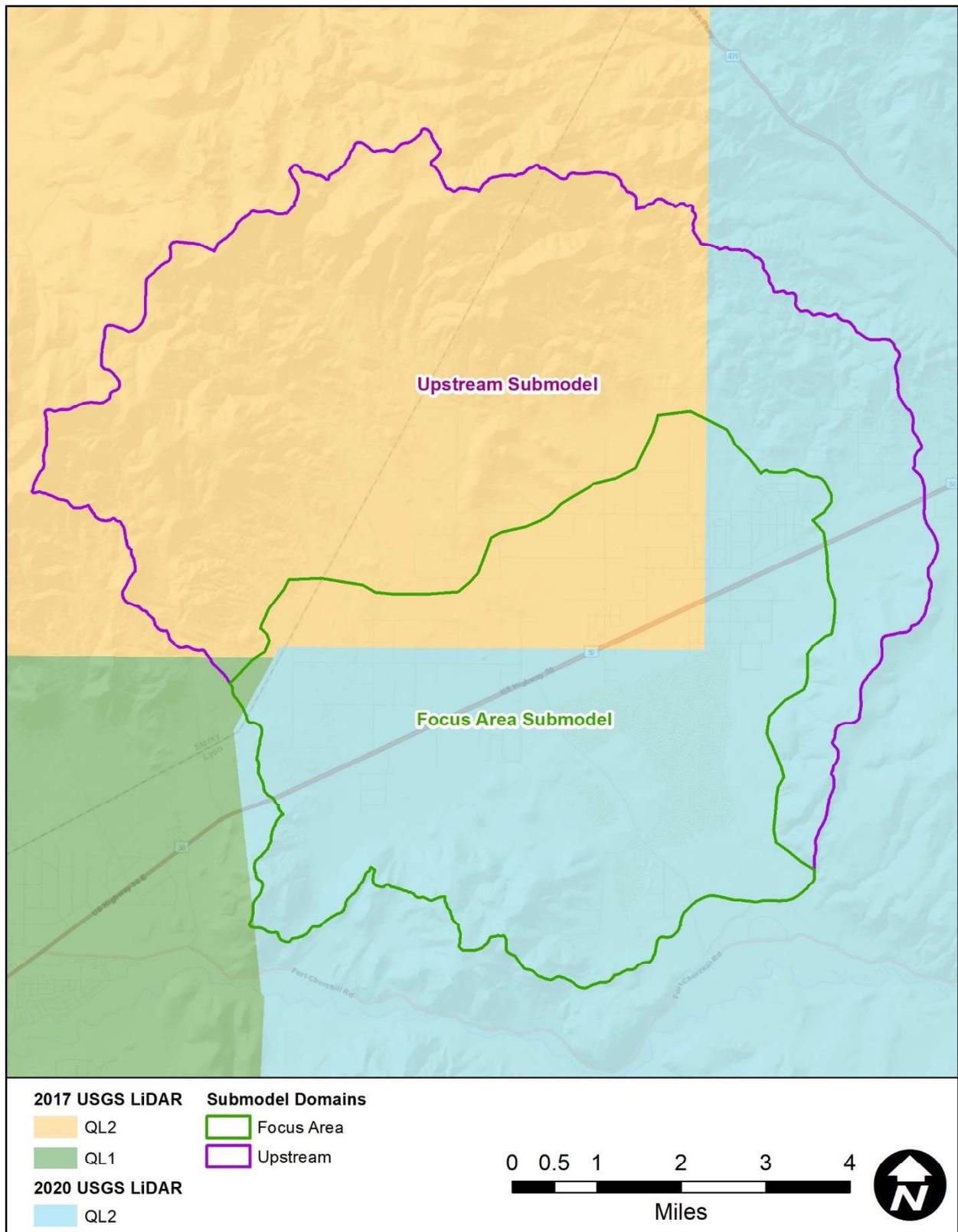


Figure 2-2. Limits of topographic mapping data

**2.2.3.1 2020 United State Geological Survey (USGS) LiDAR**

As a part of the 3D Elevation Program<sup>2</sup>, the USGS collected high resolution LiDAR data for over 4,000 square miles of Nevada through a contract with NV5 Geospatial – Contract Number: G16PC00016. This data was collected at the QL2 specifications. The planned LiDAR specifications are listed in Table 2-2. Collection of the LiDAR data began on October 15, 2020, and was completed on November 2, 2020 (NV5 Geospatial, 2022). The original LiDAR data was collected with elevations in meters, a horizontal spatial reference of UTM Zone 11 N, Meters, NAD83, and a vertical spatial reference of NAVD 88.

*Table 2-2. Planned LiDAR specifications, reproduced from NV5 Geospatial (2022).*

Average Point Density	Flight Altitude (AGL)	Field of View	Minimum Side Overlap	RMSEz
2 pts / m2	2305 m	58.5°	20%	≤ 10 cm

**2.2.3.2 2017 USGS LiDAR**

In 2017, the USGS collected high resolution LiDAR data for a large portion of Carson City and Washoe, Storey, and Lyon counties in Nevada through a contract with Digital Aerial Solutions, LLC (DAS) – Contract Number: G16PC00044. This data was collected at two specifications, QL1 and QL2. The flight parameters and point densities for both datasets are listed in Table 2-3. LiDAR collection began on September 19, 2017, and was completed on October 27, 2017 (DAS, 2018b). The original LiDAR data was collected with elevations in meters, a horizontal spatial reference of UTM Zone 11 N, Meters, NAD83, and a vertical spatial reference of NAVD 88.

*Table 2-3. LiDAR flight parameters, reproduced from DAS (2018b)*

Parameter	QL1	QL2
Flying Height Above Ground Level:	8,609 feet	9,072 feet
Nominal Sidelap:	60%	30%
Nominal Speed Over Ground:	155 Knots	155 Knots
Field of View:	15°	24°
Laser Rate:	220.2 kHz	206.2 kHz
Scan Rate:	65.2 Hz	49.2 Hz
Maximum Cross Track Spacing:	1.22 meters	1.62 meters
Maximum Along Track Spacing:	0.61 meters	0.81 meters
Average point Spacing:	0.50 meters	0.67 meters

**2.2.3.1 Combined FLO-2D Data**

Since the LiDAR data are all in meters and in a different horizontal coordinate system than the ADMP, the data was first converted to feet and the Nevada Coordinate System, West Zone, NAD83 and combined into a single high-resolution raster. This high-resolution raster was resampled to 15-foot and 30-foot rasters that reflect the average grid elevations that are used in the actual FLO-2D models.

<sup>2</sup> <https://www.usgs.gov/core-science-systems/ngp/3dep>

## 2.2.4 Model Inflow/Outflow

In general, outflow nodes were placed along the entire boundary of all model domains to let water free-flow out of the domain. For model boundaries that were coincident to a downstream model (i.e., Upstream submodel to the Focus Area submodel), the outflow hydrographs were read from the outflow node and applied to the lower model as an inflow hydrograph. Since the upper watershed has a coarser grid spacing than the lower model, the outflow hydrograph was sometimes split across two inflow elements with the outflow hydrograph being divided by two before being applied as an inflow hydrograph. The general direction of flow transfer is shown as inflow arrows in Figure 2-1.

## 2.2.5 Precipitation Development

As a part of the ADMP, four design storms were simulated:

- 5-year, 24-hour (5Y24H)
- 25-year, 24-hour (25Y24H)
- 100-year, 24-hour (100Y24H)
- 100-year, 6-hour (100Y6H)

The 24-hour durations were chosen to be consistent with the County’s drainage regulations, while the 6-hour duration was chosen because this higher intensity duration may result in higher peak flow estimates for smaller (i.e., < 20 square miles) drainage areas.

### 2.2.5.1 Precipitation Depths

NOAA Atlas 14 (NOAA14) precipitation depth estimates were downloaded from the National Weather Service (NWS) website<sup>3</sup> as raster images, then used to apply the spatially varied rainfall estimate for each grid element in the model. This means that the NOAA14 point statistics are used at each cell in the model. The maximum rainfall point values for each submodel are shown in Table 2-4.

Table 2-4. Maximum NOAA14 point rainfall estimates (in inches) by recurrence interval and model domain

Model Domain	Storm Event			
	5Y24H	25Y24H	100Y6H	100Y24H
Upstream	2.398	3.434	2.124	4.425
Focus Area	1.586	2.264	1.731	2.910

### 2.2.5.2 Hyetographs

As mentioned in Section 2.1, this ADMP followed the procedures that were applied in the adjacent Lyon County ADMPs. This means that the NDOT GLE hyetograph for this region was used for the 24-hour storms, while the “balanced storm” hyetograph (developed with the HEC-HMS frequency storm option) was used for the 6-hour duration. A comparison of the two temporal distributions that were used in the ADMP is shown in Figure 2-3.

<sup>3</sup> <https://hdsc.nws.noaa.gov/pfds/>

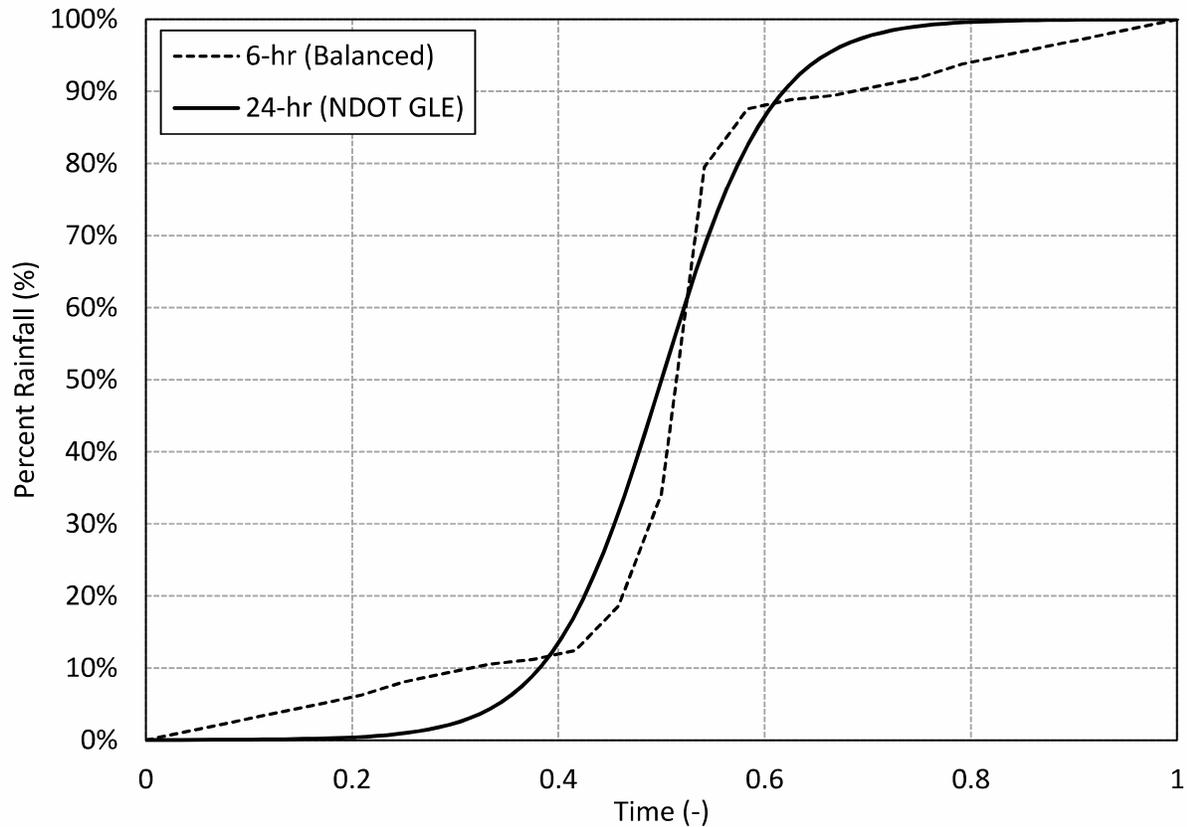


Figure 2-3. Comparison of 6- and 24-hour hyetographs

## 2.2.6 Infiltration Development

In general, the GA infiltration parameters are a function of the features on the ground surface (e.g., a layer of asphalt that covers the soil) or the subsurface soil type. For the ground features, a detailed surface feature classification (SFC) shapefile was developed for this study. This shapefile helped define the surface-based infiltration parameters that were used in the modeling. For the soils, the recent NDOT GA infiltration parameters (JEF, 2020) were used for the soil-based infiltration parameters.

### 2.2.6.1 Surface-based

The infiltration parameters which are dependent on the conditions and type of the ground surface are:

- Percent impervious,
- Initial abstraction (IA) in inches, and
- Initial moisture content.

Table 2-5 shows the surface classification and the corresponding percent impervious, IA, and initial moisture content. The spatial distribution of these surface feature categories is shown in Figure 2.3. These were selected based on experience in other studies, such as the adjacent Dayton Valley ADMPs (JEF, 2019; JEF, 2020b), field observations, and aerial photograph interpretation of the study area. The unimproved roads were given a percent impervious value of 50% to account for added compaction through repeated vehicle use.

Table 2-5. Surface feature categories with corresponding percent impervious and initial abstraction

Surface Feature	Percent Impervious (%)	Initial Abstraction <sup>1</sup> (in)	Initial Moisture Content
Agricultural	0	0.5	normal
Buildings	95	0.05	normal
Desert Rangeland Bare Ground	0	0.3	dry
Hillslope Bare Ground	0	0.3	dry
Mountain Bare Ground	0	0.4	dry
Pavement	95	0.05	normal
Playa	0	0.2	normal
Unpaved Road	50	0.1	dry
Urban Low Vegetation	0	0.1	normal
Wash Bottom	0	0.1	dry
<sup>1</sup> Note that the initial abstraction used in the modeling has been reduced by 0.048 inches to recognize that the TOL (surface detention) value used by FLO-2D acts as a part of initial abstraction			

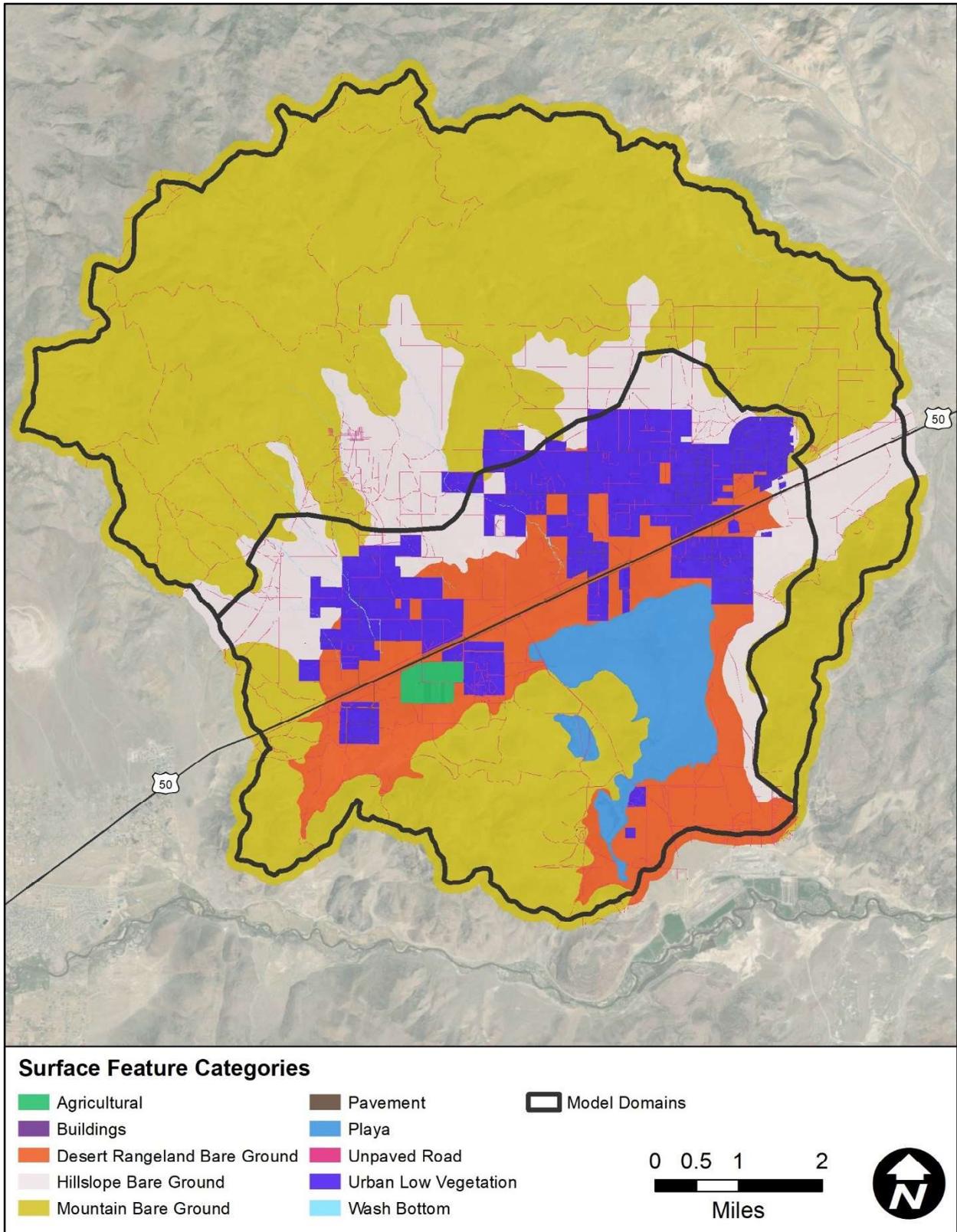


Figure 2-4. Surface feature categories used to assign surface-based infiltration parameters

### 2.2.6.2 Soils

The GA infiltration parameters that are developed from the soil data are:

- Wetting front suction (PSIF) in inches,
- Hydraulic conductivity at natural saturation (XKSAT) in inches per hour,
- Rock outcrop as a percentage,
- Initial soil water content, which is the antecedent moisture conditions either “dry” (i.e., the wilting point, WPOINT) or “normal” (i.e., the field capacity, FCAPAC), and
- Saturated soil water content (SAT).

The water content parameters allow for an estimation of the volume available for infiltration within the soil matrix. Either the WPOINT or FCAPAC (chosen based on the initial moisture content in Table 2-5) is subtracted from the SAT content to calculate the percentage of the soil column available for water infiltration.

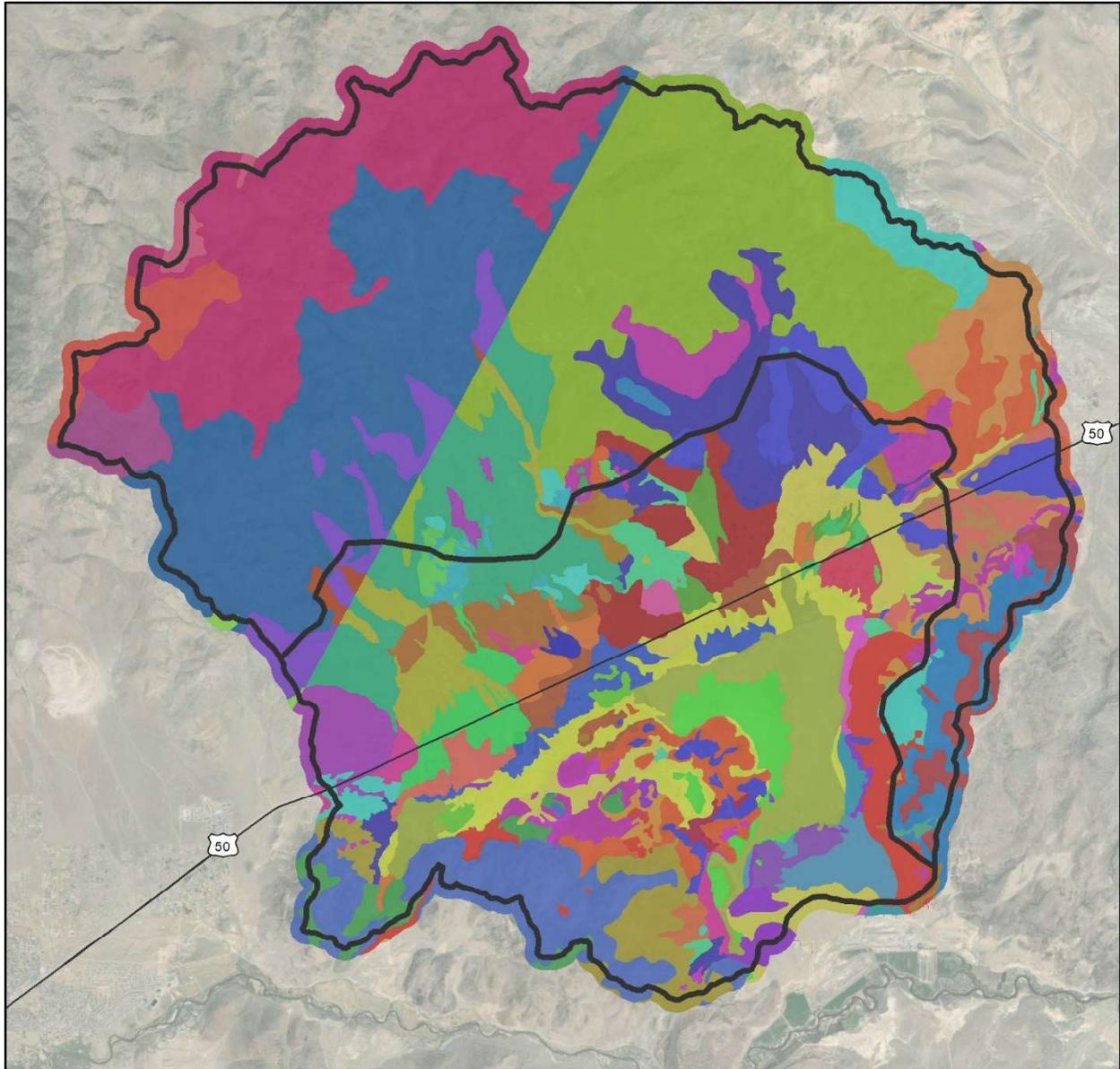
The latest soil shapefile was downloaded from NDOT website<sup>4</sup> to obtain these parameters. The spatial distribution of the soil types within the study is shown in Figure 2-5, and their corresponding GA infiltration parameters are shown in Table 2-6.

The rock outcrop percentage interacts with the surface feature percent impervious to provide a combined percent impervious to each FLO-2D cell. The rock outcrop percentage listed in the soil data is only the percent impervious as it pertains to the soil parameters and not the surface features. It is additive to the surface feature percent impervious until the maximum of 100% imperviousness is reached.

Finally, FLO-2D uses another infiltration parameter named the limiting infiltration depth (LID). This parameter is given in feet and is usually adjusted during the calibration/verification process (see Section 2.4). This parameter, in combination with the moisture content estimates, sets the maximum volume that is available for water to infiltrate the soil matrix. For this study, the LID was set to 0.5 feet.

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<sup>4</sup> <https://geohub-ndot.hub.arcgis.com/pages/ndot-divisions-hydraulics>



**Soil Identification Number (ID)**

2452469	473888	473962	473987	474017	474038	Model Domains
2452702	473894	473963	473988	474020	474054	
2452715	473902	473964	473990	474021	474070	
2452734	473903	473965	473991	474022	474075	
2452746	473904	473976	473992	474023	474080	
2452813	473905	473979	473993	474024	479325	
3110176	473906	473981	473994	474026	479348	
3110177	473907	473982	473997	474027	670323	
473862	473912	473984	474004	474029	670622	
473867	473948	473985	474007	474030		
473875	473957	473986	474012	474035		



Figure 2-5. Soil types within the study area

Table 2-6. Soil GA infiltration parameters

Soil ID	Wilting Point (or Dry)	Field Capacity (or Normal)	Saturated	PSIF (in)	XKSAT (in/hr)	Rock Outcrop (%)
473862	0.044	0.142	0.428	3.128	1.0717	0
473867	0.047	0.153	0.428	4.196	0.8143	0
473875	0.039	0.087	0.416	0.133	1.6639	0
473888	0.021	0.083	0.409	0.720	1.7562	0
473892	0.149	0.250	0.403	7.851	0.1471	0
473894	0.274	0.405	0.486	14.391	0.0221	0
473902	0.057	0.147	0.422	2.97	0.8066	0
473903	0.017	0.080	0.409	0.745	1.8767	0
473904	0.053	0.138	0.394	4.073	0.6241	0
473905	0.049	0.131	0.394	3.668	0.623	0
473906	0.088	0.211	0.393	11.717	0.2157	0
473907	0.088	0.211	0.393	11.717	0.2157	0
473908	0.05	0.132	0.394	3.686	0.6586	0
473912	0.046	0.122	0.403	2.187	0.9858	0
473917	0.132	0.265	0.443	11.08	0.2791	0
473920	0.172	0.303	0.464	8.697	0.1792	0
473922	0.086	0.192	0.430	3.642	0.6026	0
473923	0.128	0.260	0.442	10.499	0.3058	0
473925	0.189	0.325	0.461	12.848	0.1238	0
473926	0.139	0.274	0.455	10.19	0.2972	0
473927	0.184	0.317	0.454	12.557	0.1213	0
473929	0.111	0.231	0.434	8.004	0.397	0
473930	0.144	0.277	0.436	12.711	0.2022	0
473932	0.18	0.312	0.446	13.456	0.1131	0
473933	0.185	0.319	0.455	12.788	0.1185	0
473936	0.018	0.056	0.438	0.014	1.8606	0
473937	0.07	0.166	0.407	5.339	0.7323	0
473938	0.07	0.164	0.406	5.163	0.7307	0
473939	0.068	0.162	0.403	5.333	0.7291	0
473948	0.076	0.203	0.427	6.559	0.5249	0
473957	0.044	0.086	0.415	0.046	1.4912	0
473961	0.061	0.146	0.397	3.739	0.5125	0
473962	0.18	0.332	0.446	20.586	0.0694	0
473963	0.193	0.321	0.437	13.075	0.0443	0
473964	0.135	0.237	0.433	8.8	0.1511	0
473965	0.153	0.28	0.43	10.765	0.1074	0
473974	0.063	0.146	0.405	3.063	0.8441	0
473976	0.127	0.250	0.406	10.605	0.1737	0
473979	0.065	0.149	0.392	4.33	0.457	0
473981	0.042	0.085	0.412	0.059	1.3693	0
473982	0.042	0.085	0.412	0.059	1.3693	0
473984	0.008	0.044	0.433	0.010	2.000	0
473985	0.069	0.139	0.421	0.369	0.7408	0
473986	0.046	0.125	0.394	3.459	0.9391	0

Soil ID	Wilting Point (or Dry)	Field Capacity (or Normal)	Saturated	PSIF (in)	XKSAT (in/hr)	Rock Outcrop (%)
473987	0.066	0.187	0.388	10.953	0.4225	0
473988	0.051	0.135	0.404	3.274	0.9107	0
473990	0.094	0.219	0.404	11.445	0.3218	0
473991	0.037	0.103	0.412	0.918	1.4142	0
473992	0.106	0.226	0.398	11.384	0.1607	0
473993	0.106	0.226	0.398	11.384	0.1607	0
473994	0.093	0.175	0.393	4.771	0.2759	0
473997	0.236	0.358	0.46	12.226	0.0255	0
473999	0.185	0.296	0.44	9.196	0.0616	0
474000	0.076	0.161	0.401	4.247	0.6258	0
474004	0.224	0.349	0.451	13.867	0.0319	0
474005	0.087	0.215	0.426	9.284	0.4667	0
474007	0.027	0.069	0.43	0.022	1.5958	0
474009	0.057	0.145	0.412	3.296	0.9791	0
474012	0.027	0.070	0.433	0.024	1.6727	0
474015	0.032	0.100	0.421	0.555	1.5671	0
474017	0.071	0.159	0.395	4.925	0.3403	0
474020	0.066	0.149	0.402	3.38	0.7182	0
474021	0.066	0.149	0.402	3.38	0.7182	0
474022	0.066	0.149	0.402	3.38	0.5783	0
474023	0.045	0.119	0.409	1.247	1.0799	0
474024	0.168	0.299	0.421	14.631	0.0457	0
474025	0.16	0.294	0.422	16.63	0.0593	0
474026	0.154	0.278	0.424	12.147	0.0794	0
474027	0.209	0.339	0.441	14.149	0.0388	32
474029	0.011	0.046	0.426	0.015	1.9422	0
474030	0.021	0.056	0.421	0.018	1.8635	0
474033	0.13	0.256	0.404	14.506	0.1635	0
474035	0.069	0.159	0.423	2.935	0.5933	0
474038	0.066	0.167	0.398	6.169	0.405	0
474054	0.056	0.105	0.399	0.712	0.8422	0
474055	0.059	0.142	0.392	4.114	0.6227	0
474056	0.06	0.144	0.391	4.724	0.5688	0
474057	0.044	0.097	0.406	0.566	1.076	0
474070	0.085	0.168	0.389	5.319	0.4693	0
474075	0.296	0.423	0.478	11.21	0.02	100
474080	0.201	0.374	0.460	30.511	0.0354	0
479325	0.057	0.143	0.420	2.774	0.8114	0
479348	0.275	0.402	0.488	13.122	0.0268	0
670323	0.03	0.080	0.418	0.098	1.3867	1
670617	0.122	0.232	0.434	5.774	0.3385	0
670622	0.112	0.214	0.414	7.051	0.2234	0
2452469	0.115	0.233	0.427	8.134	0.2082	0
2452702	0.119	0.248	0.436	10.305	0.2716	0
2452715	0.156	0.287	0.435	12.665	0.0810	0

Soil ID	Wilting Point (or Dry)	Field Capacity (or Normal)	Saturated	PSIF (in)	XKSAT (in/hr)	Rock Outcrop (%)
2452734	0.106	0.227	0.435	7.815	0.3501	0
2452735	0.104	0.239	0.439	10.203	0.3326	0
2452741	0.112	0.244	0.431	11.274	0.2762	0
2452746	0.069	0.159	0.423	2.935	0.5933	0
2452813	0.156	0.287	0.435	12.665	0.0810	0
3110176	0.095	0.206	0.420	6.947	0.3625	0
3110177	0.095	0.206	0.420	6.947	0.3625	0

### 2.2.7 Grid Element Roughness (Manning’s Base and Shallow n Values)

The FLO-2D model uses two Manning’s n values to estimate roughness on each element. These are the shallow n value and the base n value. These two parameters allow FLO-2D to calculate a depth-varying roughness, which better approximates physical flood routing in a natural system. For depths below 0.5 feet, the shallow n or half the shallow n value is used. Between 0.5 feet and 3 feet, a function based on the base n value is used; and, at depths greater than 3 feet, the base n value is used. Please see the FLO-2D Data Input Manual (FLO-2D Software, Inc., 2021) for the details about how depth-varying roughness is applied in the software.

Each grid element is assigned an average shallow n and base n value based on the underlying surface conditions. The SFC shapefile (see Section 2.2.6.1) that was developed for this study was used to define the surface conditions. The base and shallow n values for each classification were chosen based on experience on other studies, engineering judgment, and research papers, such as Yochum et al. (2014), Jarret (1985), and JEF (2020a). Table 2-7 lists the surface feature category and its corresponding Manning’s n values that were used in this study. The spatial distribution of the surface classification is shown in Figure 2-4.

Table 2-7. Surface feature categories and corresponding Manning’s n values

Surface Feature	Base n	Shallow n
Agricultural	0.060	0.300
Buildings	0.024	0.100
Desert Rangeland Bare Ground	0.040	0.150
Hillslope Bare Ground	0.050	0.200
Mountain Bare Ground	0.080	0.350
Pavement	0.020	0.100
Playa	0.100	0.300
Unpaved Road	0.026	0.120
Urban Low Vegetation	0.045	0.200
Wash Bottom	0.035	0.150

### 2.2.8 Hydraulic Structures (Culverts and Storm Drains)

Both culverts and minor storm drains can be simulated with the hydraulic structure routine within the FLO-2D software. Please see the FLO-2D Data Input Manual (FLO-2D Software, Inc., 2021) for more details on the application of this routine and its associated modeling options.

NDOT provided sizes and locations for culverts along US 50 via their online GIS application<sup>5</sup>. Additionally, JEF staff conducted site visits to the study area in July and October 2023 to locate additional structures and to verify the information developed from the collected data (e.g., locations, sizes, overall condition, etc.). Not all driveway culverts were input to the model since many were observed to be completely clogged with sediment or crushed (e.g., Figure 2-6). However, driveway culverts that appeared to impact initial results and were observed to be in relatively good condition and clear of sediment were added to the model.

Circular and box culverts were simulated with the generalized culvert option, while other shapes (e.g., ellipse or arch) were simulated with a rating table developed from HY-8 (or EPA-SWMM). All culverts used an INOUTCONT parameter of 1 to account for tailwater effects, and no clogging factor was applied. The modeled culverts were generally clear of sediment. One larger (estimated as a 3 barrel 36-in pipe culvert, see Section 3.1) was completely buried and was not added to the model.

All culverts (there are no storm drains within the study area) that were modeled as a part of this ADMP are shown in Figure 2-7.



Figure 2-6. Example of clogged/crushed driveway culvert

<sup>5</sup> <https://www.dot.nv.gov/doing-business/about-ndot/ndot-divisions/stormwater/mapping>

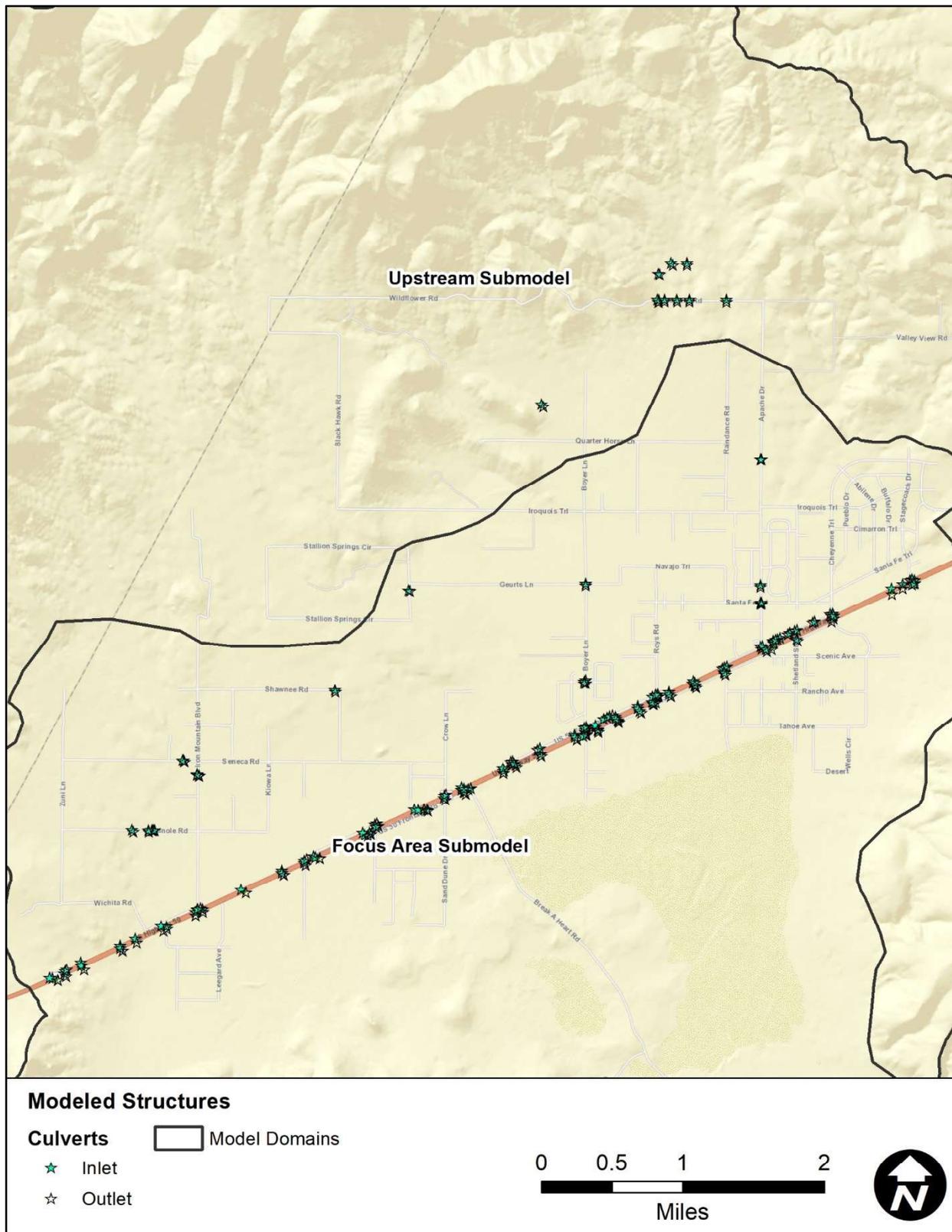


Figure 2-7. Modeled hydraulic structures (culverts)

### 2.2.9 Buildings (as Flow Obstructions)

An updated building footprint shapefile was developed and used to create a global area-weighted 15- and 30-foot blocked obstruction rasters. These rasters were used to extract the percentage of area obstructed by buildings and assigned to area reduction factors (ARF) for each grid in the 15-foot grid Focus Area and the 30-foot grid Upstream submodels. Cells that had blocked percentages greater than 90% in the 30-foot grid and 85% in the 15-foot grid submodel were modeled with the “totally blocked element” T record. This sets the cell blocked percentage to 100% to avoid small area instabilities on these cells. Finally, width reduction factors (WRF) were not used in this study and were assigned a 0 value in the ARF.DAT file. The buildings that were modeled with the ARF functionality are shown in Figure 2-4.

### 2.2.10 Model Adjustments

Two types of minor adjustments were made to the models. These were:

- 1) Lowering cell elevations to approximate culverts that were inaccessible or far upstream in the watershed. There were three locations where cells were lowered to approximate culverts.
- 2) Two cells that were identified as “depressed cells” (i.e., cells that are more than 4 feet lower than any adjacent cell) had their elevations raised to allow flow off the cell.

### 2.2.11 Model Control Parameters

CONT.DAT and TOLER.DAT contain numerical stability and simulation controls for the FLO-2D model. The CONT.DAT file controls simulation time, output report time interval, some numerical controls and model switches, such as infiltration and rain. The total simulation time was set to 20 hours for the 6-hour storm, while the total simulation time was set to 40 hours for the 24-hour storms. The times were adequate to ensure the floodwave moved through the watershed and that the maximum depth in the Playa was achieved.

#### 2.2.11.1 CONT.DAT

In the CONT.DAT file, the global Manning’s n value adjustment factor (AMANN) and the limiting Froude number (FROUDL) were the numerical controls that were used in the Stagecoach ADMP study. For this study, these controls were set to:

- AMANN = 0 (depth integrated roughness is used with the SHALLOWN parameter)
- FROUDL = 0.95
- SHALLOWN = 0.35 (spatially varied shallow Manning’s n was also used, see Section 2.2.7)

For the limiting Froude number, a value of 0.95 was used in this study since most flow is subcritical in natural watersheds. The global SHALLOWN parameter was set to 0.35, but a spatially varied shallow n was applied per the detailed surface feature classification to adjust the shallow n where appropriate (e.g., on paved streets).

#### 2.2.11.2 TOLER.DAT

The TOLER.DAT file contains the numerical tolerance settings specified for the model. These settings are the flow exchange tolerance (TOL), percent allowed change in flow depth (DEPTOL), dynamic wave stability criteria (WAVEMAX), and Courant-Friedrich-Lewy numerical stability parameter for floodplain grid element flow exchange (COURANTFP). For the Stagecoach ADMP models, the settings applied were:

- TOL = 0.004 feet (the depth at which FLO-2D begins to route flow)

- DEPTOL = 0 (not used, model uses Courant number as stability criteria)
- WAVEMAX = 0 (not used, model uses Courant number as stability criteria)
- COURANTFP = 0.6 (main stability criterion used by FLO-2D)

These values have been used in similar studies, which yielded reasonable results. For this project, these values have produced good model stability and reasonable results.

## 2.3 MODEL RESULTS

### 2.3.1 Floodplain Cross-Sections

Floodplain cross-sections were developed and included in the FPXSEC.DAT file to query flow hydrographs, peak discharges, and flow volumes from the FLO-2D model at key locations, such as:

- Major flow concentration locations,
- Areas near potential mitigation sites, and
- Areas of interest to Lyon County

Major floodplain cross-section locations are shown on Figure 2-8. Hydrograph plots at the floodplain-cross-sections for each storm event are included in Appendix B. The peak flow and volume for each floodplain cross-section are shown in Table 2-8.

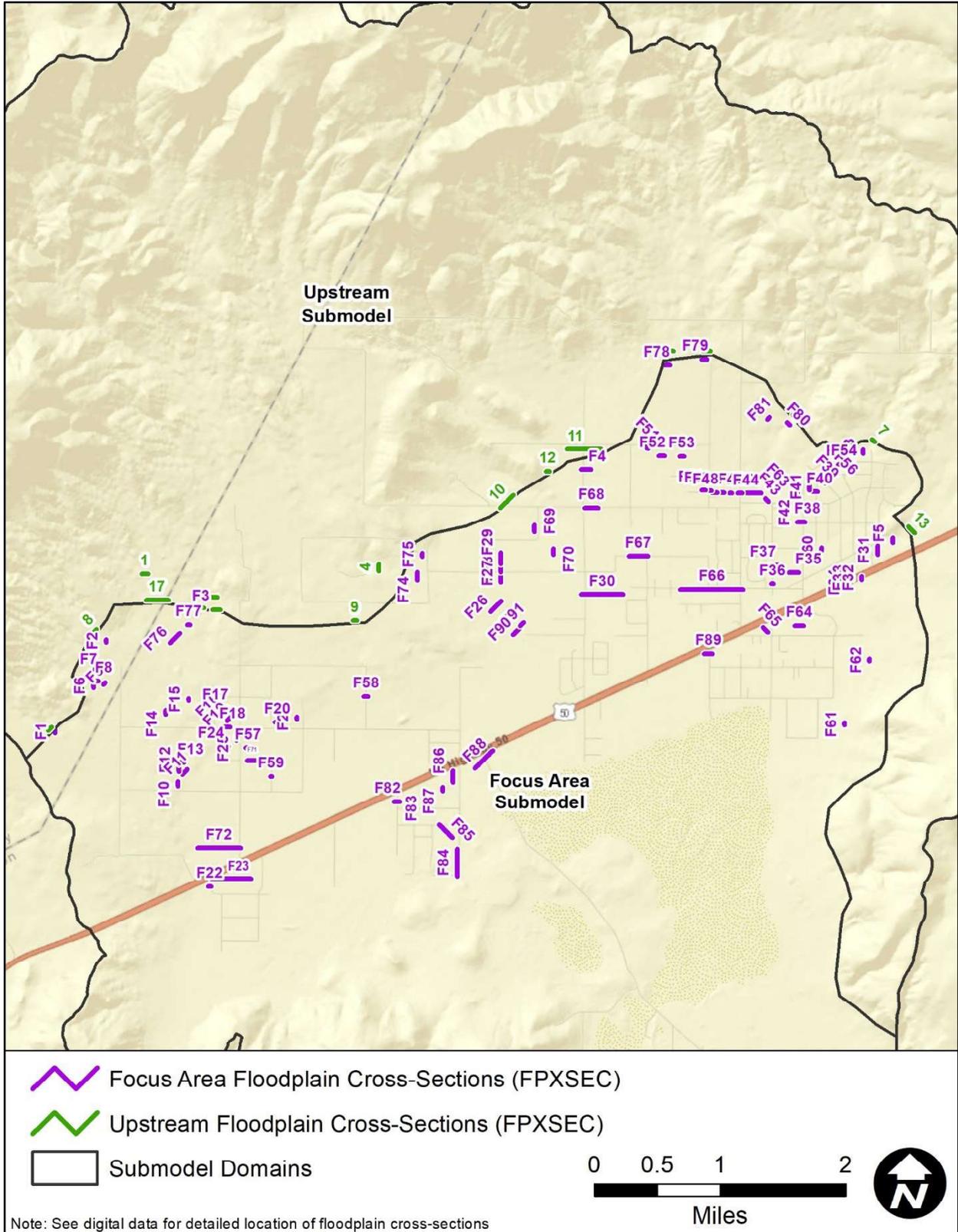


Figure 2-8. Floodplain cross-section locations

Table 2-8. Peak flow and volume results from the FLO-2D floodplain cross-sections

ID	100Y24H		100Y6H		25Y24H		5Y24H	
	Peak Flow (cfs)	Volume (ac-ft)						
<b>Note: Volumes shown below may not contain the entire hydrograph volume due to extreme model runtimes when modeling ponded conditions. Please verify correct volume is used for any design.</b>								
<b>Upstream Model</b>								
1	191.22	69.63	309.27	21.59	76.09	26.38	0	0
2	1395.72	390.26	631.74	71.23	674.91	186.67	2.84	1.58
3	39.73	15.26	46.76	4.8	6.92	2.95	0	0
4	1928.93	634.55	1168.98	125.45	892.77	296.38	5.27	1.51
5	830.44	312.67	923.76	84.92	379	142.82	0	0
6	134.81	45.09	142.16	12.26	48.82	17.59	0	0
7	234.86	95.36	184.22	30.13	48.22	23.54	0	0
8	261.47	94.48	406.08	28.54	119.98	39.76	0	0
9	29.93	11.19	0.49	0.15	0	0	0	0
10	77.43	32.38	24.98	7.68	19.96	9.05	1.19	0.56
11	856.1	333.09	335.93	68.53	323.91	131.8	0.45	0.1
12	51.02	18.84	14.79	3.63	8.71	4.53	0	0
13	39.43	20.91	0.19	0.01	0	0	0	0
14	737.24	216.39	421.49	45.1	355.5	99.79	0	0
15	69.29	27.92	77.47	9.79	12.68	4.92	0	0
16	1396.26	390.57	624.97	71.03	674.56	186.45	2.75	1.36
17	196.9	72.18	249.77	21.51	75.93	26.73	0	0
<b>Focus Area Model</b>								
1	748.75	215.66	420.15	45.02	357.28	99.5	0	0
2	262.09	93.92	403.16	28.62	127.01	39.58	0	0
3	41.67	15.37	46.69	4.8	6.94	2.96	0	0
4	854.8	332.94	322.72	68.29	328.31	132.35	0.96	0.32
5	38.89	20.79	0.2	0	0	0	0	0
6	49.96	20.24	60.78	7.41	11.92	4.75	0	0
7	22.04	8.09	38.52	2.83	6.7	2.08	0	0
8	7.65	2.84	10.37	0.96	2.3	0.74	0	0
9	15.35	5.48	19.46	1.68	3.96	1.24	0	0
10	121.97	31.59	26.92	4.74	34.35	10.44	0	0
11	672.9	222.23	340.29	47.8	276.06	93.27	0.06	0.02
12	71.51	15.5	9.49	1.24	17.05	3.74	0	0
13	603.84	206.74	342.59	46.67	276.33	89.64	0.14	0.03
14	124.66	44.22	129.63	11.7	55.99	16.78	0	0
15	141.27	51.93	91.14	13.81	55.02	20.98	0	0

ID	100Y24H		100Y6H		25Y24H		5Y24H	
	Peak Flow (cfs)	Volume (ac-ft)						
16	142.04	52.43	88.84	13.98	55.05	21.1	0.46	0.09
17	1585.97	497.78	718.81	97	672.82	215.08	2.5	0.51
18	1694.43	551.01	791.91	111.1	686.85	236.5	2.64	0.6
19	1.55	0.4	1.86	0.16	0.73	0.21	0.16	0.05
20	36.38	12.6	15.27	3.33	15.9	5.41	2.3	0.66
21	12.9	6.56	12.1	1.05	5.34	1.5	1.3	0.32
22	173.88	108.39	102.39	32.7	133.15	66.86	0	0
23	398.89	99.03	18.48	6.48	57.74	17.72	0.01	0
24	130.78	48.23	110.21	12.92	56.99	18.94	1.79	0.5
25	1693.02	551.21	790.47	111.01	687.2	236.49	2.38	0.47
26	1571.61	555.89	877.21	118.98	789.08	276.92	2.98	0.4
27	83.94	15.03	2.24	0.8	2.45	1.27	0.31	0.19
28	67.72	15.38	10.83	1.51	15.58	3.75	0.4	0.17
29	202.68	56.33	55.62	6.53	75.03	17.09	0.62	0.27
30	1254.44	498.31	259.22	90.42	385.1	183.84	3.61	3.12
31	38.88	21.08	1.13	0.05	0.38	0.13	0.02	0.01
32	43.27	22.94	1.24	0.53	0.76	0.55	0.11	0.03
33	11.49	1.84	0.43	0.01	0.07	0.01	0.01	0
34	0.13	0.04	0.14	0	0	0	0	0
35	123.94	53.36	41.81	12.85	26.53	16.27	0	0
36	32.48	15.24	9.62	3.31	4.22	3	0.02	0.01
37	34.24	15.28	10.44	3.38	4.18	2.91	0.03	0
38	125.09	58.14	87.15	18.93	28.38	17.43	0.49	0.22
39	246.37	99	178.19	30.15	47.2	23.92	0.07	0.05
40	117.18	53.27	88.14	17.76	27.94	16.03	0.07	0.05
41	130.41	46.16	89.27	12.36	19.33	8.08	0	0
42	117.75	41.95	70.27	10.89	16.28	7.2	0.02	0.01
43	97.4	37.51	32.28	7.64	9.85	5.61	0	0
44	499.46	180.8	333.7	46.71	159.43	55.09	0	0
45	40.94	16.45	30.06	5.2	19.01	8.08	0.15	0.04
46	26.17	9.84	9.61	2.35	9.41	4.05	0.48	0.15
47	51.92	22.57	22.16	5.64	24.13	11.59	0.98	0.34
48	58.29	29.53	36.47	7.41	35.3	17.89	0.51	0.2
49	209.95	91.93	117.69	19.48	106.04	47.6	1.17	0.4
50	32.91	13.08	15.16	2.28	14.78	5.63	0.02	0
51	56.1	19.59	24.53	4.69	21.01	7.59	0.09	0.02
52	72.75	27.31	35.32	6.38	26.46	9.98	0.26	0.07
53	23.97	7.09	3.14	0.58	2.05	0.74	0	0

ID	100Y24H		100Y6H		25Y24H		5Y24H	
	Peak Flow (cfs)	Volume (ac-ft)						
54	243.07	97.73	180.88	30.17	47.35	23.48	0.09	0.03
55	0.71	0.24	0.94	0.04	0.37	0.08	0.02	0.01
56	244.86	98.39	180.28	30.18	47.37	23.7	0.09	0.02
57	1658.33	549.58	789.07	111.04	688.04	236.66	2.45	0.37
58	78.78	26.47	20.72	1.53	3.36	1.07	0	0
59	515.07	253.58	387.46	75.98	369.69	153.7	0.15	0.07
60	47.1	17.6	6.4	2.76	15.31	7.51	1.11	0.5
61	24.54	10.82	32.74	2.85	8.96	2.27	0	0
62	22.23	8.82	16.29	1.53	4.83	1.19	0	0
63	67.34	25.83	24	5.21	7.96	4.01	0	0
64	38.68	31.58	3.72	0.7	1.85	1.11	0.82	0.31
65	6.62	4.75	2.89	0.34	0.86	0.23	0.05	0.01
66	1052.63	439.14	203.74	85.19	316.53	156.11	0.54	0.43
67	181.98	76.54	31.65	15.53	36.52	27.74	7.92	4.45
68	756.36	309.21	296.56	66.56	313.38	129.87	0.1	0.06
69	132.98	56.12	37.04	13.11	41.13	18.78	3.2	1.71
70	214.81	63.54	49.07	7.84	75.29	19.68	1.15	0.62
71	1692.68	655	781.42	153.26	688.09	312.02	2.16	0.54
72	678.84	233.05	154.88	45.74	240.38	94.26	0.03	0.03
73	241.17	95.76	184.19	30.39	49.87	23.45	0	0
74	1933.11	636.75	1117.88	126.7	895.47	296.44	3.04	1.08
75	15.51	3.96	10.68	1.41	8.44	1.88	0.84	0.3
76	214.07	81.04	208.46	22.13	71.32	27.04	0	0
77	12.7	4.24	6.97	0.87	1.89	0.77	0	0
78	135.84	45.74	141.53	12.35	51.27	17.79	0	0
79	837.01	314.2	917.31	84.36	379.35	142.51	0	0
80	70.93	28.14	78.22	9.75	13.23	4.92	0	0
81	11.26	3.49	2.31	0.44	0.67	0.24	0	0
82	27.1	16.14	10.45	5.5	13.5	9.83	0	0
83	22.12	14.67	7.68	4.72	10.43	8.91	0	0
84	265.9	131.99	19.41	7.2	29.92	29.75	0	0
85	303.36	108.81	0.3	0.15	13.52	11.71	0	0
86	160.8	96.92	64.7	29.26	84.61	53.34	0.26	0.14
87	32.92	5.9	0.02	0	0	0	0	0
88	720.85	368.34	184.12	44.48	429.37	145.57	0	0
89	40.61	45.03	27.8	17.99	31.31	29.3	0.03	0.03
90	1567.54	553.51	733.13	116.55	780.69	274.58	0.75	0
91	0.04	0.01	0.6	0.02	0.01	0	0	0

### 2.3.2 Depth and Discharge Results

Flow depth and discharge results from the existing conditions FLO-2D modeling are shown on Figure 2-9 through Figure 2-16. These figures are for general illustrative purposes and not practical for obtaining detailed information at site-specific locations. For more detailed information, please see the digital data in Appendix B, which includes the grid-based results for maximum flow depth, maximum peak discharge, maximum velocity, and other FLO-2D output. The model output from the Focus Area submodel also includes the time-varying output that is compatible with the FLO-2D ArcGIS plug-in where hydrographs or water surface elevations can be queried at any location by drawing a line using the tool within the plug-in. The Lyon County contract engineer (DOWL) has access to the FLO-2D ArcGIS plugin-in tool developed by JE Fuller.

### 2.3.3 Animations

Google Earth animations of the Lower model have been included with the digital model results (see Appendix B). These animations are helpful for visualizing the dynamic nature of the flooding as it moves through the study area.

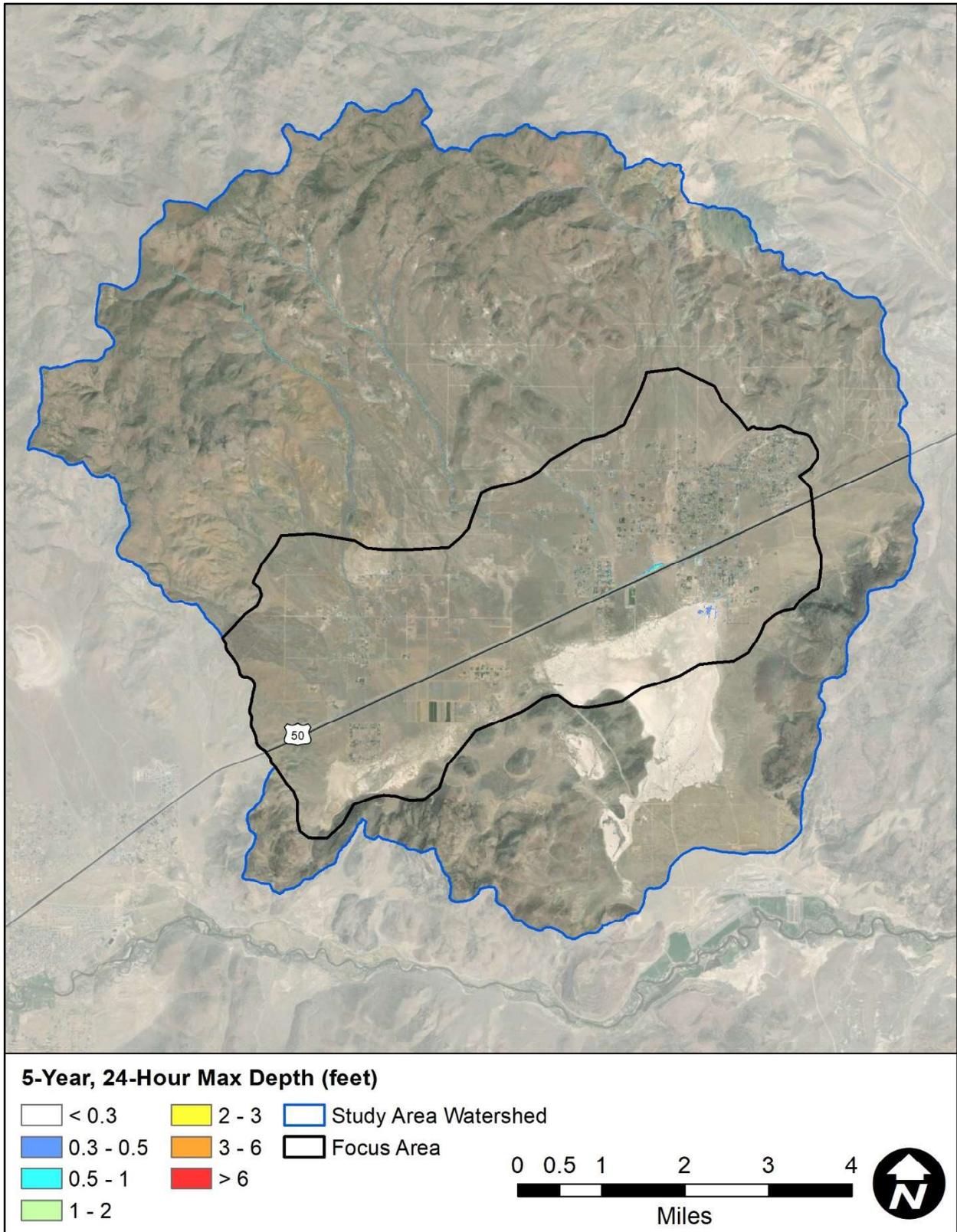


Figure 2-9. Base conditions 5-year, 24-hour depth results

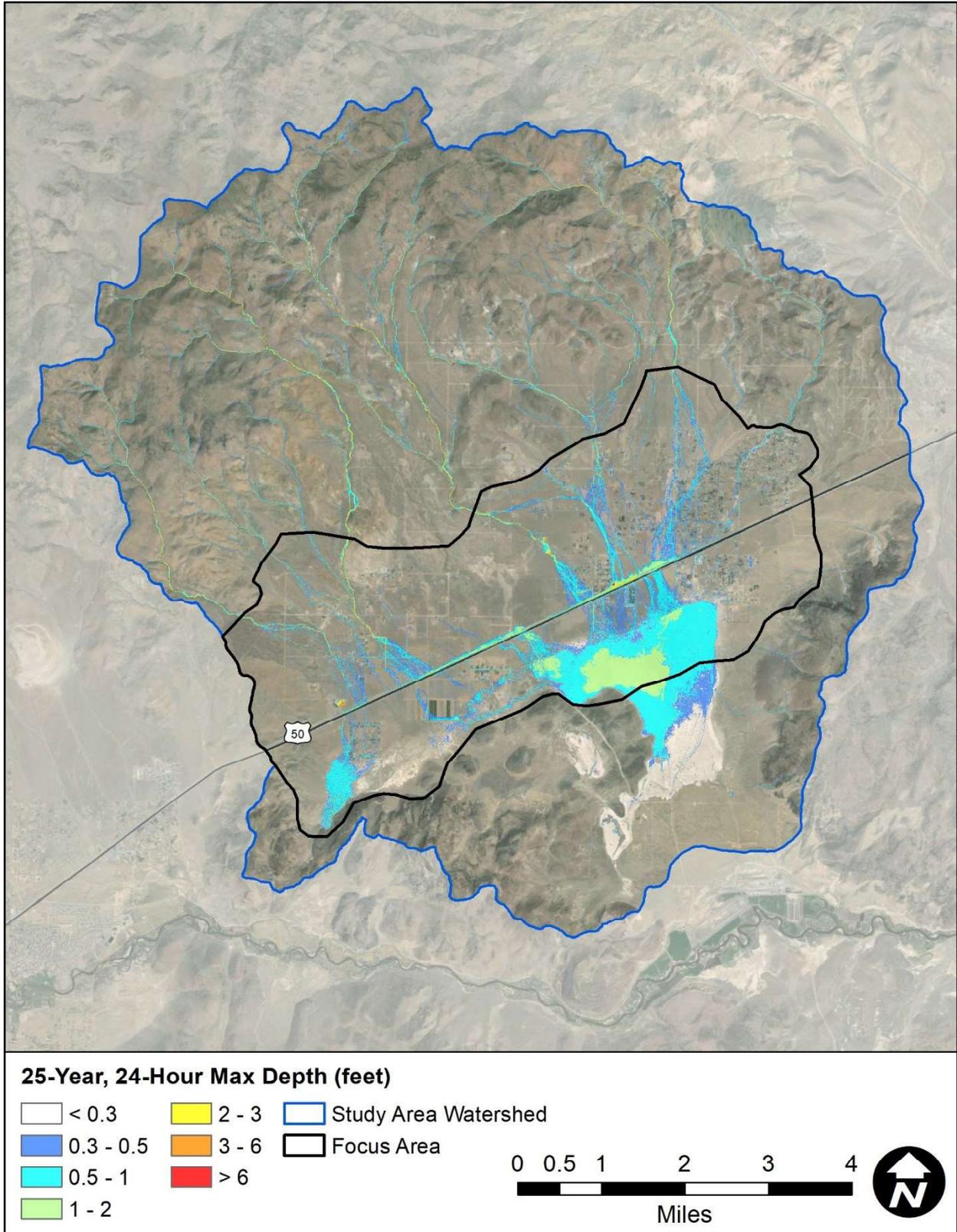


Figure 2-10. Base conditions 25-year, 24-hour depth results

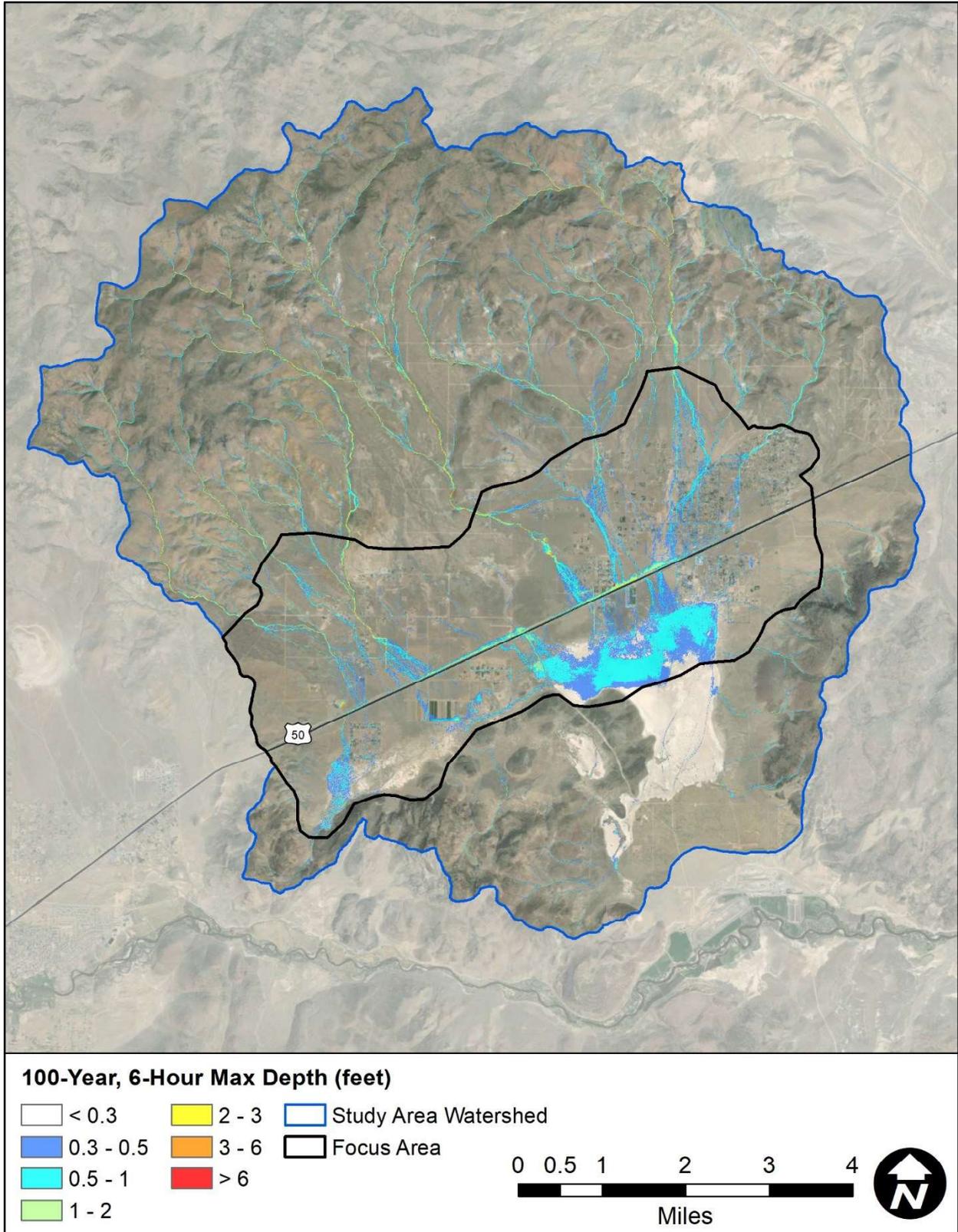


Figure 2-11. Base conditions 100-year, 6-hour depth results

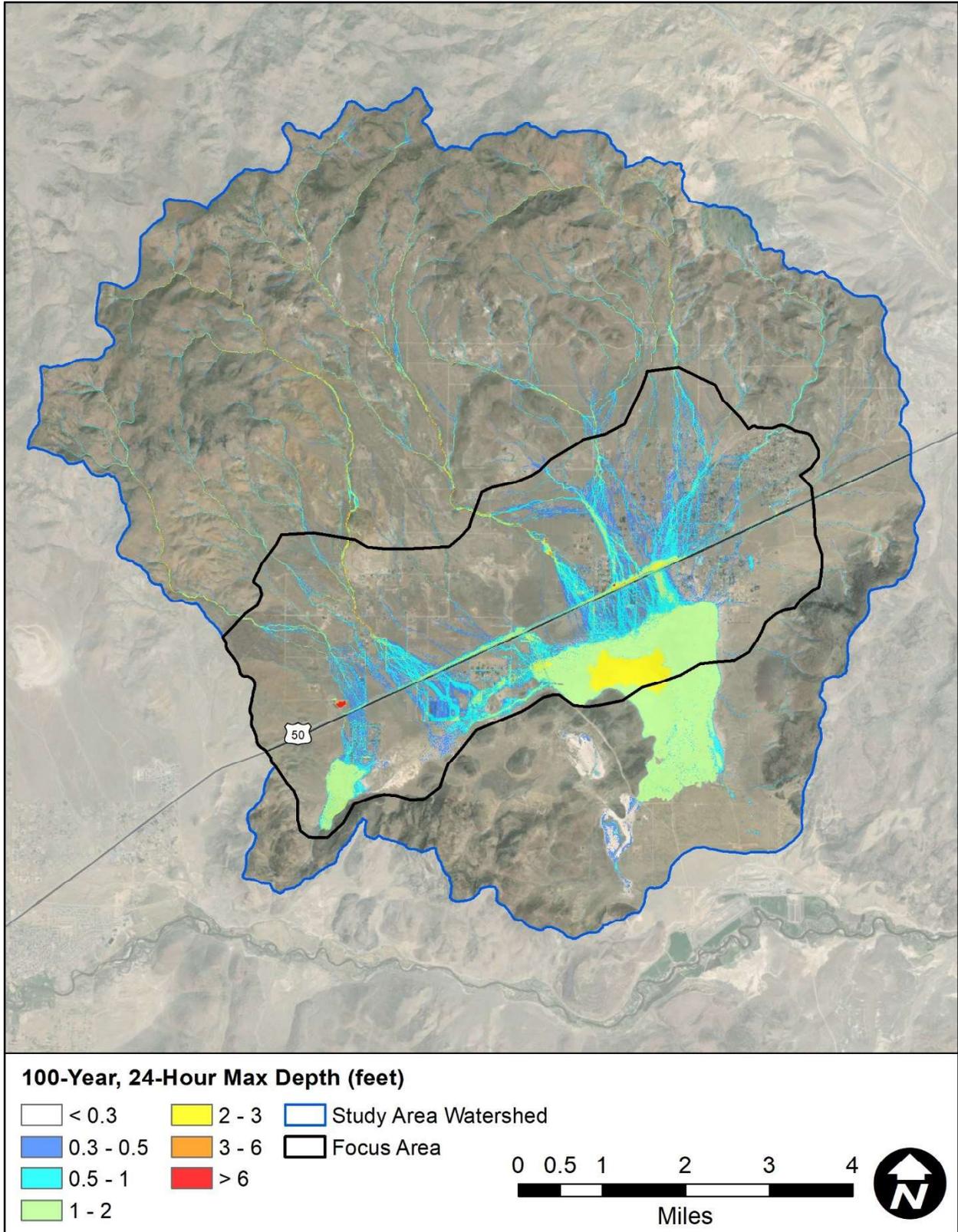


Figure 2-12. Base conditions 100-year, 24-hour depth results

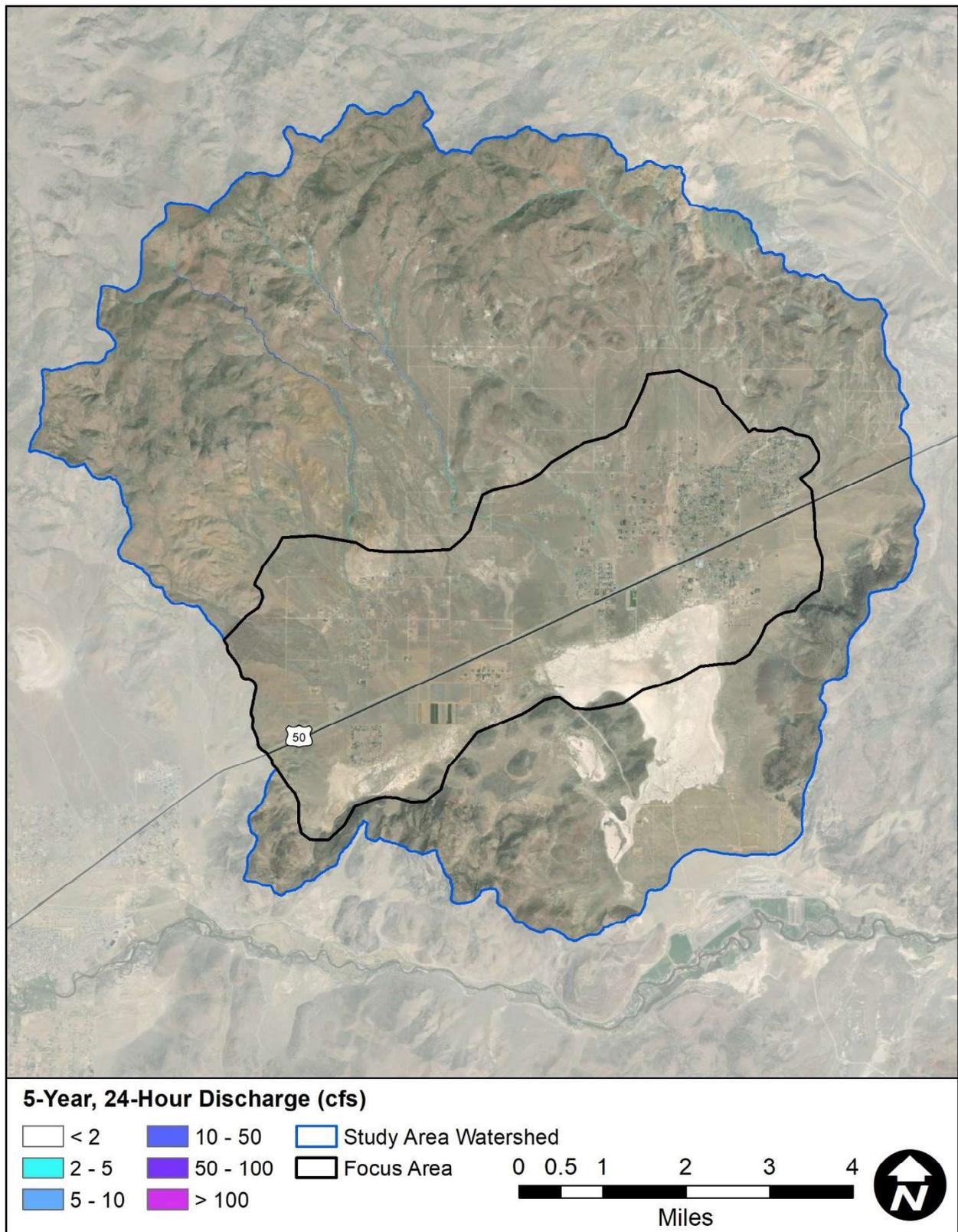


Figure 2-13. Base conditions 5-year, 24-hour discharge results

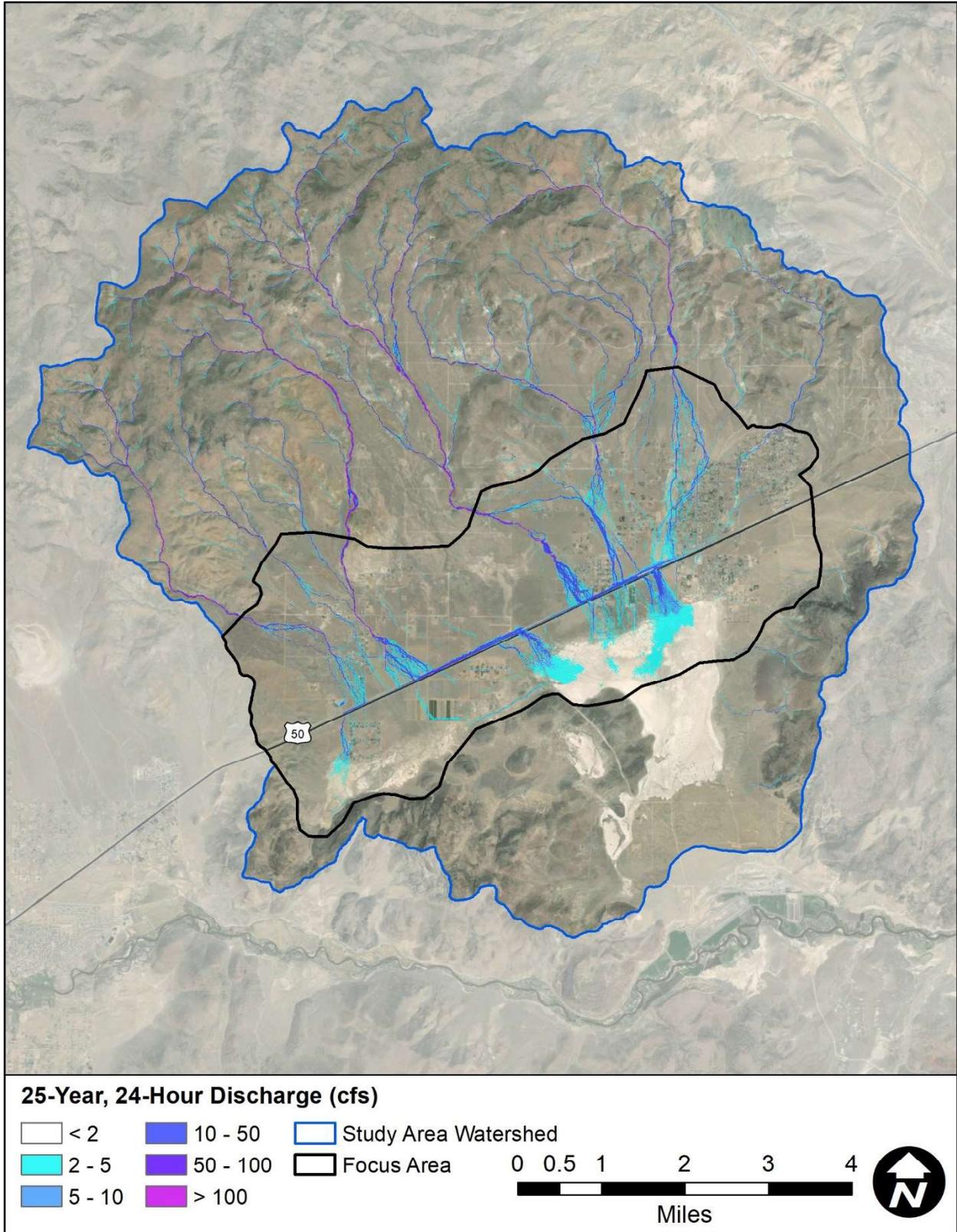


Figure 2-14. Base conditions 25-year, 24-hour discharge results

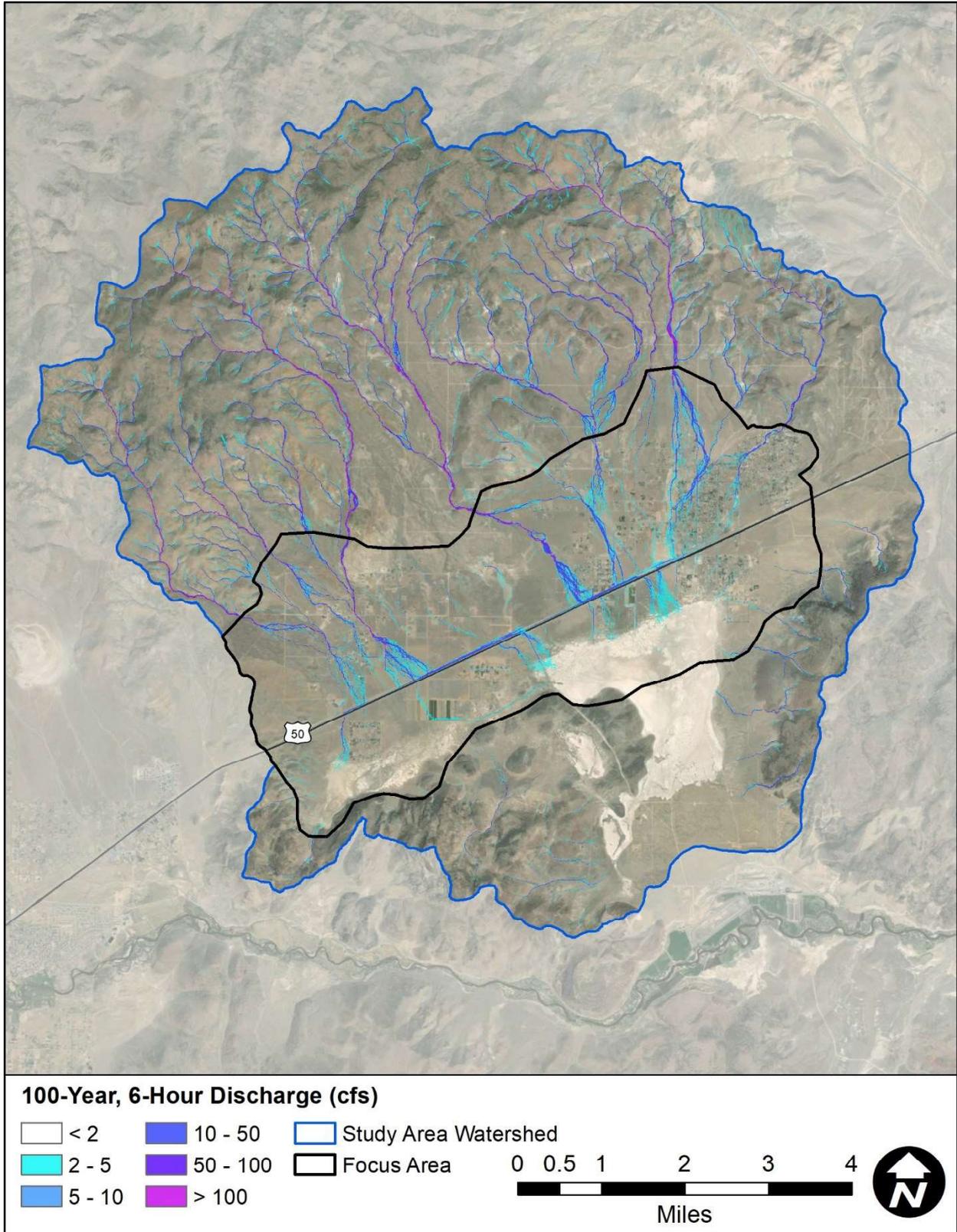


Figure 2-15. Base conditions 100-year, 6-hour discharge results

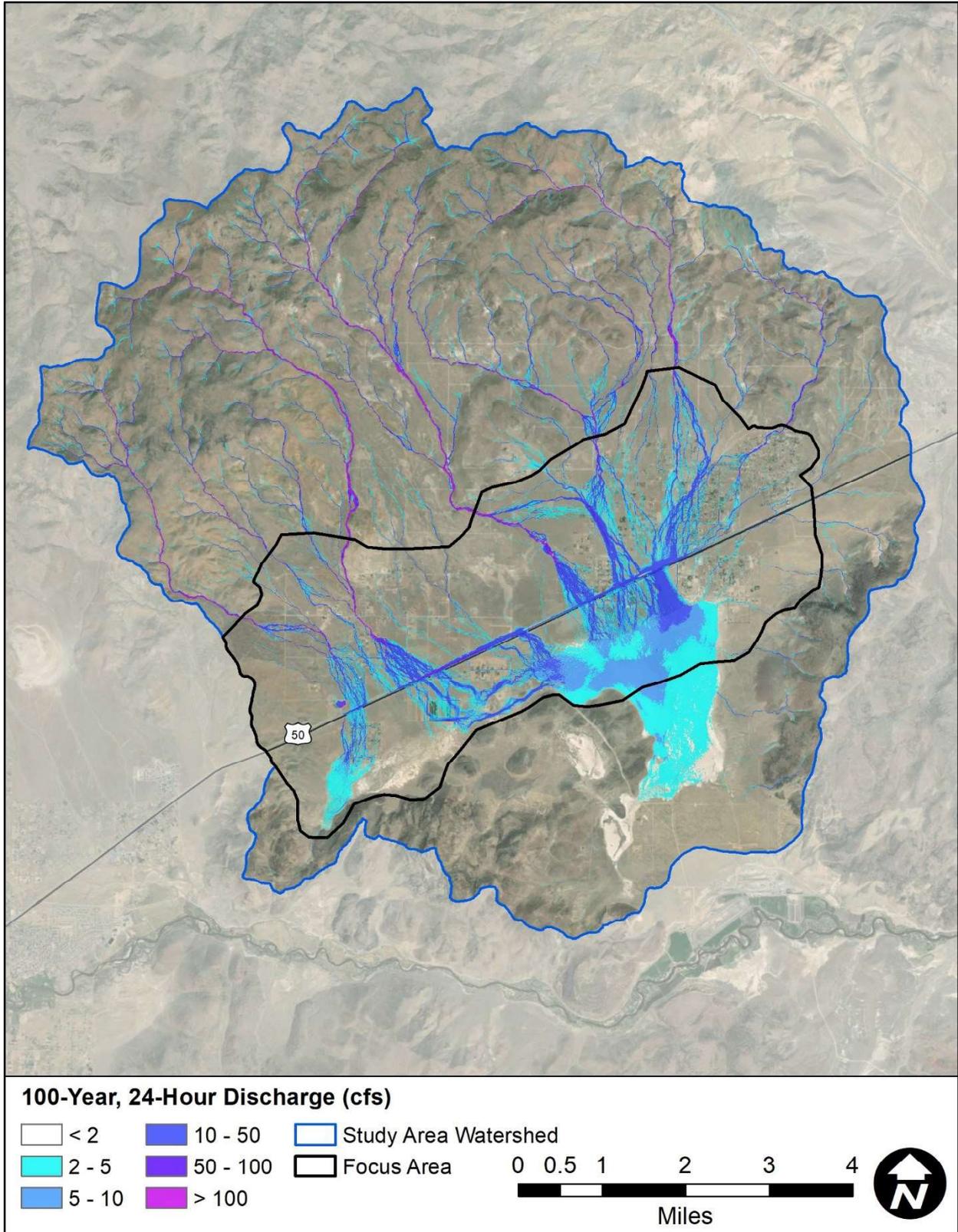


Figure 2-16. Base conditions 100-year, 24-hour discharge results

## 2.4 VERIFICATION OF BASE CONDITION RESULTS

### 2.4.1 Comparison with USGS Regression Equations

As a verification of model results, the 100-year 6- and 24-hour results at nine drainage basins were compared with the 100-year USGS regression equation, shown as Equation (1), for the Eastern Sierras Region 5 (USGS, 1997).

$$Q_{100} = 7000AREA^{0.782}(ELEV/1000)^{-2.18}(LAT - 28)^{4.6} \quad (1)$$

Where:

- $Q_{100}$  is the 100-year peak discharge (cfs)
- $AREA$  is the drainage area (square miles)
- $ELEV$  is mean basin elevation (ft)
- $LAT$  is the latitude of site (decimal degrees)

The results from this comparison are shown in Table 2-9 and Figure 2-17, while the basin locations used for this comparison are shown in Figure 2-18.

Table 2-9 shows both the 100-year 6-hour (labeled as 100Y6H) and the 100-year 24-hour (labeled as 100Y24H) peak flow results from the FLO-2D modeling compared with the 100-year flow from the regression equation. The unit discharges for each basin and the median values are also calculated and shown in the table.

The comparison indicates that the FLO-2D results for the entire study area are satisfactory. In Figure 2-17, all results fall below the USGS envelope curve and within the cloud of values. However, 100-year 24-hour storm results fall below the low- to middle-elevation study area line (which includes USGS Regression Regions 2-16, not just Region 5) for the smaller drainage areas in the study area, while the results from larger drainage areas rise above the line. Results from the 100-year 6-hour storm show a similar pattern to the 100-year 24-hour storm.

When compared with the 100-year discharge relation for Region 5, the results are similar. The 100-year 24-hour storm results follow the slope of the line, but all results fall above the line. The 100-year 6-hour storm results show the same pattern as the 24-hour storm. These results may indicate that the results are slightly conservative for the 100-year event. However, it should be noted that the USGS used mean values for variables other than drainage area when plotting this line. Therefore, this plot may not appear to fit the data.

Table 2-9. Comparison with 100-year USGS regression equation

Basin ID	Basin Area	Regression Peak Flow	Regression Unit Discharge	100Y6H Peak Flow	100Y6H Unit Discharge	100Y24H Peak Flow	100Y24H Unit Discharge
	mi <sup>2</sup>	cfs	cfs/mi <sup>2</sup>	cfs	cfs/mi <sup>2</sup>	cfs	cfs/mi <sup>2</sup>
1	4.48	875	196	625	140	1,396	312
2	7.60	1,415	186	1,169	154	1,929	254
3	4.95	1,284	259	336	68	856	173
4	0.68	289	425	142	209	135	198
5	3.89	872	224	924	237	830	213
6	2.21	752	340	184	83	235	106
7	2.66	573	216	421	159	737	277
8	1.36	391	286	406	298	261	192
9	1.13	361	318	309	273	191	169
		<b>Median:</b>	<b>259</b>	-	<b>159</b>	-	<b>198</b>

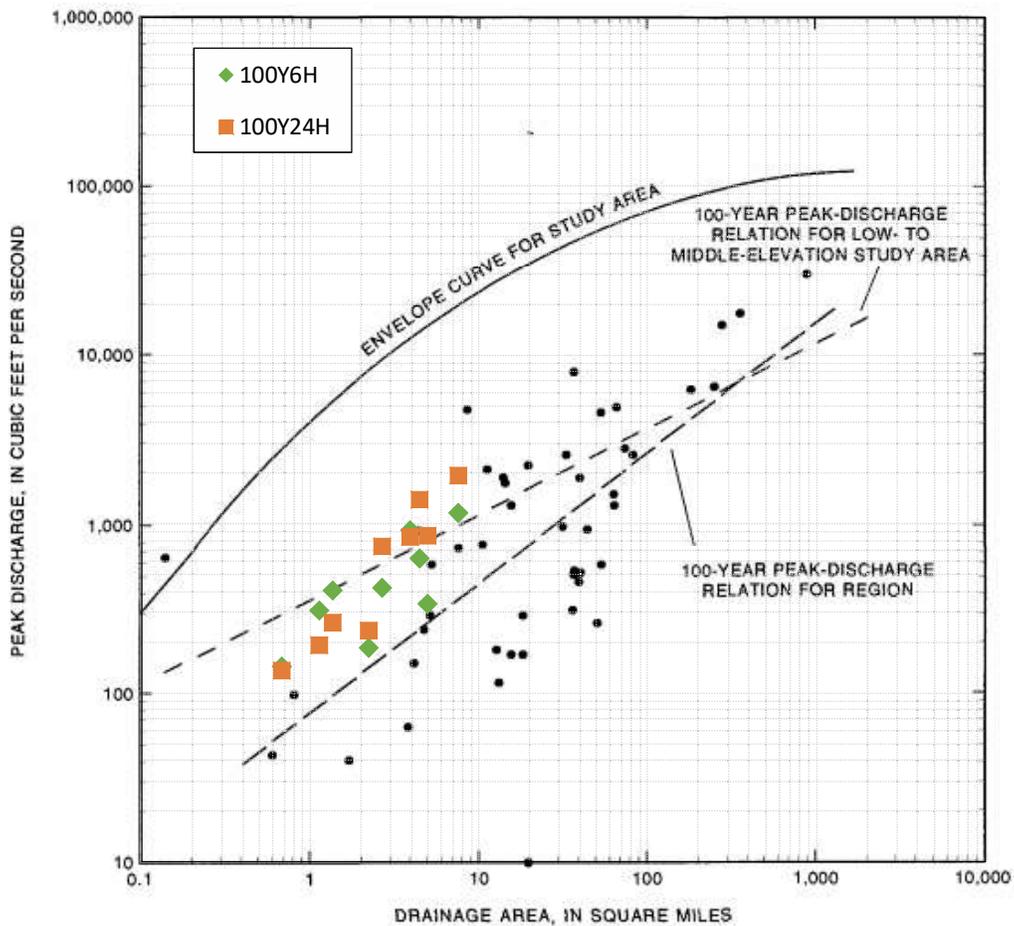


Figure 2-17. Comparison of FLO-2D results with the relations between 100-year peak discharge and drainage area and plot of maximum peak discharge of record and drainage area for gaged sites in the Eastern Sierras Region 5, adapted from USGS (1997).

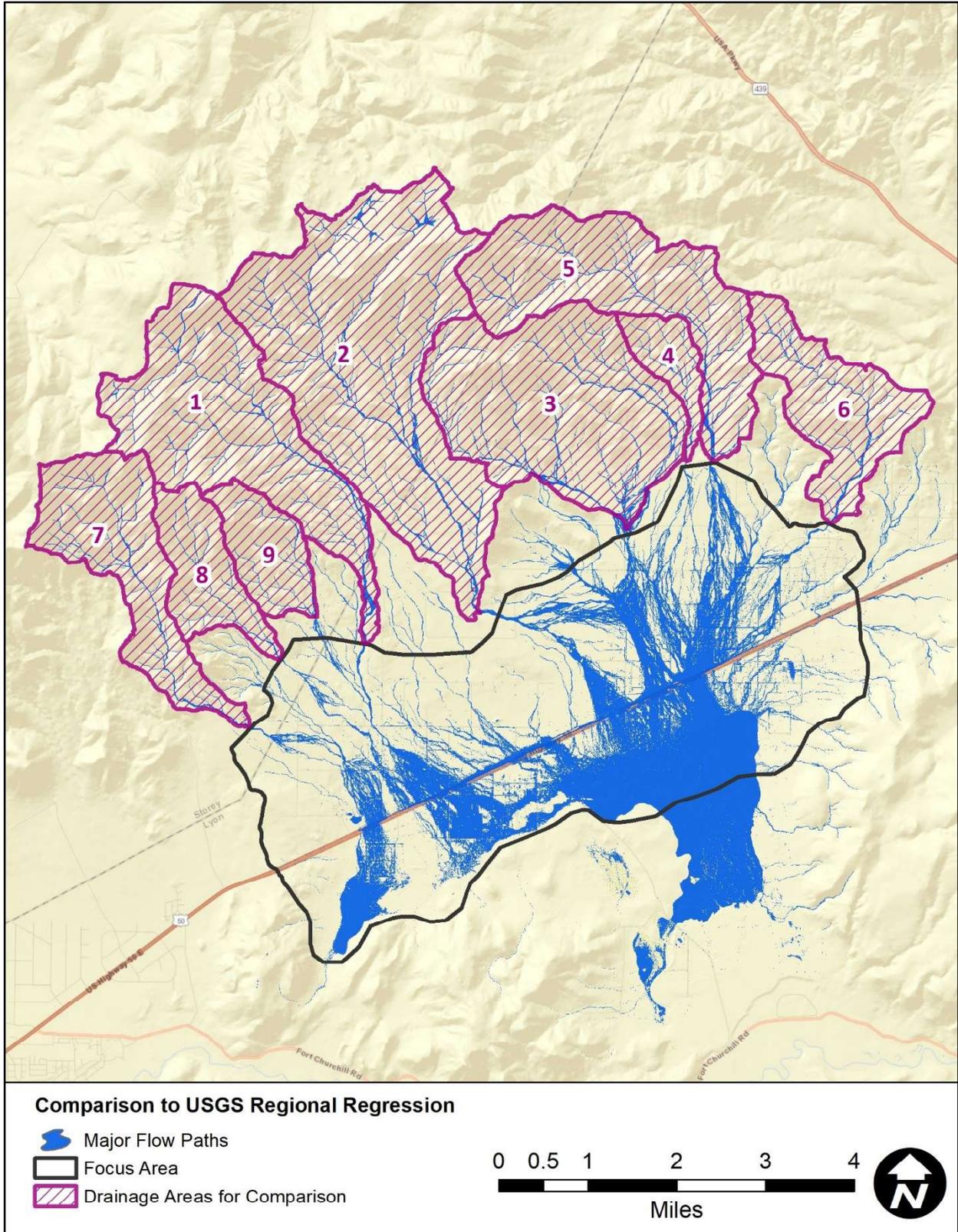


Figure 2-18. Drainage basins used for comparison of FLO-2D results to the USGS 100-year regression equations

### 2.4.2 Additional USGS Data

As part of another hydrology update project, NDOT obtained peak flow estimates for both inactive and active crest stage sites. Since these were crest gages, they contained only peak flow estimates rather than entire hydrographs.

The maximum peak of record for each gage was parsed from the USGS data peak flow data, while the drainage area was collected from each gage's site description on the USGS website. However, not all gages listed the drainage area in the site description. Of the 216 total sites, forty-five did not list the drainage area, and these sites were excluded from the comparison to the ADMP peak results. A summary of the drainage area statistics for all 216 sites is shown in Figure 2-19, and the spatial location of the sites is shown in Figure 2-20.

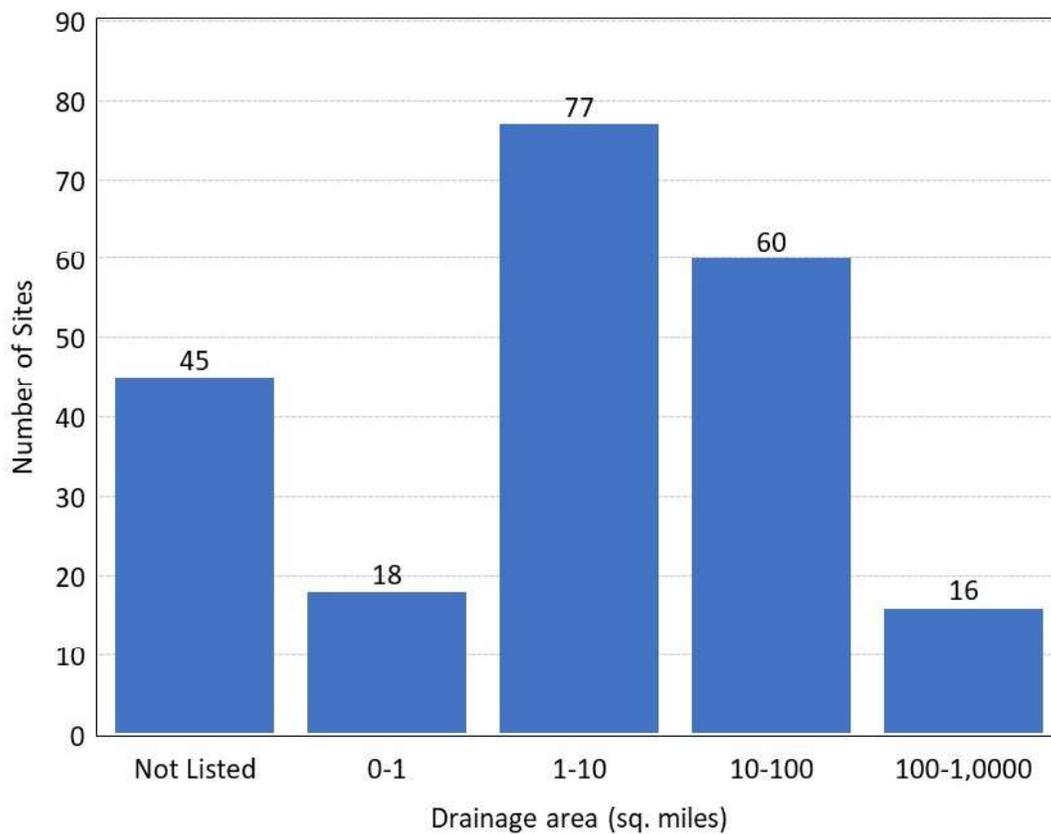


Figure 2-19. Drainage area statistics for USGS crest stage sites

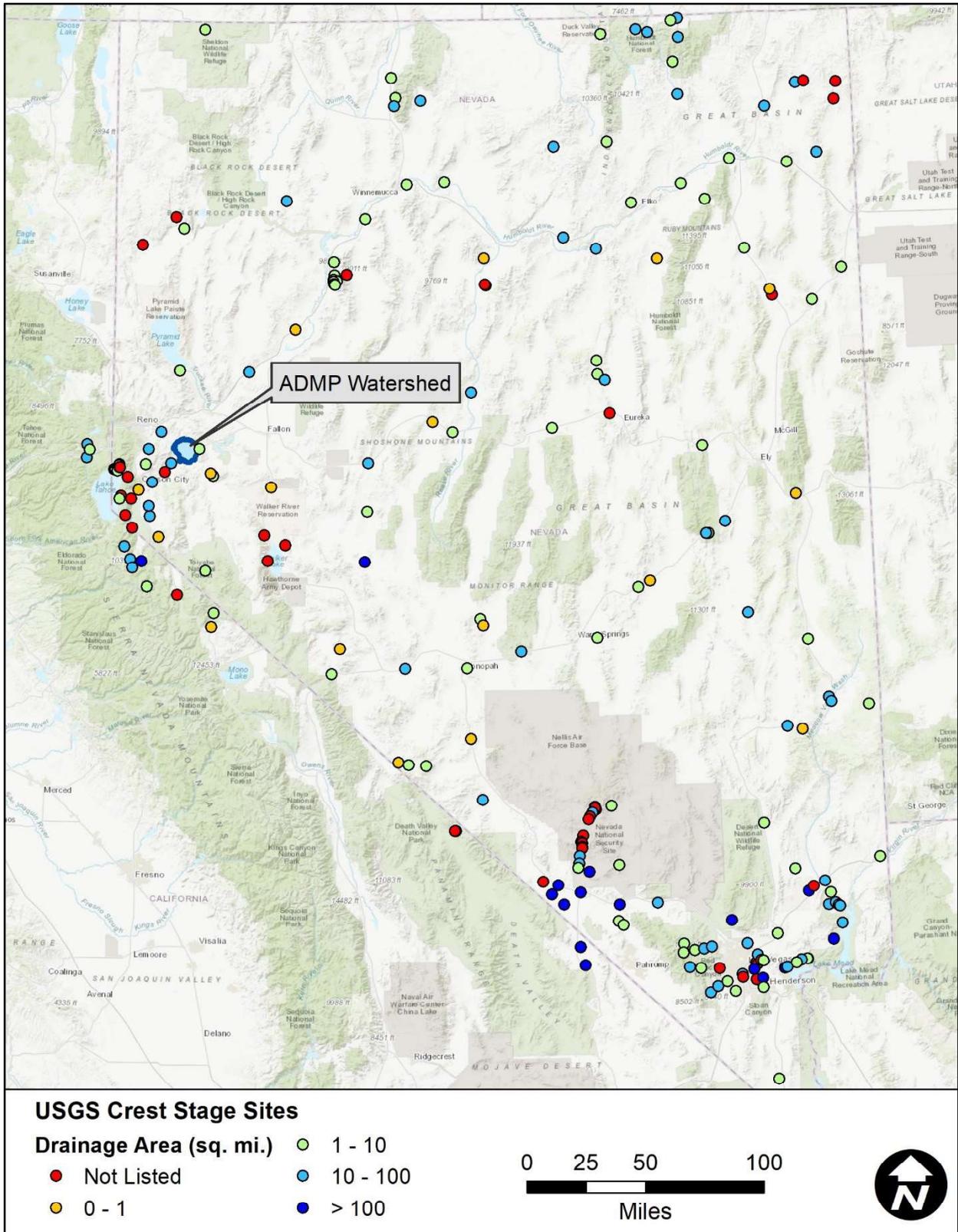


Figure 2-20. Location and drainage areas of USGS crest stage sites

As an additional verification of the peak flow estimates, the crest stage flow peak estimates were compared with the 100-year, 24-hour and 100-year, 6-hour FLO-2D results and peak flow estimates from the 1997 100-year regression equation (Equation 1). This comparison is shown as Figure 2-21. As before, both the FLO-2D and the 1997 100-year regression estimates fall within the cloud of data, which provides another indicator that the ADMP results are reasonable.

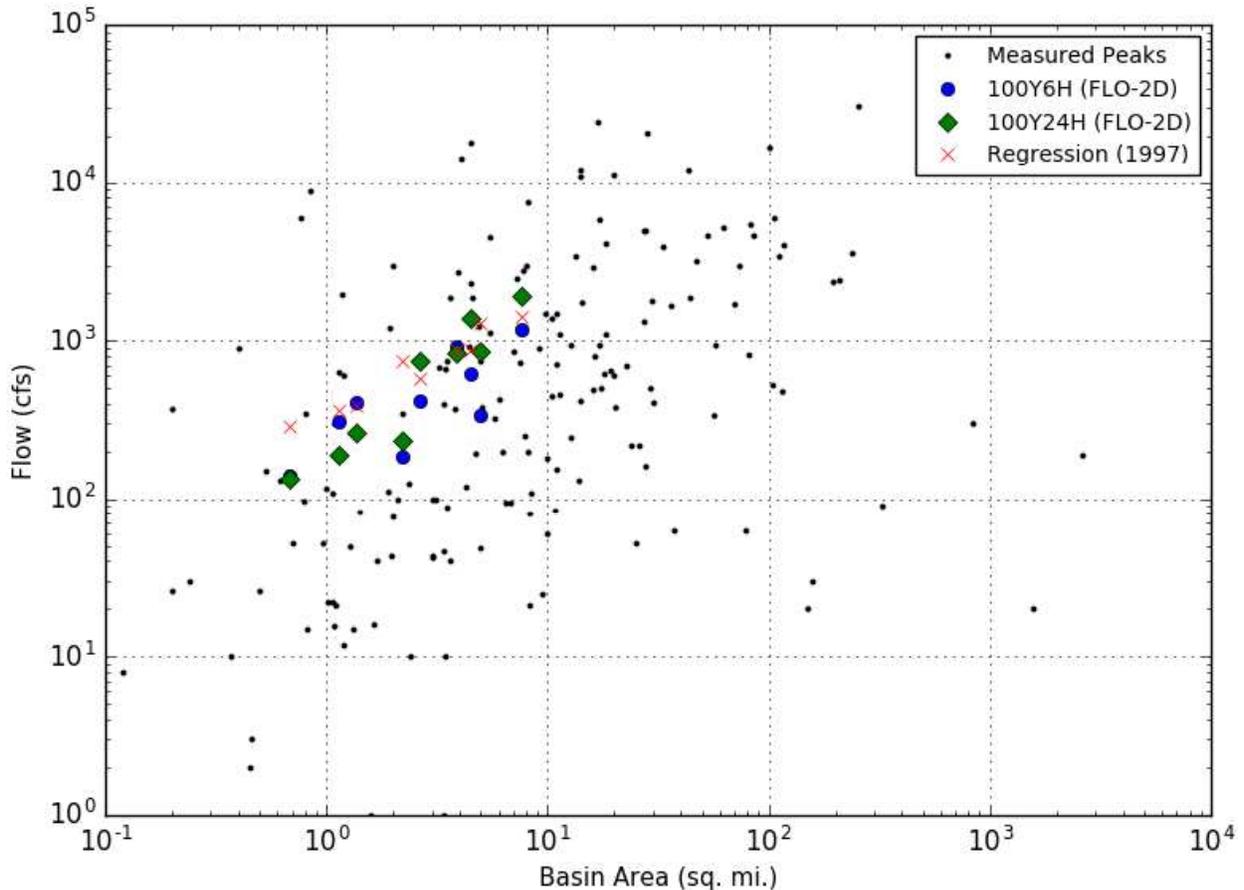


Figure 2-21. Comparison of FLO-2D results, 1997 100-year regression equation, and peak flow estimates from crest stage sites

### 2.4.3 Historical Flooding Documentation

As a part of the public outreach effort, the project team collected information of historical flooding from residents within the ADMP study area. This information was used to help verify and adjust the FLO-2D model if needed. In general, the model results corresponded well with the historical information. Four representative examples from different parts of the study area were chosen to highlight the correlation between model results and actual flooding and are shown in Figure 2-22 through Figure 2-25.

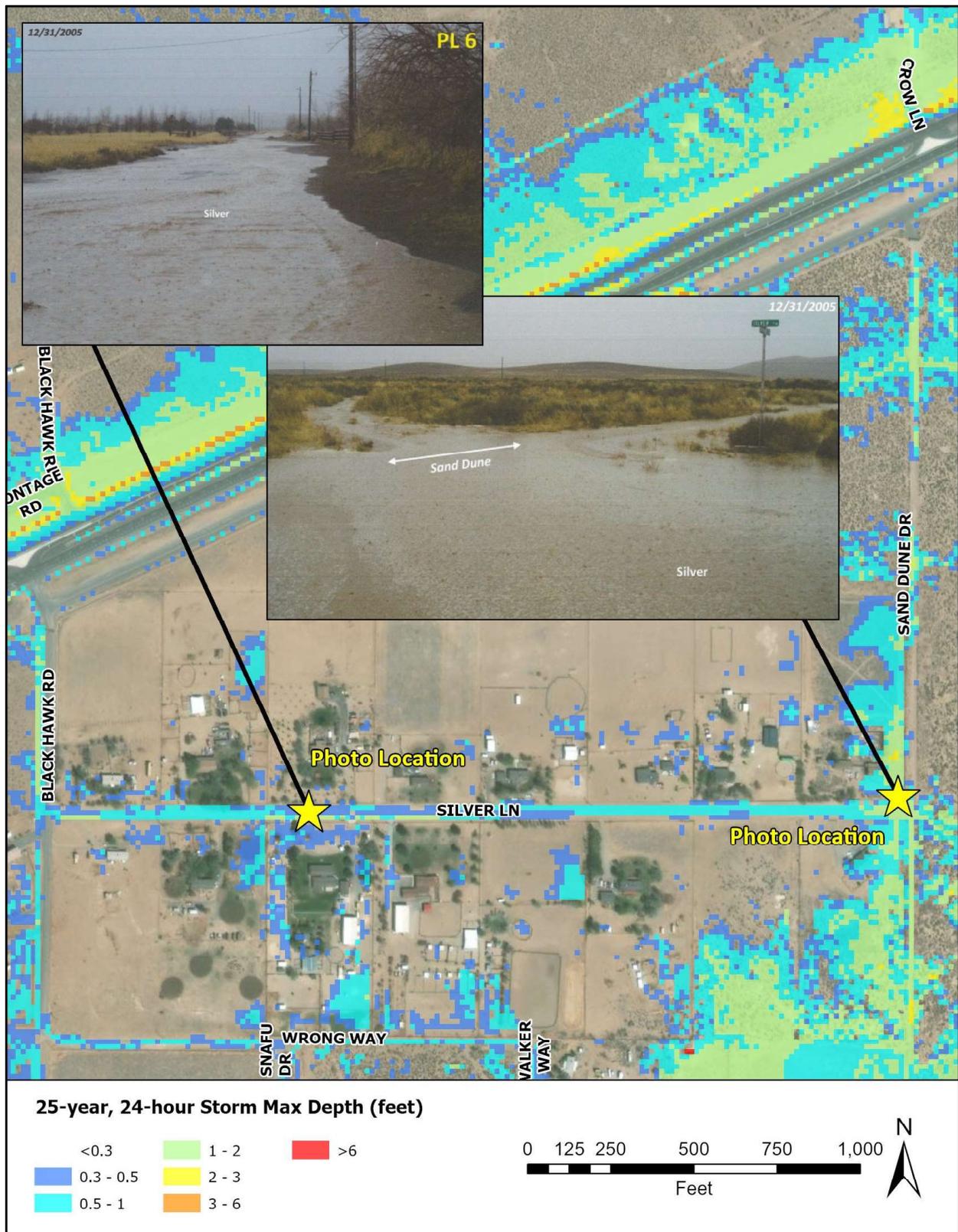


Figure 2-22. Resident flood record along Silver Lane

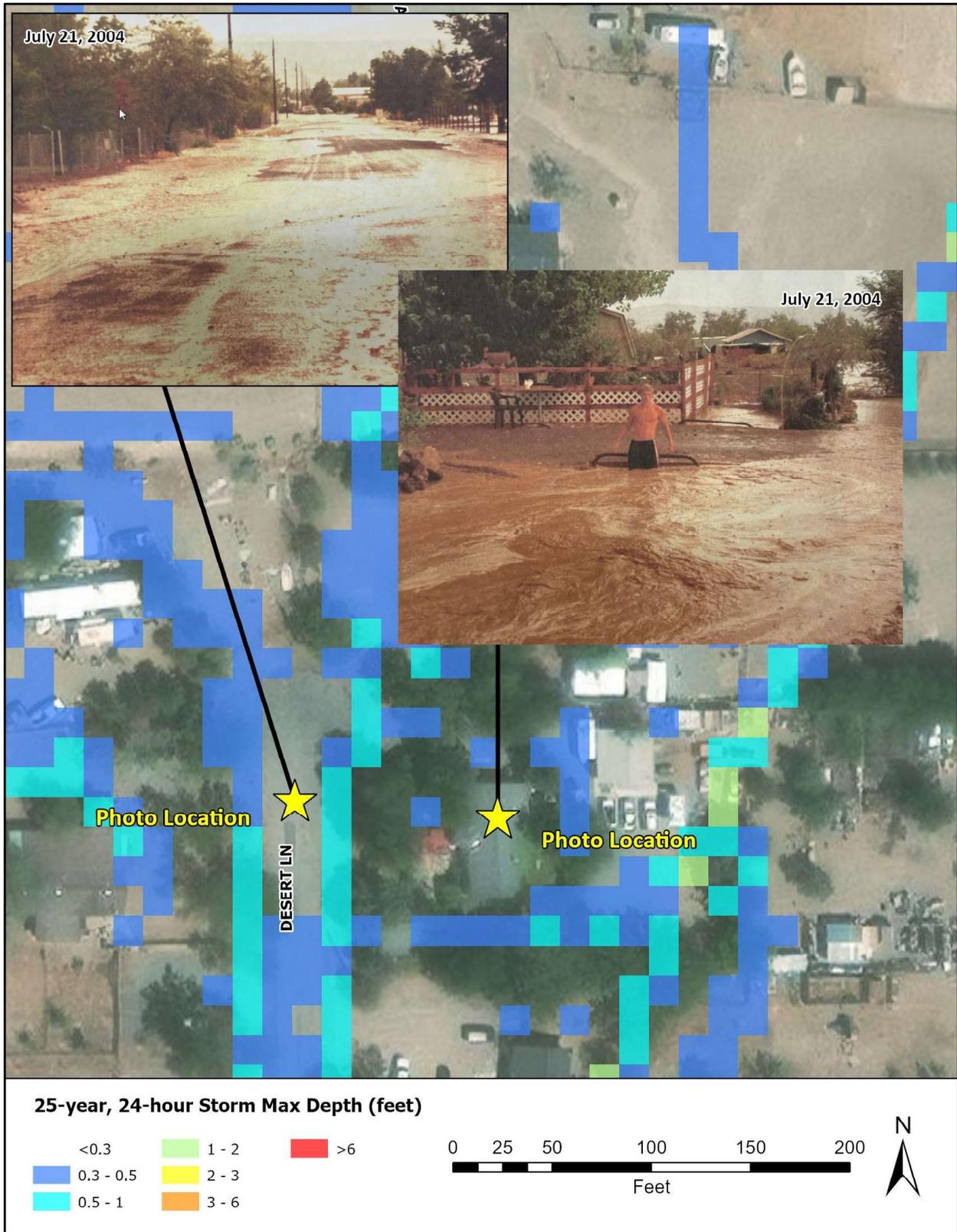


Figure 2-23. Resident flood record near Desert Lane

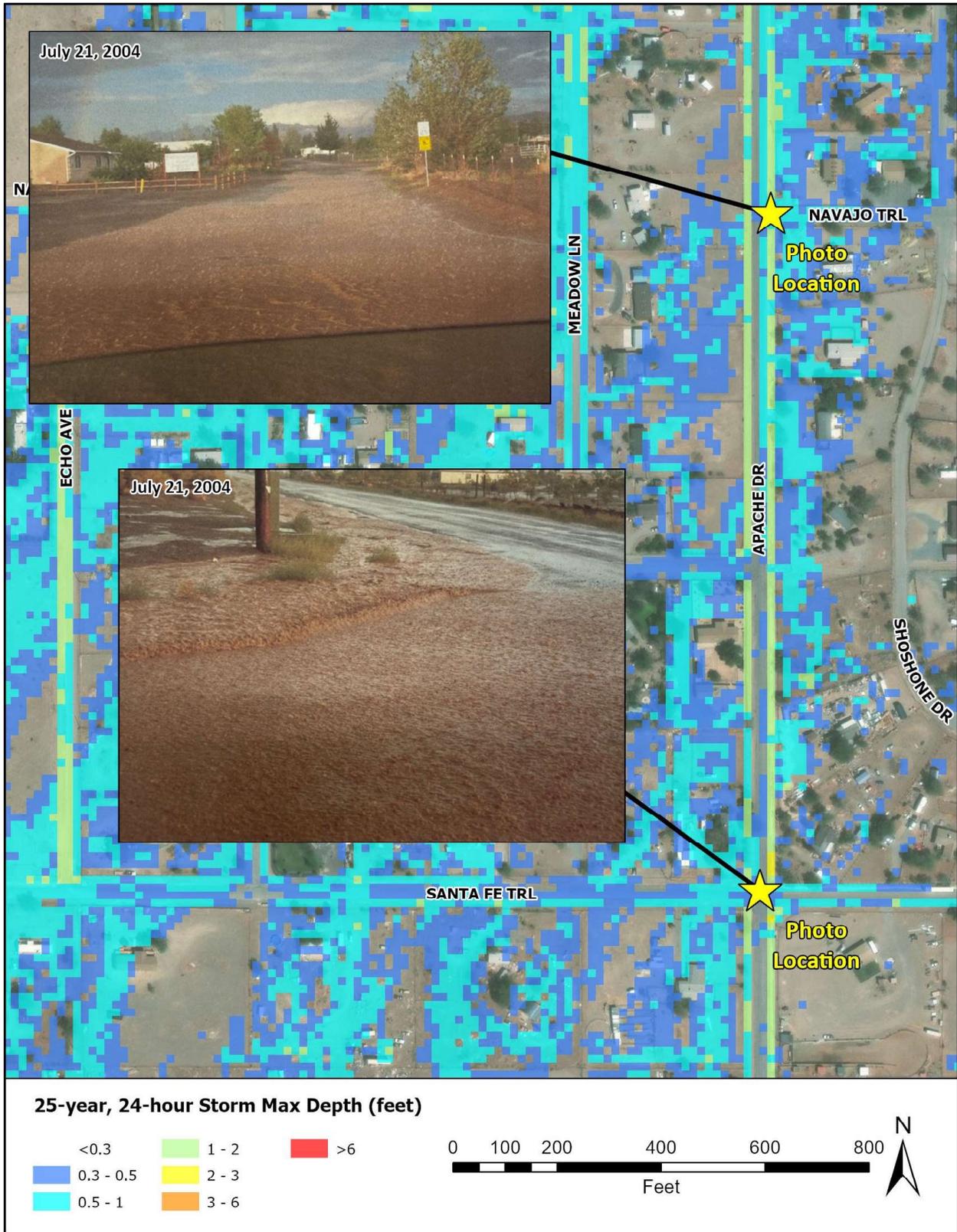


Figure 2-24. Resident flood record along Apache Drive

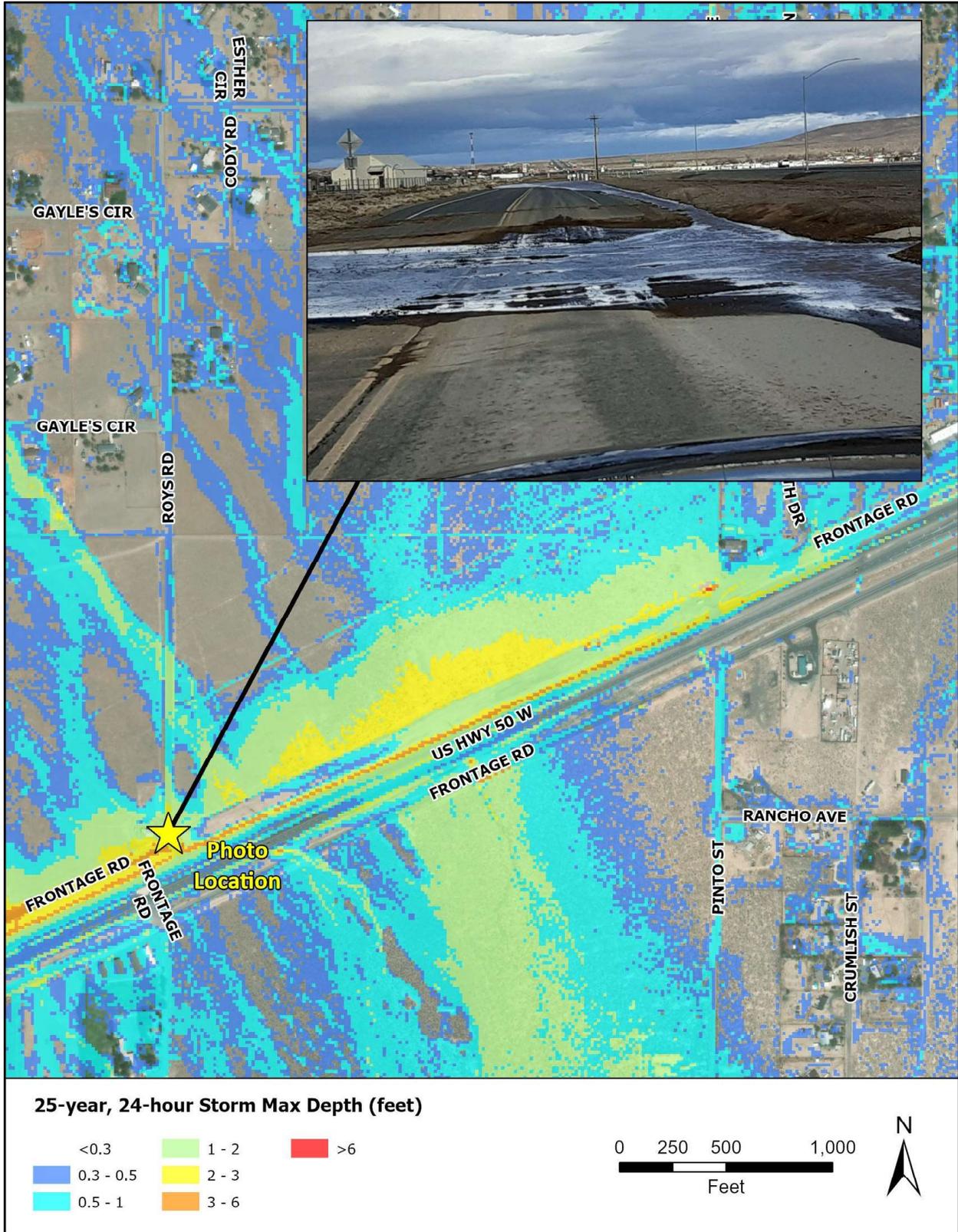


Figure 2-25. Resident flood record at Roys Road and Highway 50 Frontage Road

#### 2.4.4 Note on 5-year 24-hour Results

As can be seen in the above results, the 5-year 24-hour storm event produces very little runoff. After detailed inspection of the input data, these results make sense. Figure 2-26 shows a comparison of the cumulative rainfall, the corresponding rainfall intensity, and the average XKSAT for the Upstream and Focus Area submodels. When the rainfall intensity is less than the XKSAT, no runoff will occur. All rainfall will be infiltrated until the limiting infiltration depth is reached. Since the rainfall intensity only exceeds the XKSAT for a short period of time in the Upstream model, only minor runoff is produced. In the Focus Area submodel, no<sup>6</sup> runoff is produced from on-site rainfall, since the average XKSAT far exceeds the rainfall intensity. Most of the flow in the Focus Area submodel would be from off-site inflow from the Upstream submodel. However, since transmission losses are still happening (until the limiting infiltration depth is reached) as the water flows over each cell, a high percentage of all rainfall is infiltrated during this event.

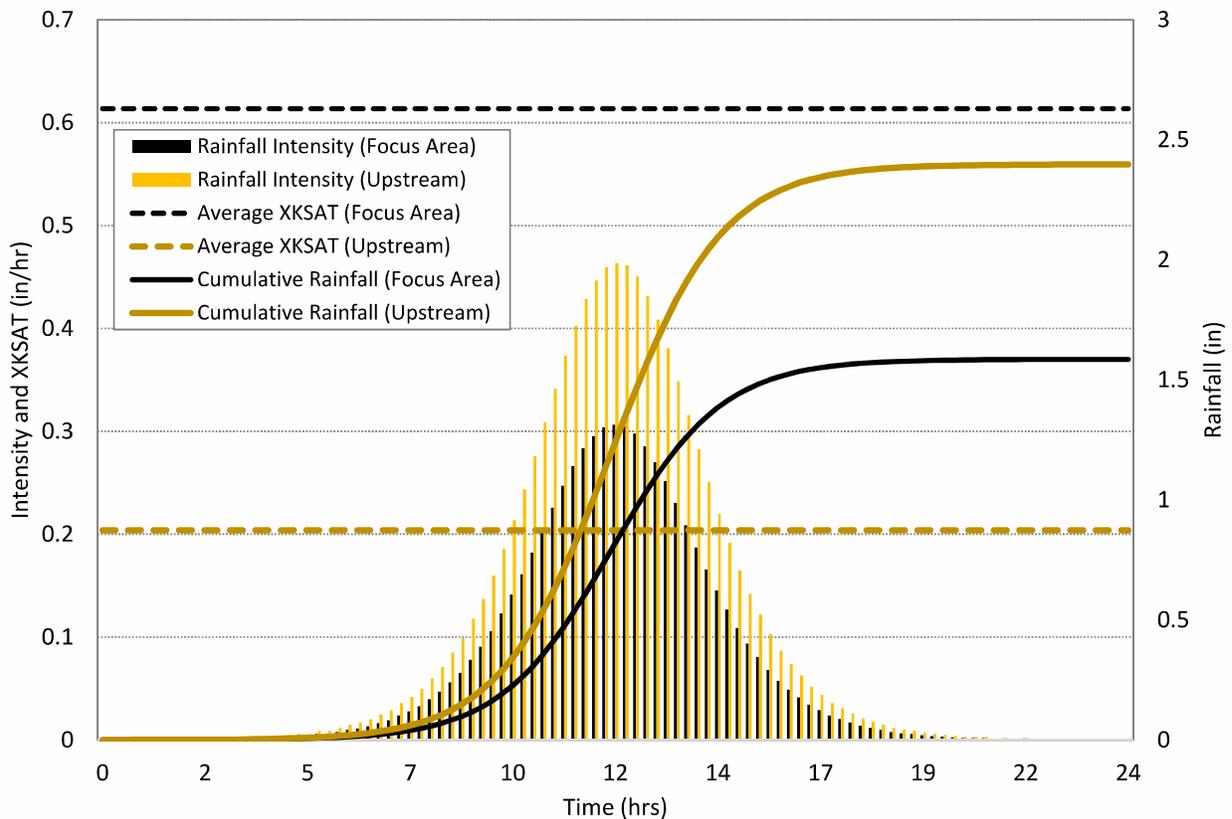


Figure 2-26. Comparison of cumulative rainfall, rainfall intensity, and the average XKSAT for the 5Y24H event

<sup>6</sup> Note that areas with high percent imperviousness, such as buildings and streets, would still have runoff. Also, an average XKSAT is shown, some areas have lower XKSAT values and would have runoff.

## 2.5 BASE CONDITIONS MODELING SUMMARY

The base (or existing) conditions FLO-2D models were created using the best available information for land cover, land use, topography, and hydrology that were available at the time of the study. Every effort was made to ensure the models represented existing conditions as of the date of the LiDAR topography.

Photographs and anecdotal flood information collected from the residents within the community were used to help calibrate and verify the modeling results. Like all models, these base conditions FLO-2D models are a simulation of potential conditions that could occur during a range of storm events. The models cannot exactly replicate actual, observed storm events at all locations within the watershed due to the vast number of variables that change with each unique storm event.

The modeling results reflect the complex flooding and sedimentation hazards that exist within the study area. The results provide valuable, quantitative, detailed information on which future planning and development decisions can be based. The existing conditions models also serve as a foundation from which potential mitigation alternatives can be assessed (Section 4.4).

Although the ADMP FLO-2D modeling effort was not intended to replicate an actual historical flood event, the comparison of the modeling results with USGS regression equations and anecdotal flood information indicate the project FLO-2D models suitably depict the design storm runoff conditions. Given the distributary nature of the flooding within the community, and the high sediment transport rates, flooding characteristics (e.g., depth, discharge, location) are likely to change from one flood event to the next. Even small anthropogenic changes to the landscape (e.g., dirt piles, berms, construction of outbuildings, landscaping debris piles, etc.) will result in sediment accumulation, channel scour, and changes in flowpath directions that may not be represented in the project FLO-2D modeling. In other words, the results of the modeling represent potential flooding conditions as of the date of the project topographic mapping. Updated mapping and FLO-2D modeling are recommended if major changes to the landscape occur in the future. Additionally, a flowpath uncertainty analysis was developed to inform on potential flood hazards if flowpaths were to change direction over time through aggradation or scour.

## 2.6 FLOWPATH UNCERTAINTY

An avulsion is the process by which flow is diverted out of an established channel into a new course on the adjacent floodplain (Slingerland & Smith, 2004). Avulsions divert flow from one channel into another, leading to a total or partial abandonment of the previous channel (Field, 2001; Bryant et. al., 1995), or may involve simple flowpath shifts in a braided or sheet flooding system (Slingerland & Smith, 2004). Avulsions are commonly associated with alluvial fan flooding but are also known to occur on riverine systems and river deltas (Slingerland & Smith, 2004).

The occurrence of avulsions is what makes an alluvial fan “active.” Avulsions give the alluvial fan the ability to distribute water and sediment over the surface of the landform, which results in the radial “fan” shape. Avulsions influence flood hazards on alluvial fan landforms by changing the location, concentration, and severity of flooding on the fan surface. That is, an area not previously inundated by flooding (or inundated only by shallow flow) may in a subsequent flood become the locus of flood inundation, sediment deposition, and/or erosion. If an alluvial fan has no risk of avulsion, flood hazard

delineation and mitigation become much simpler engineering problems, consisting only of modeling two-dimensional flow and/or normal riverine hydraulic and sedimentation issues.

The occurrence of major avulsions in an alluvial fan drainage system introduces the following complications into an engineering analysis of the flood hazard:

- Uncertain and changing flowpath locations, during and between floods
- Continually changing channel and overbank flowpath topography
- Inundation and/or sedimentation hazards in previously un-flooded areas
- Uncertain and changing flow rate distribution for areas downstream of avulsions
- Uncertain and changing watershed boundaries for areas downstream of avulsions
- Aggrading, net depositional land surfaces and channels with diminishing capacity
- Unsteady, rapidly varied flow conditions
- High rates of infiltration and flow attenuation across the fan surface

As discussed previously in Section 1.4, the ADMP focus area is comprised of multiple active alluvial fan landforms. The flowpath uncertainty issue was addressed in this analysis by the use of the maximum flood hazard two-dimensional hydraulic modeling results. Flowpath uncertainty is caused by abrupt channel avulsions that occur during flood events. The cause of the channel avulsion can vary from channel aggradation (sedimentation) causing a rapid lateral shift in channel position, to overbank flooding carving a new channel, to upstream headcutting resulting in channel piracy. Regardless of the cause, the resulting abrupt change in channel position is something that is generally unpredictable and uncertain. The flowpath uncertainty analysis methodology addresses the channel avulsion potential element of the hazard analysis. Flowpath uncertainty modeling was conducted for this study to account for the potential hazards

### 2.6.1 Flowpath Uncertainty Modeling

The overall objective of flowpath uncertainty modeling was to force flooding in directions that would simulate avulsions, and to estimate maximum depths and discharges over the whole radial width of the alluvial fan area by modeling a series of “virtual” levees. The number, geometry, and alignment of the virtual levees were selected to achieve those objectives. Each virtual levee scenario was optimized to direct flow from a bifurcation point to a different area across the width of the alluvial fan. Adjustments to channel positions due to avulsions at an alluvial fan apex can have significant impacts to downstream developments. Given the coalescing nature of the alluvial fans, there are multiple scenarios.

The following criteria were considered when developing the virtual levees for the ADMP flowpath uncertainty analysis:

- **Levee Length.** The virtual levee lengths varied at each location. The lengths were determined based on professional judgment to achieve the objective of concentrating flows to various target locations downstream.
- **Number of Levee Scenarios.** The number of virtual levee scenarios modeled were dependent on the surface morphology and the downstream target objectives.
- **Alignment.** The virtual levees were aligned at moderate angles to the fan axis so that they did not cause a significant “pile up” of flow in the model results.
- **Coding.** The virtual levees were coded into the model to prevent overtopping (or failing) during the model simulations.

- **Model Iteration.** Multiple modeling integrations were performed to meet the target area objectives. Several virtual levees scenarios can be run within the same hydraulic model if the model results indicate there is no hydrologic inter-mixing of the two scenario results downstream of the virtual levees.
- **Final Hazard Delineation.** The maximum depth and velocity at each model grid cell from the maximum flood hazard modeling results were used as the final flood depth and velocity hazard results. In other words, the maximum flow depth at each grid cell was computed using the highest depth value considering all the scenarios. This approach was applied to all the grid cells in the model.
- **Conservative Results.** The virtual levee scenario employed for this analysis produces conservative flood depth and velocity results, particularly given the fact that actual avulsions do not typically divert the entire hydrograph along a particular alignment.

### 2.6.2 Model Development

While an existing (base) conditions hydraulic models depict the existing, fixed-bed condition of an X-year flood hazard event, it does not predict the full flood hazard associated within the active alluvial fan flooding and should not be the only scenario used to compute flow depths. To account for flowpath uncertainty, avulsion scenarios were developed and simulated within the model to account for the possibility of avulsions that would adversely affect (increase the inflow discharge) downstream.

The flowpath uncertainty scenarios were developed by reviewing existing flow bifurcations observed in aerial photography, topography, field reconnaissance, and the base hydraulic model. In locations where avulsions appeared likely or evidence of prior avulsions was observed, avulsions were simulated by adding berm-like features to redirect flow along an avulsion path. Figure 2-27 is an example of a flowpath uncertainty analysis with model results.

The flowpath uncertainty scenarios were modeled by redirecting flow with a hard barrier accomplished by artificially elevating ground topography. These barriers were given an arbitrary height well-above the ground elevation to ensure no overtopping. The barriers essentially were aligned to direct all the flow in the avulsion direction. Figure 2-28 shows the virtual levees and number of model scenarios used for the ADMP.

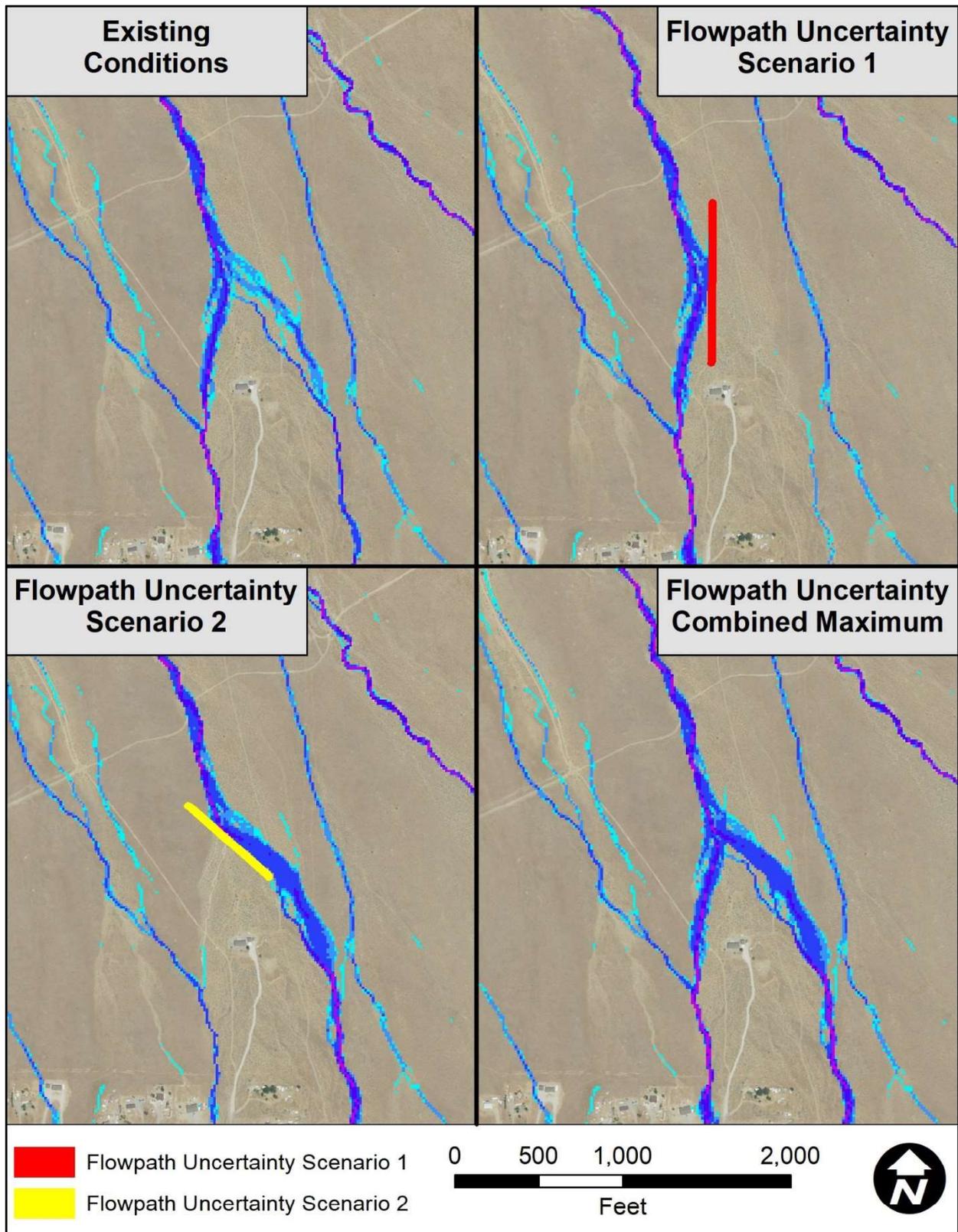


Figure 2-27. Flowpath uncertainty analysis example

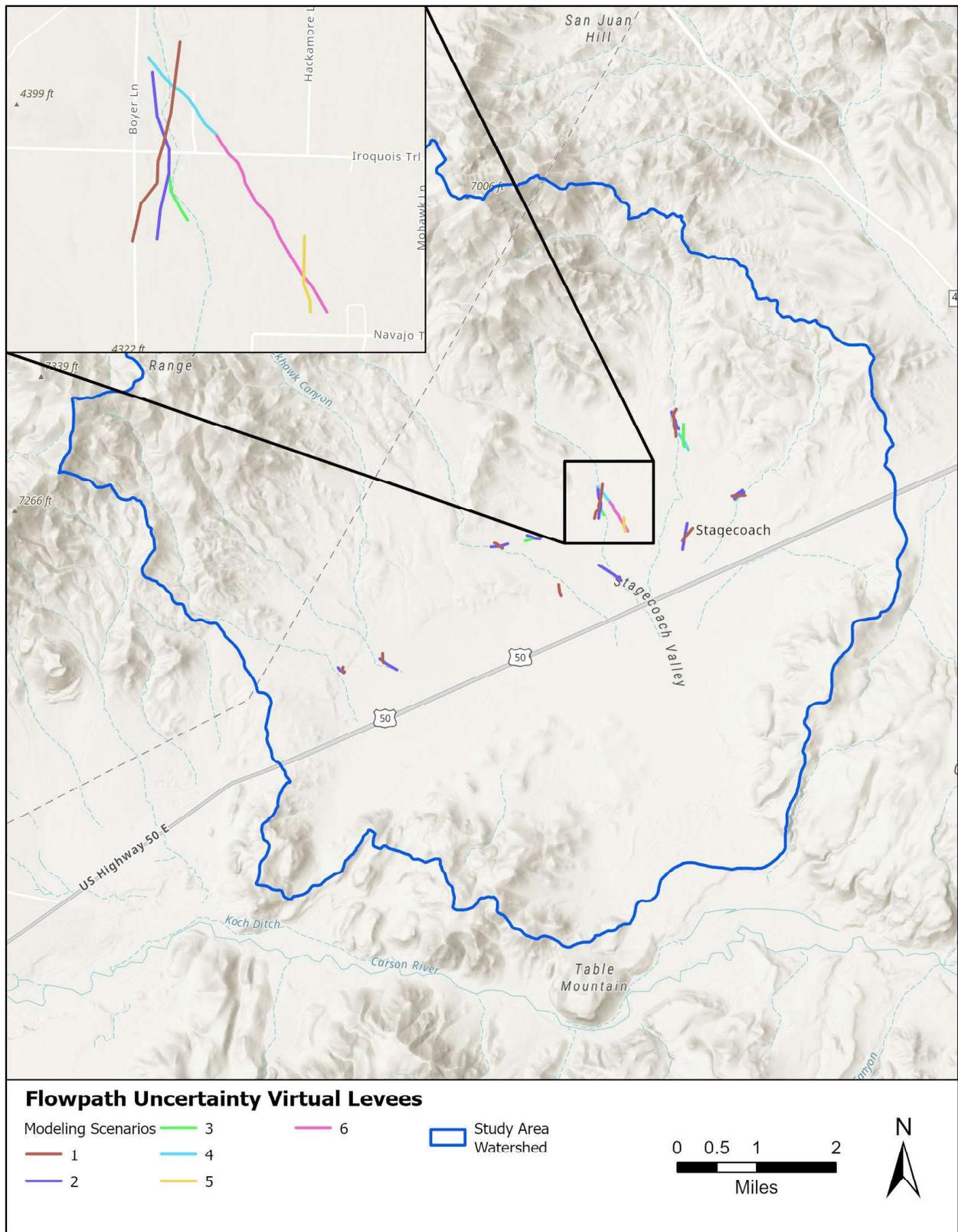


Figure 2-28. Flowpath uncertainty virtual levees

### 2.6.3 Composite Results

Given the immense density/quantity of output data associated with two-dimensional modeling, results are best depicted graphically in figures, exhibits, maps, etc. Therefore, the composite (Maximum) flood hazard condition is depicted graphically for the 25-year, 24-hour; 100-year-24-hour, and 100-year, 6-hour storm events. Figure 2-29 and Figure 2-30 depicts the maximum flow depth flowpath uncertainty scenarios modeled, and Figure 2-31 and Figure 2-32 depicts the maximum discharge flowpath uncertainty scenarios modeled. See Figure 2-9 through Figure 2-16 for the base model results for comparison.

It is important for the reader to distinguish, for the purpose of this study, the difference between flowpath uncertainty flood scenarios and composite flood hazard conditions. Each of the flow depth models (Base, Scenario 1, Scenario 2) is considered a flowpath uncertainty flood scenario. Composite flood hazard conditions (maximum flow depth, velocity, and discharge) were determined by compiling the flowpath uncertainty scenario rasters using ArcGIS software tools to extract the highest value for each pixel (combined maximum values), then convert those values to a single output raster grid. The output raster represents the potential composite flood hazard condition per model grid element.

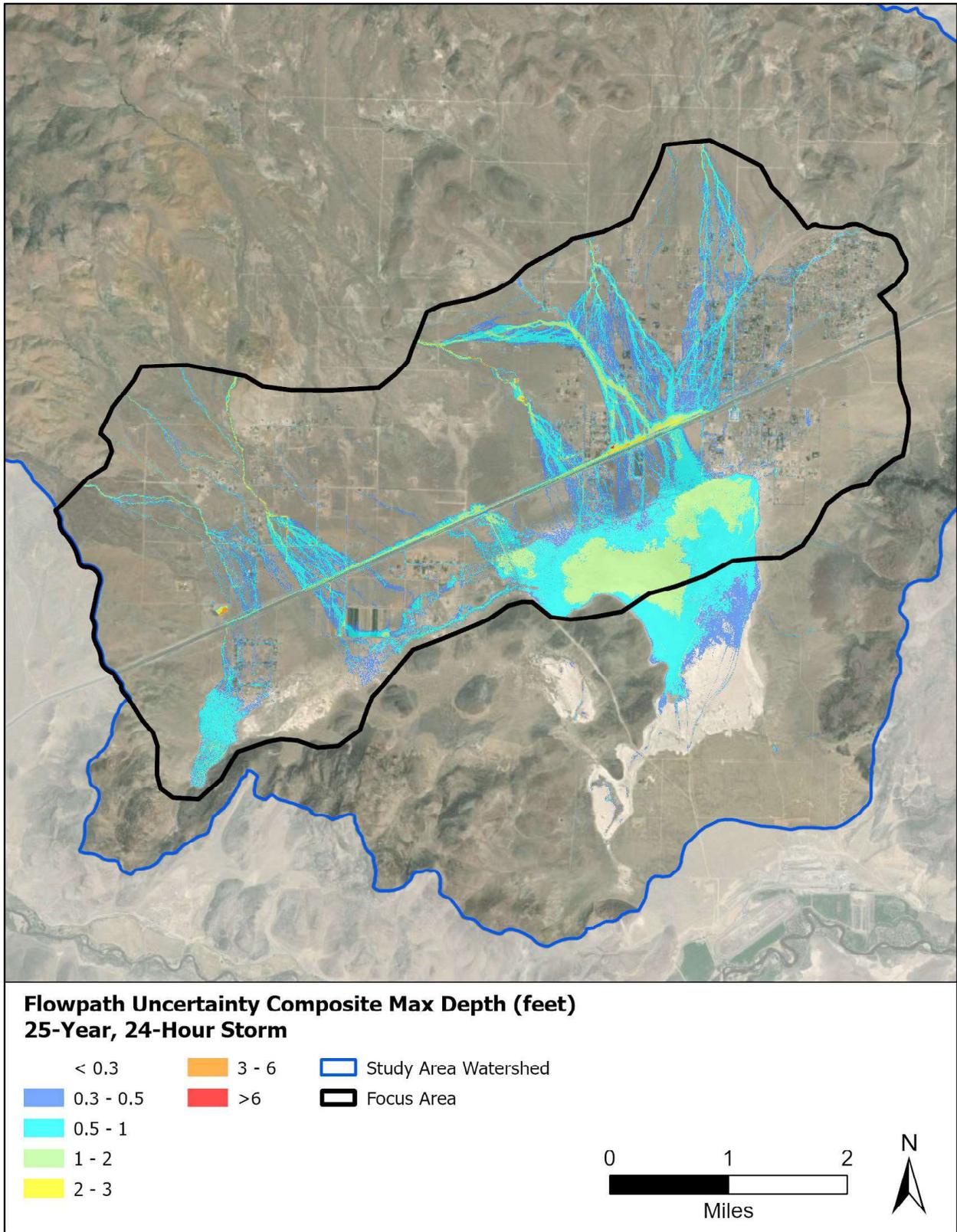


Figure 2-29. Flowpath Uncertainty combined maximum flow depth results (25-Year, 24-Hour)

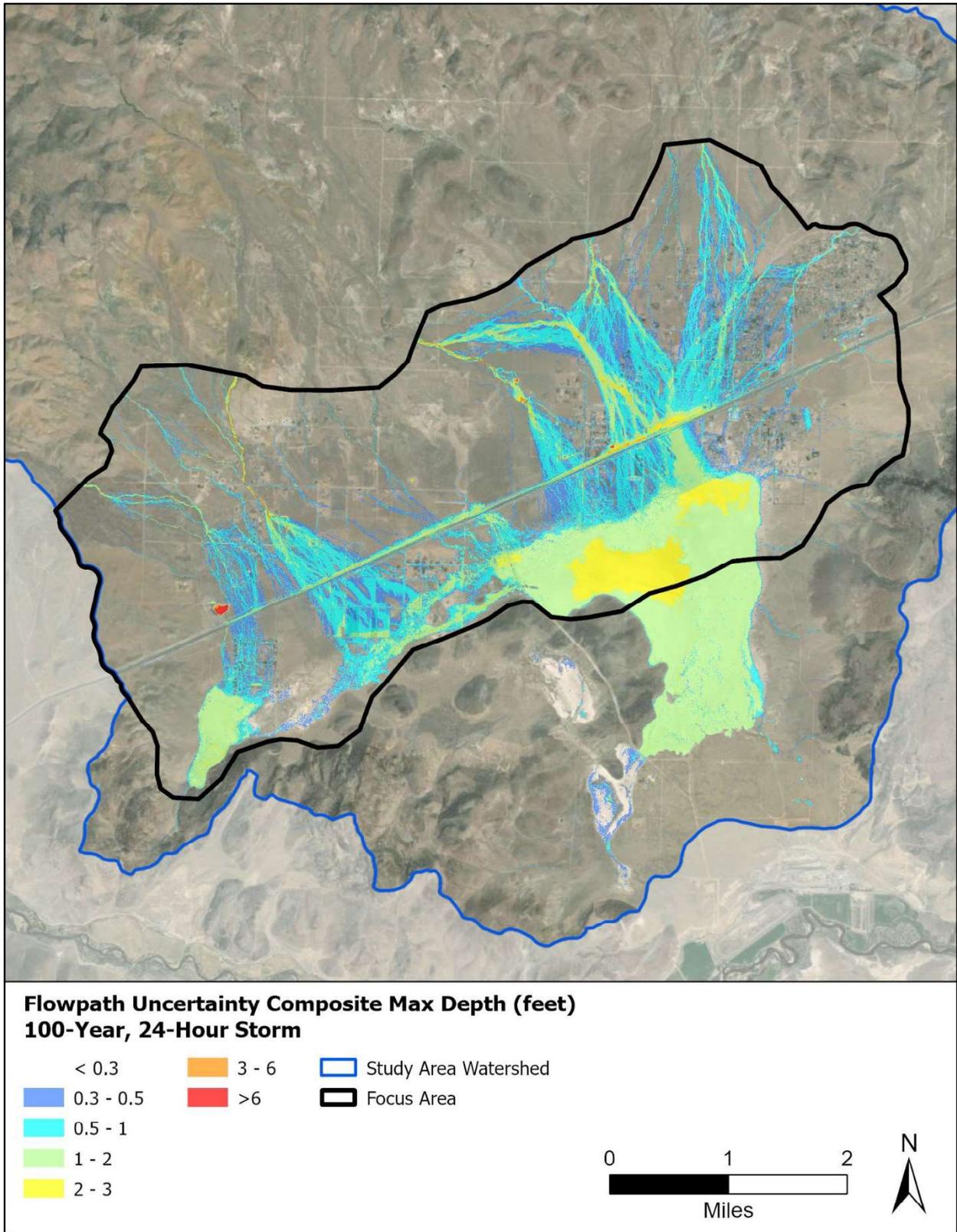


Figure 2-30. Flowpath Uncertainty combined maximum flow depth results (100-Year, 24-Hour)

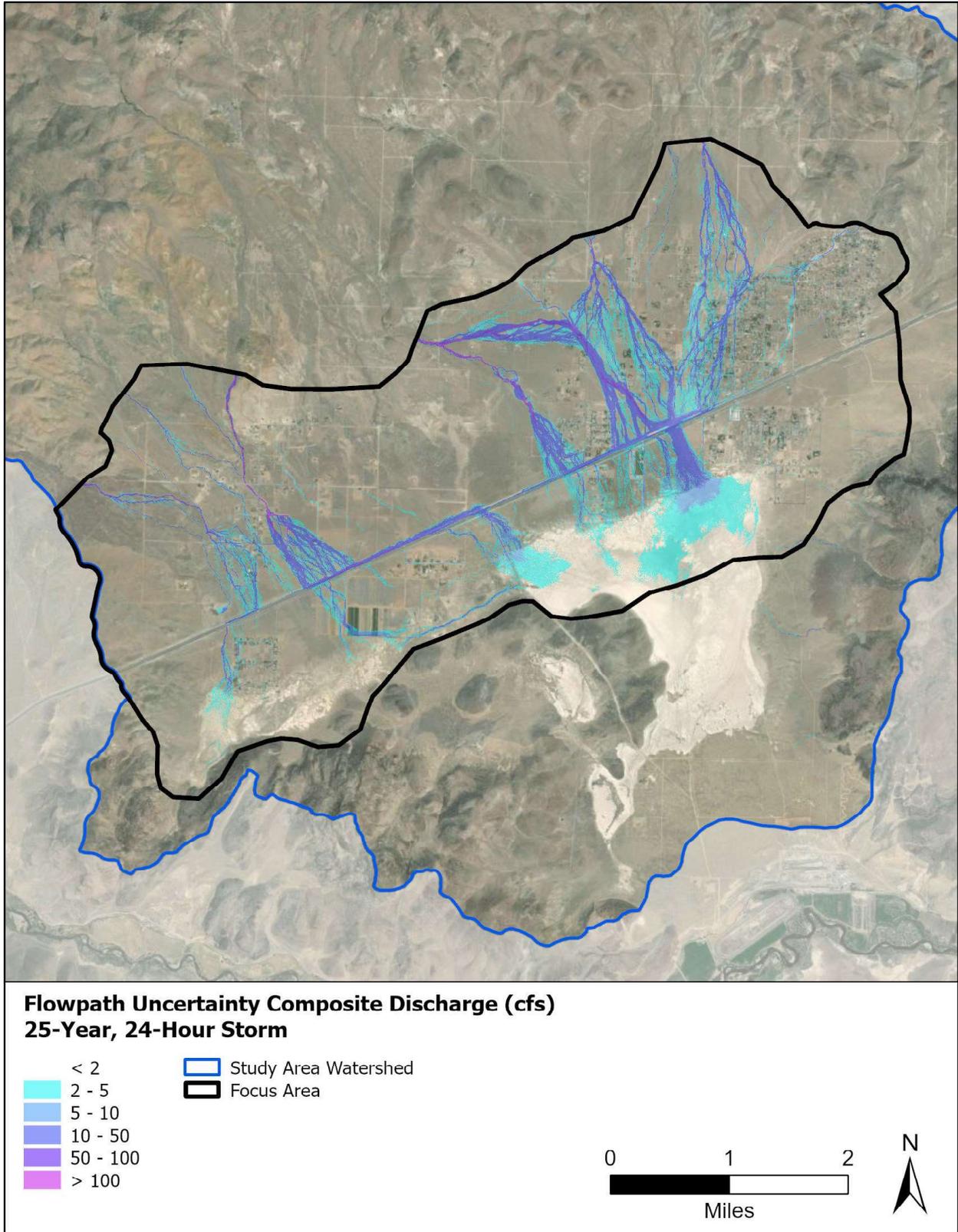


Figure 2-31. Flowpath Uncertainty discharge results (25-Year, 24-Hour)

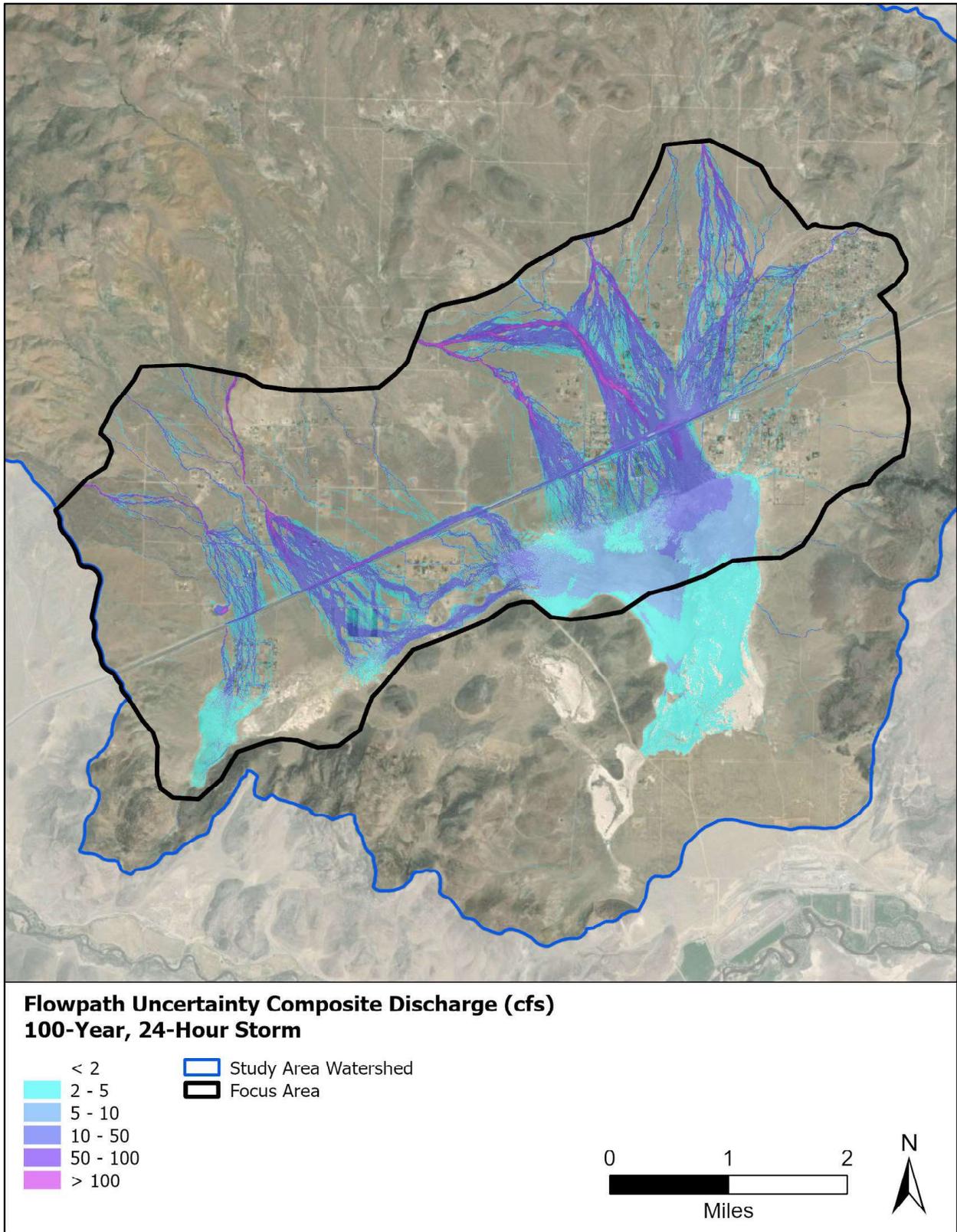


Figure 2-32. Flowpath Uncertainty discharge results (100-Year, 24-Hour)

## 2.6.4 How the Composite Result Data is Used in this Study

The flowpath uncertainty composite results were used in the development of the Flood Risk Quick Reference (see Section 5.6).

### 2.6.4.1 *Recommended Future Use*

It is recommended that all future use of the ADMP hydraulic data consider the flowpath uncertainty scenario results (i.e. the maximum values considering all scenarios are used at the desired location).

## 3 SEDIMENT ENGINEERING

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### 3.1 INTRODUCTION

An estimate of sediment delivery volumes is beneficial when assessing proposed infrastructure locations and designs. Unlike scour, Long-term sediment deposition is a common issue for infrastructure and often cannot be mitigated through on-site improvements. Common examples of sediment deposition challenges for infrastructure included culvert sedimentation and loss of storage volume in detention basins due to deposition. To assist in the design of proposed infrastructure improvements, sediment yield was calculated for the contributing watersheds.

### 3.2 SEDIMENT SAMPLING AND TRANSPORT ANALYSIS

#### 3.2.1 Sediment Sampling

Since sediment is easily transported throughout the study area (note minor sediment deposition in culvert in Figure 3-1), twelve sediment samples were collected in July 2023 by JEF staff to help classify the type of sediment that exists in the watershed. The sampling locations are shown along with the sample identification numbers (IDs) in Figure 3-2. These samples were processed via mechanical sieve procedures to compute the sediment gradation. The gradation curves from each sample are shown in Figure 3-3, while major characteristics of the sediment are tabulated in Table 3-1.



*Figure 3-1. Example of sediment deposition at culvert*

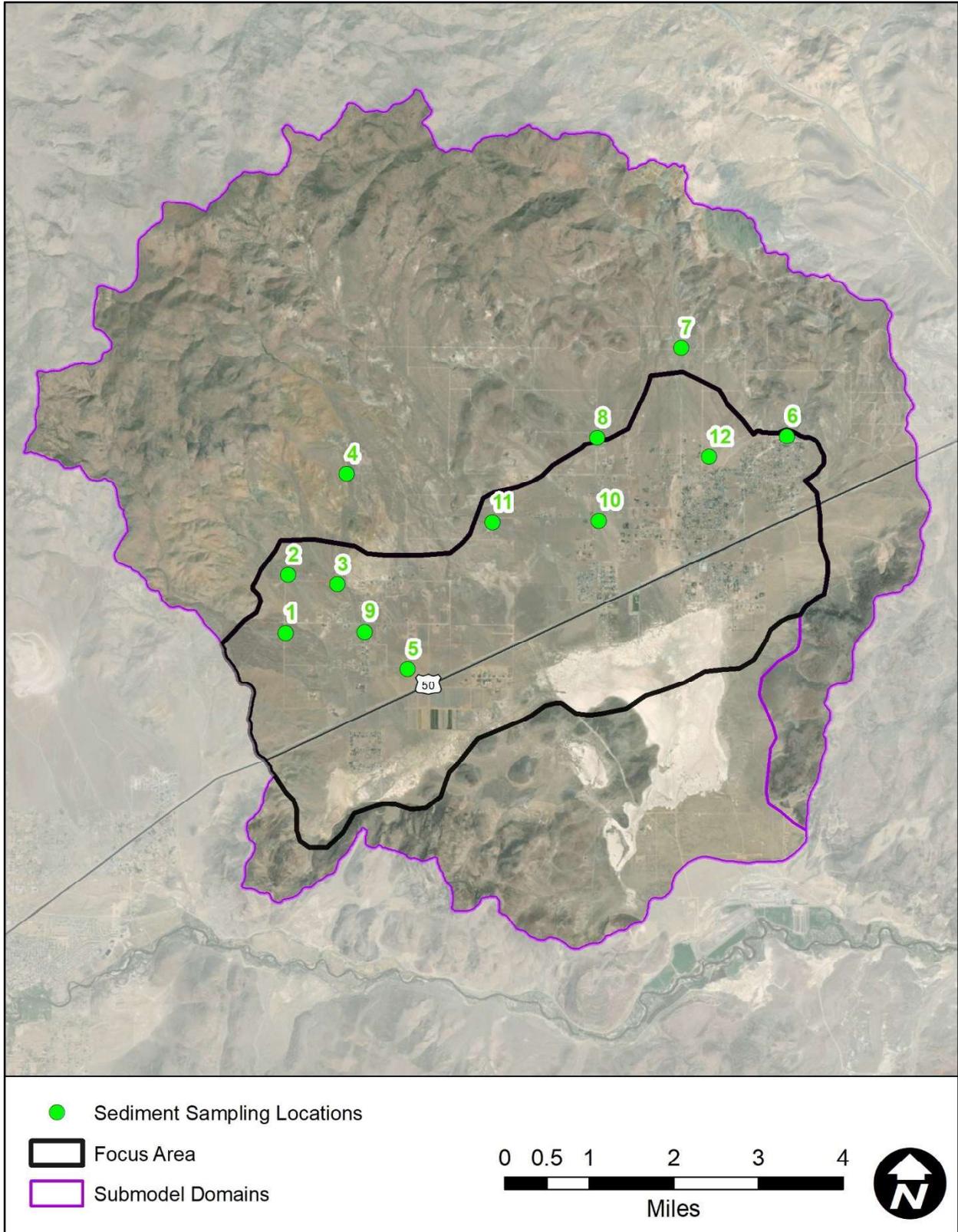


Figure 3-2. Sediment sampling locations labeled with ID

### CUMULATIVE DISTRIBUTION

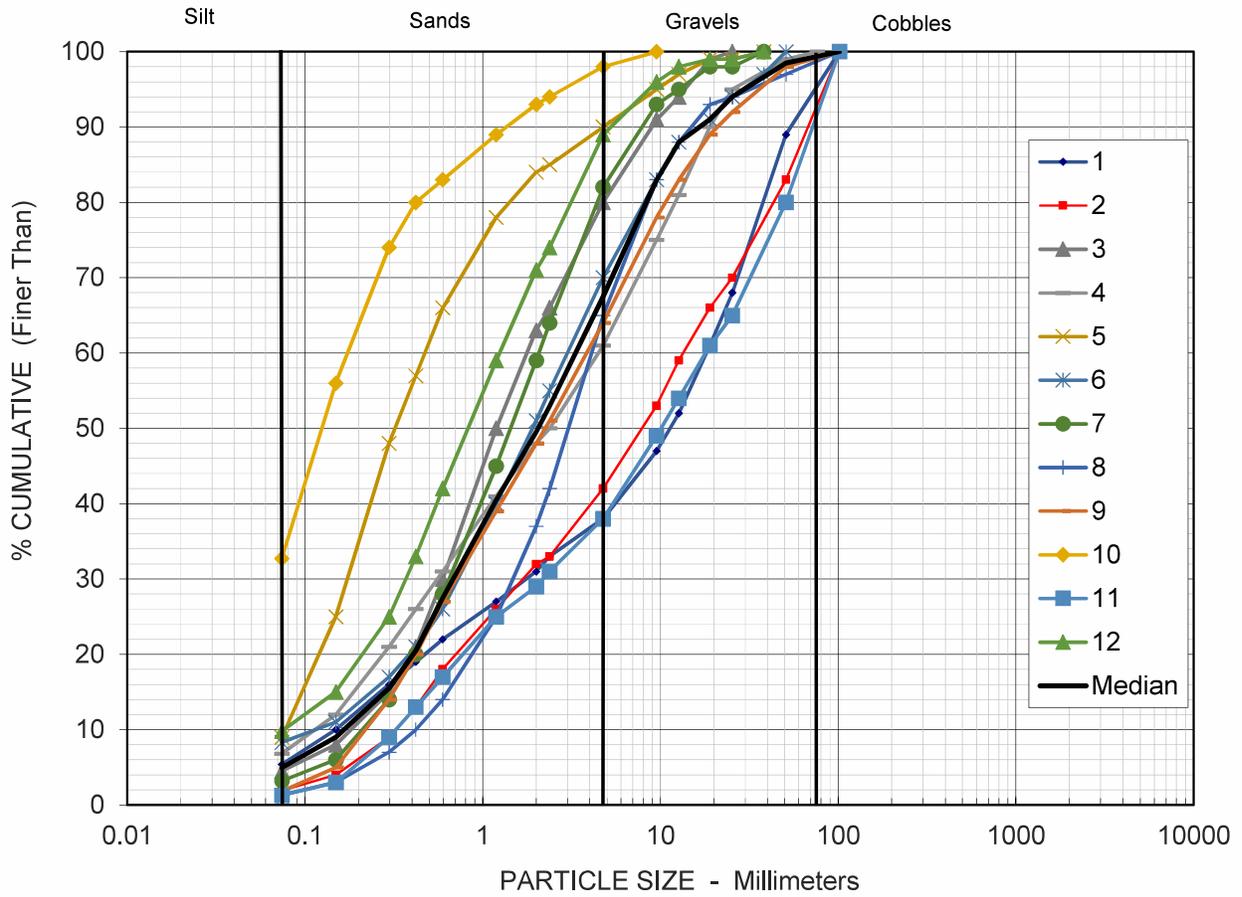


Figure 3-3. Gradation curves for the twelve sediment samples

Table 3-1. Major characteristics of the sediment

ID	Type	D16 (mm)	D50 (mm)	D84 (mm)	G
1	Sieve Analysis	0.30	11.42	44.75	33.91
2	Sieve Analysis	0.53	8.22	53.79	17.76
3	Sieve Analysis	0.32	1.19	6.49	7.36
4	Sieve Analysis	0.22	2.38	14.80	13.83
5	Sieve Analysis	0.11	0.32	2.00	7.36
6	Sieve Analysis	0.27	1.93	10.15	9.88
7	Sieve Analysis	0.34	1.48	5.62	6.54
8	Sieve Analysis	0.70	3.21	10.15	6.18
9	Sieve Analysis	0.34	2.25	13.75	10.22
10	Sieve Analysis	-	0.13	0.69	-
11	Sieve Analysis	0.55	10.15	60.96	19.54
12	Sieve Analysis	0.16	0.88	3.97	7.90
<b>Median</b>	-	0.32	2.09	10.15	9.88
<b>Average</b>	-	0.35	3.63	18.93	12.77

### 3.2.2 Sediment Transport Analyses

The FLO-2D hydraulic modeling was used to assess the trends of both flooding and sedimentation throughout the study area. Hydraulic data from FLO-2D inherently includes both discharge and flow depth at each grid element. This hydraulic data was used to estimate sedimentation using the Yang sediment transport equation (1973, 1984) on a cell-by-cell scale. The median values from Table 3-1 were used in the sediment calculations.

For each modeled storm event, the total accumulated (i.e., throughout the entire storm event) sediment transport capacities were calculated at each cell. These accumulated capacities can identify areas where deposition or scour may be expected. The detailed results will be discussed in Section 3.3

#### 3.2.2.1 Yang Equation

Sediment transport was calculated using the Yang sediment transport methodology. This approach followed the calculation outline found in the HEC-RAS Hydraulic Reference Manual (USACE, 2016). The grain size distribution was discretized into three equal mass components where sediment transport capacity was computed separately for each compartment and the results were combined while weighting the capacity of each compartment by its relative mass contribution. The governing equation for estimating sediment concentration for each grain size using the Yang approach is as follows:

$$\log C_t = 5.435 - 0.286 \log \frac{\omega d_m}{v} - 0.457 \log \frac{u_*}{\omega} + \left( 1.799 - 0.409 \log \frac{\omega d_m}{v} = 0.314 \log \frac{u_*}{\omega} \right) \log \left( \frac{VS}{\omega} - \frac{V_{cr}S}{\omega} \right) \quad (2)$$

$$\log C_t = 6.681 - 0.633 \log \frac{\omega d_m}{v} - 4.816 \log \frac{u_*}{\omega} + \left( 2.874 - 0.305 \log \frac{\omega d_m}{v} = 0.282 \log \frac{u_*}{\omega} \right) \log \left( \frac{VS}{\omega} - \frac{V_{cr}S}{\omega} \right) \quad (3)$$

Where:

- $C_t$  is the total sediment concentration (ppm)
- $\omega$  is the particle fall velocity (ft/s)
- $d_m$  is the median particle diameter (ft)
- $\nu$  is the kinematic viscosity (ft<sup>2</sup>/s)
- $u_*$  is the shear velocity (ft/s)
- $V$  is the average channel velocity (ft/s)
- $S$  is the energy gradient (ft/ft)

Equation (2) is used for sand with a median diameter < 2mm, while Equation (3) is for gravel with a median diameter is  $\geq$  2mm. Within a model spanning 2-dimensions in plan-view, such as FLO-2D, the Yang methodology differentiates itself through application of vectorized parameters – average channel velocity and slope, notably. Using time-varying output from FLO-2D, the direction of maximum velocity at each time step was determined and the terms utilized in the Yang equation were applied in that direction. This method allows the sediment transport capacity analysis to adapt to changes in peak flow direction which is especially valuable in areas of flowpath uncertainty such as coalescing alluvial fans and areas subject to varied flooding sources.

### 3.3 RESULTS

Since the total accumulated transport is calculated at each cell, an overall map of the study area with sediment transport capacities can be produced like the FLO-2D results presented in Section 2. Also, because these sediment results are based on the hydraulic results at each cell, off-site flows are considered because inflow hydrographs have been input at all major watercourses through the upstream FLO-2D modeling.

The total accumulated sediment transport results for each storm event are shown in Figure 3-4 through Figure 3-7. These figures represent the estimated total sediment that is transported through each cell during the entire simulated storm event. Figure 3-8 is slightly different in that it shows the estimated annualized sediment transport calculated based on the statistical probability of each individual storm event. Note - The 25Y24H, 100Y6H, and 100Y24H storm events use the same values in the color ramp with the maximum value taken from the storm event shown in the figure. The Annualized and 5Y24H Figures use different values since the transport is much lower during these events relative to the larger events. However, all these figures use the same color scheme where green is relatively low compared to red, but green is higher than areas without color.

In general, the results are straightforward. Higher sediment transport rates appear in the channels, while lower rates appear as the flow spreads out over the piedmont with the ultimate outfall of the watershed (i.e., the Playa) showing no transport since it is a depositional area. Finally, these results (i.e., the 100Y24H storm event and Annualized) were used to calculate a design sediment volume that was used in the sizing of any basins used in the flood mitigation alternatives (see Section 5).

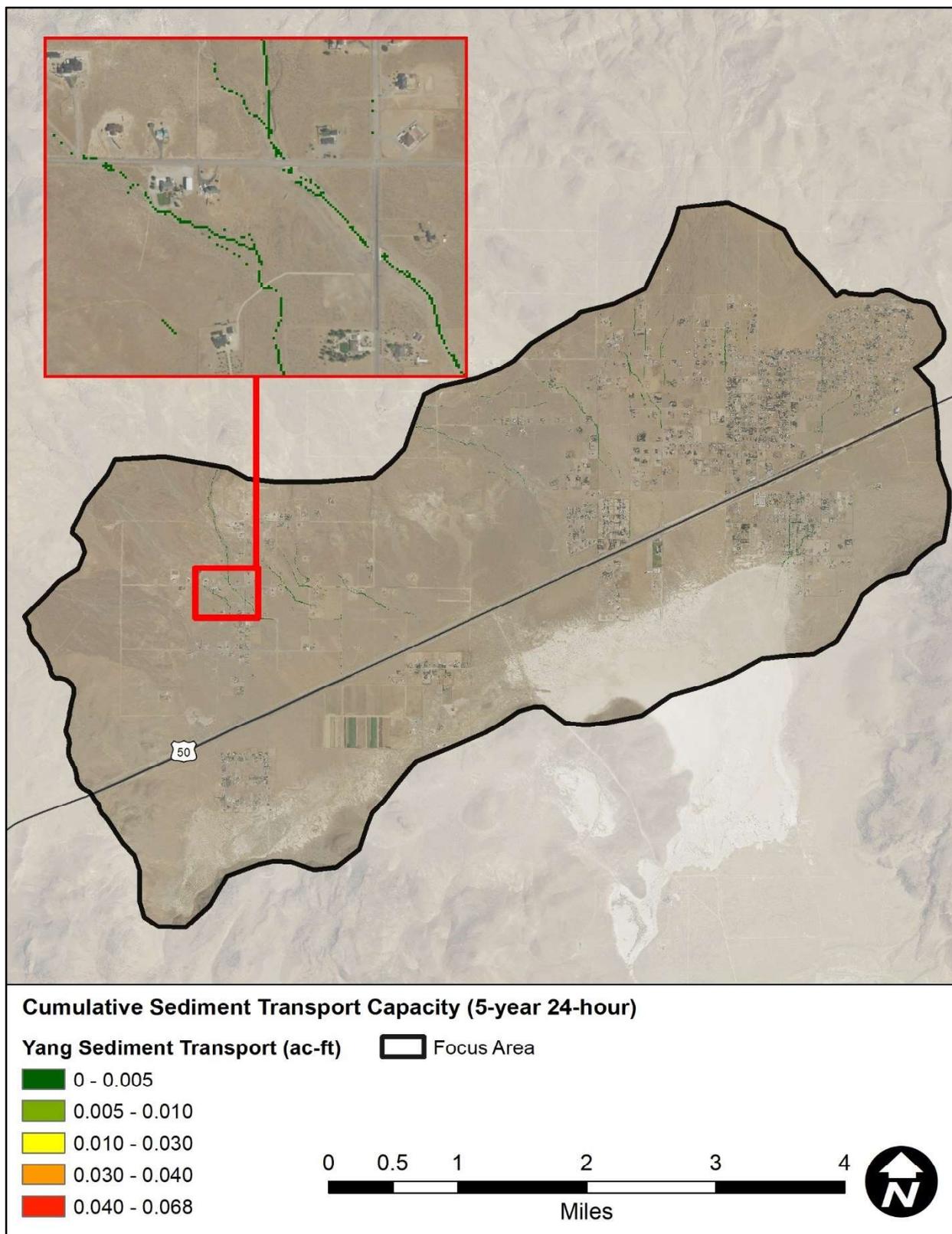


Figure 3-4. Accumulated sediment transport through the Focus Area during the 5-year 24-hour event

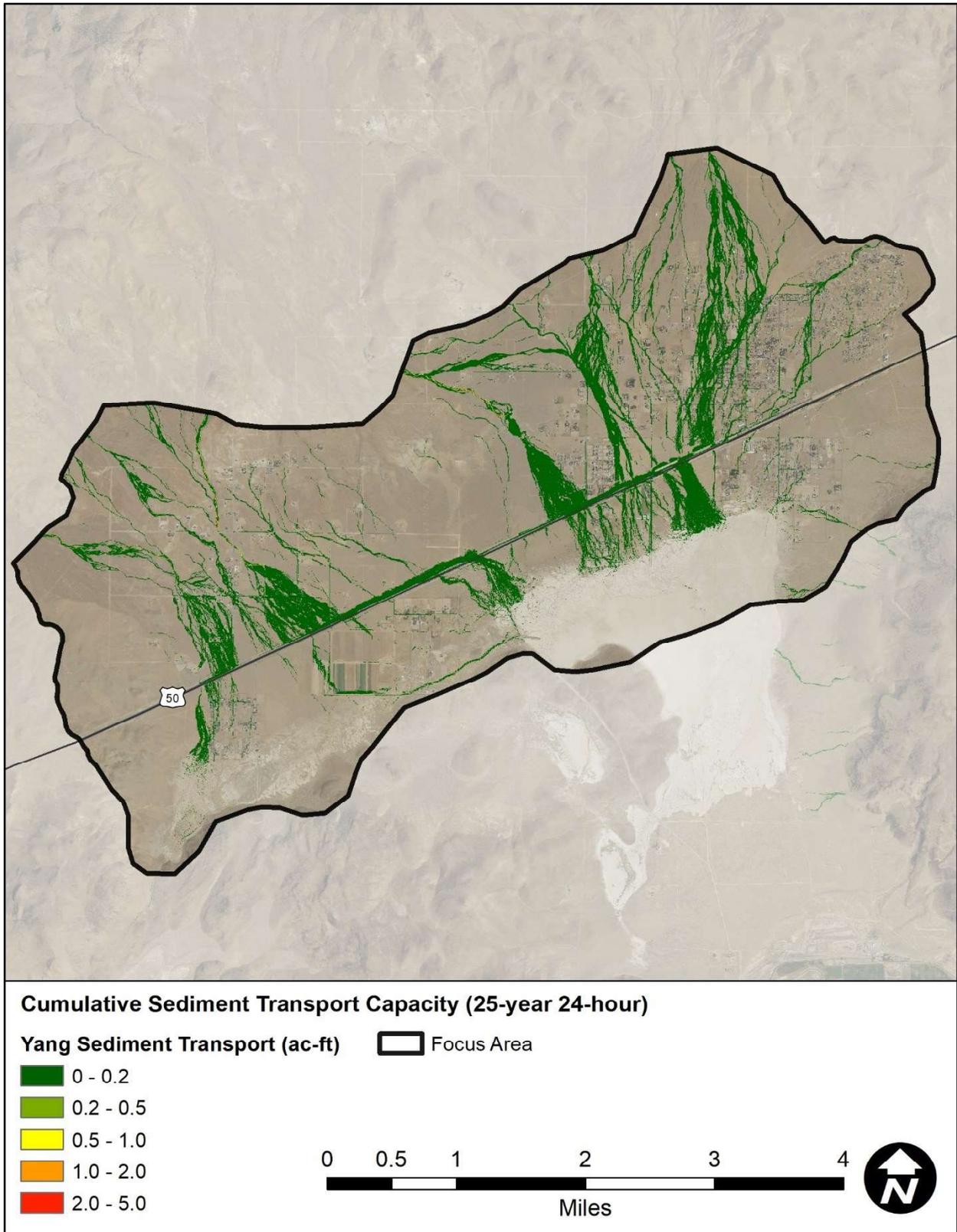


Figure 3-5. Accumulated sediment transport through the Focus Area during the 25-year 24-hour event

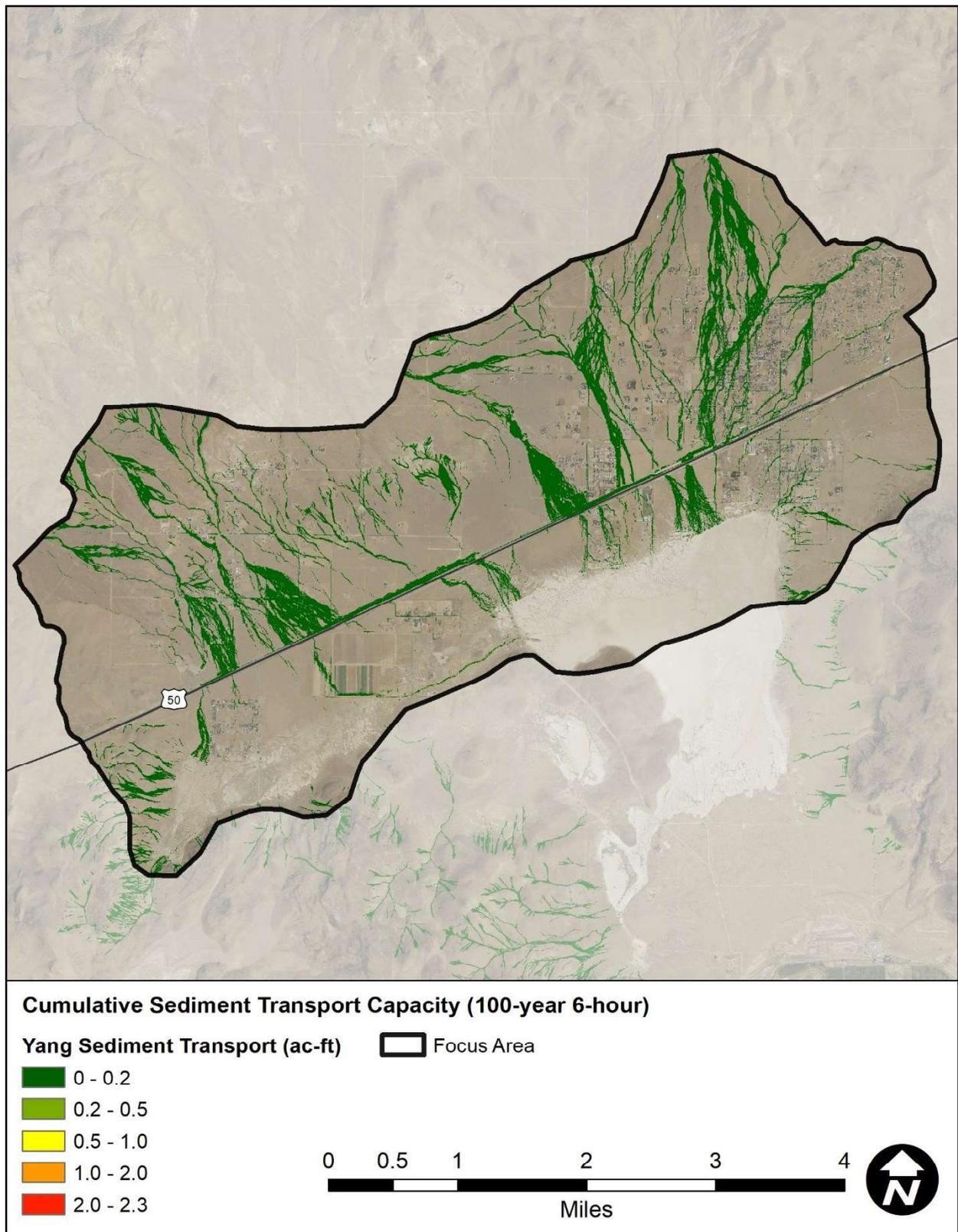


Figure 3-6. Accumulated sediment transport through the Focus Area during the 100-year 6-hour event

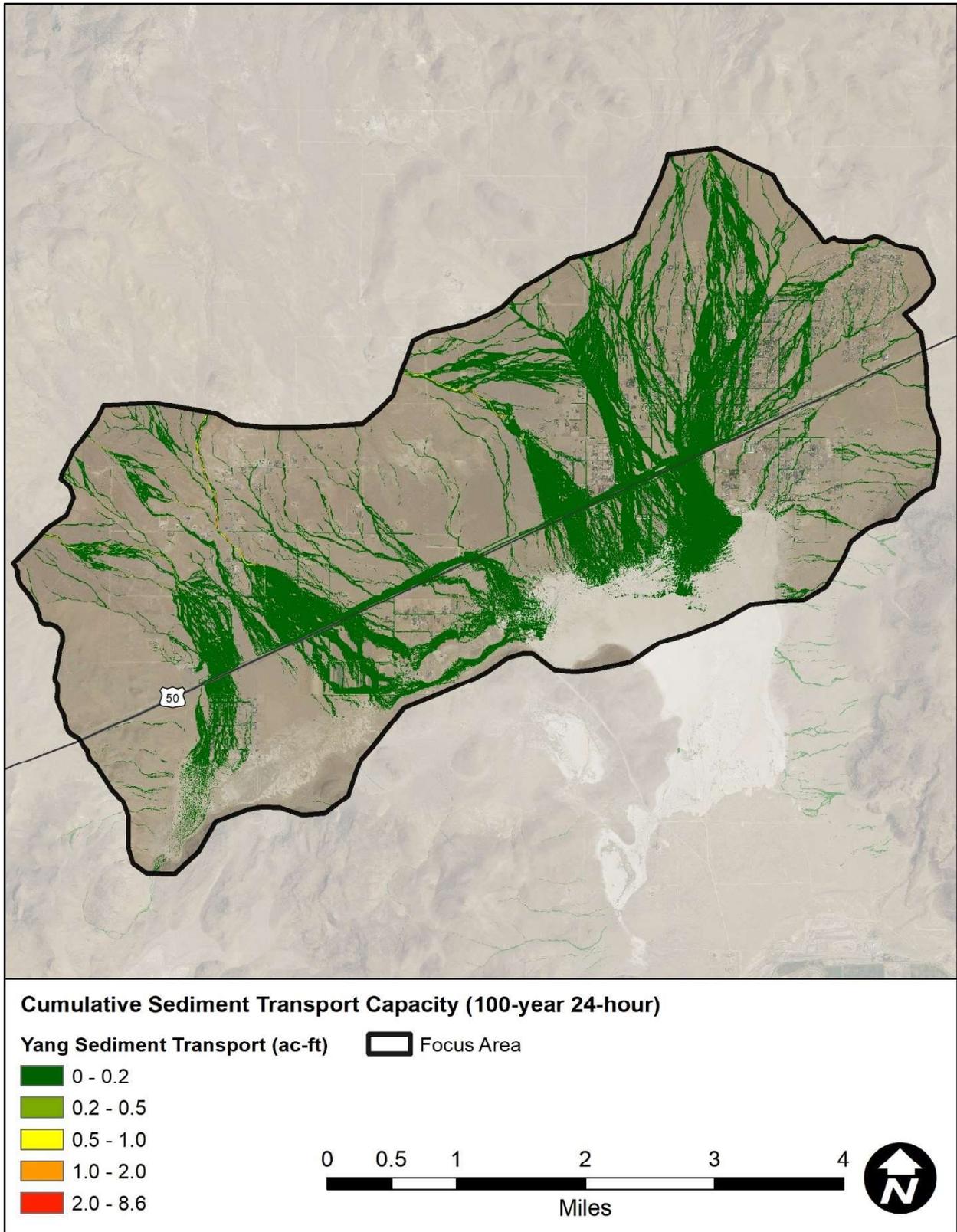


Figure 3-7. Accumulated sediment transport through the Focus Area during the 100-year 24-hour event

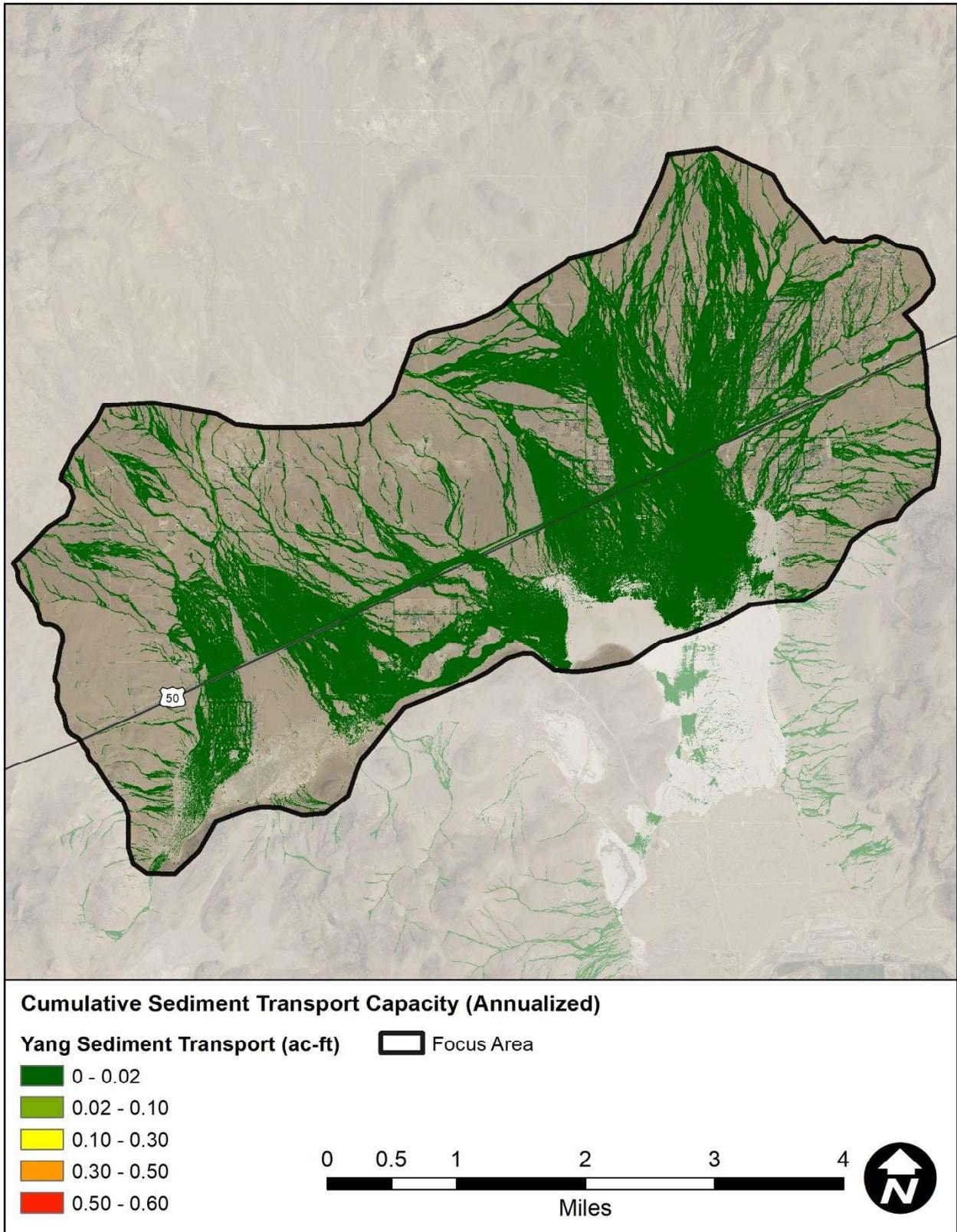


Figure 3-8. Annualized sediment transport through the Focus Area

## 4 FLOOD HAZARD CLASSIFICATION

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### 4.1 PURPOSE

During a severe storm event, flood waters flow throughout the study watershed. However, not all flood hazards pose a risk to people or to their properties. Flood risk depends on the presence of both a flood hazard and a person (or their property). As an example, flow in a constructed flood control channel does not present a risk until someone enters the channel. Identifying areas where flood waters may cause risks that potentially harm people (or property) is an important objective of the ADMP. Identification of potential flood risks in the study area helps the consultant team prioritize which flood problems should be addressed and in what order.

For the purposes of this study, flood hazards were defined based on the physical characteristics of the flood water – that is, the location, depth, and velocity associated with those flood waters. The hydrology and hydraulic modeling results were used to define flood hazards for four storms:

- The 5-year, 24-hour event,
- The 25-year, 24-hour event,
- The 100-year, 24-hour event, and
- The 100-year, 6-hour event.

The flood risk assessment involved selecting criteria and quantifying flood risks throughout the study watershed using the FLO-2D model results. Three types of potential flood risks were assessed – flooding risks to pedestrians, passenger vehicles, and structures.

In addition to the flooding risks, a building inundation assessment (BIA) was conducted. The BIA is a planning level analysis that estimates the number of habitable impacted by flow depths greater than six inches. Since this analysis will be done for both the base (i.e., existing) and the with-alternatives conditions (see Section 5), it gives a quantitative estimate of the effectiveness of the potential mitigation structures. The BIA was performed for all four storm events.

The following sections describe the flood classification criteria, methodology, and description of provided electronic files for each hazard assessment.

### 4.2 FLOODING HAZARDS TO PEDESTRIANS

Pedestrian flood hazards were classified using the depth-velocity relationship outlined in the United States Bureau of Reclamation (USBR) Technical Memorandum 11 (TM 11) (1988). The depth-velocity relationships presented in TM 11 are a good basis for flood hazard classification since the criteria are widely accepted. TM 11 presents two possible classifications for pedestrians: flood danger levels for adults and for children. It was decided to use the flood danger classification for children throughout the entire watershed to simplify the methodology and to be conservative. The depth-velocity flood danger level relationship from TM 11 is shown as Figure 4-1.

The following three categories exist for pedestrian flood hazards:

- **Low:** These are areas with depths and velocities corresponding to the Low Danger Zone as shown in Figure 4-1. Low pedestrian hazards are not displayed on the map exhibits because, per TM11, low hazard zones do not present a threat to children of almost any size (excluding infants) and cover all areas not classified with a higher flood hazard.
- **Moderate:** Areas with depths and velocities corresponding to the Judgment Zone in Figure 4-1 have been labeled as having a moderate potential flood hazard to pedestrians.
- **High:** Areas with depths and velocities corresponding to the High Danger Zone in Figure 4-1 have been labeled as having a high potential flood hazard to pedestrians.

The flood hazards to pedestrians have been digitized in GIS in the form of a raster. The rasters generated for the risk analysis coincide with the FLO-2D grid elements with a 10-foot by 10-foot pixel size. The raster contains values of 1, 2, and 3 which correlate to a low, moderate, and high hazard classification, respectively. Since the 100-year, 24-hour storm produces the largest peak runoff for most areas, the flooding hazard from this storm event is shown as Figure 4-2.

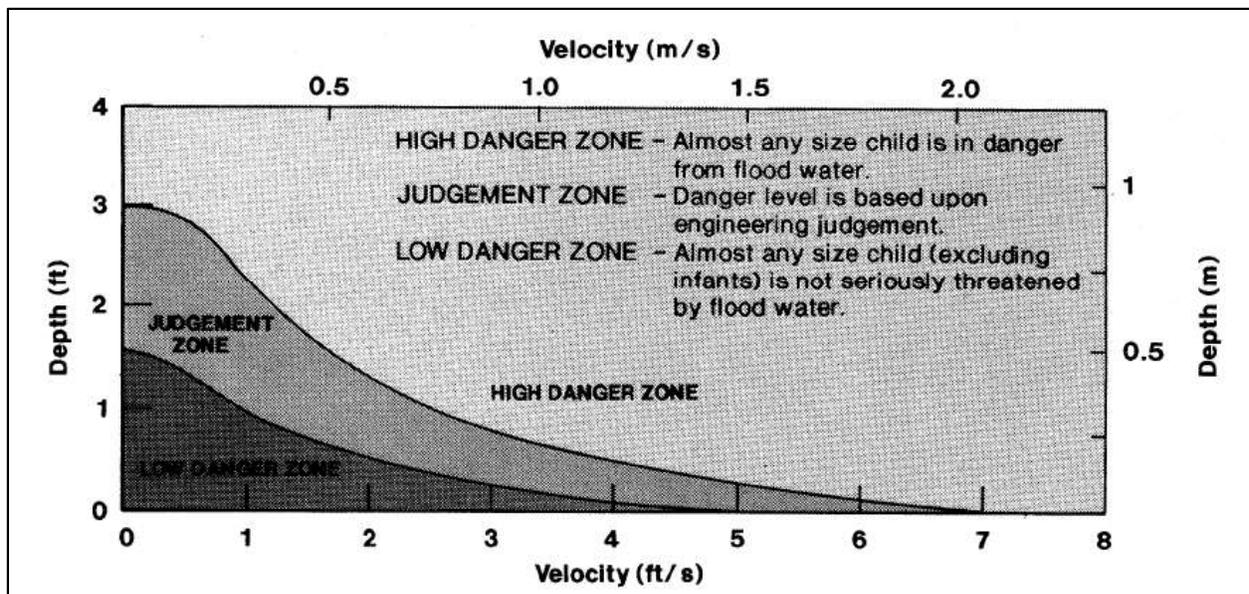


Figure 4-1. Depth-Velocity flood danger level relationship for children, from USBR (1988)

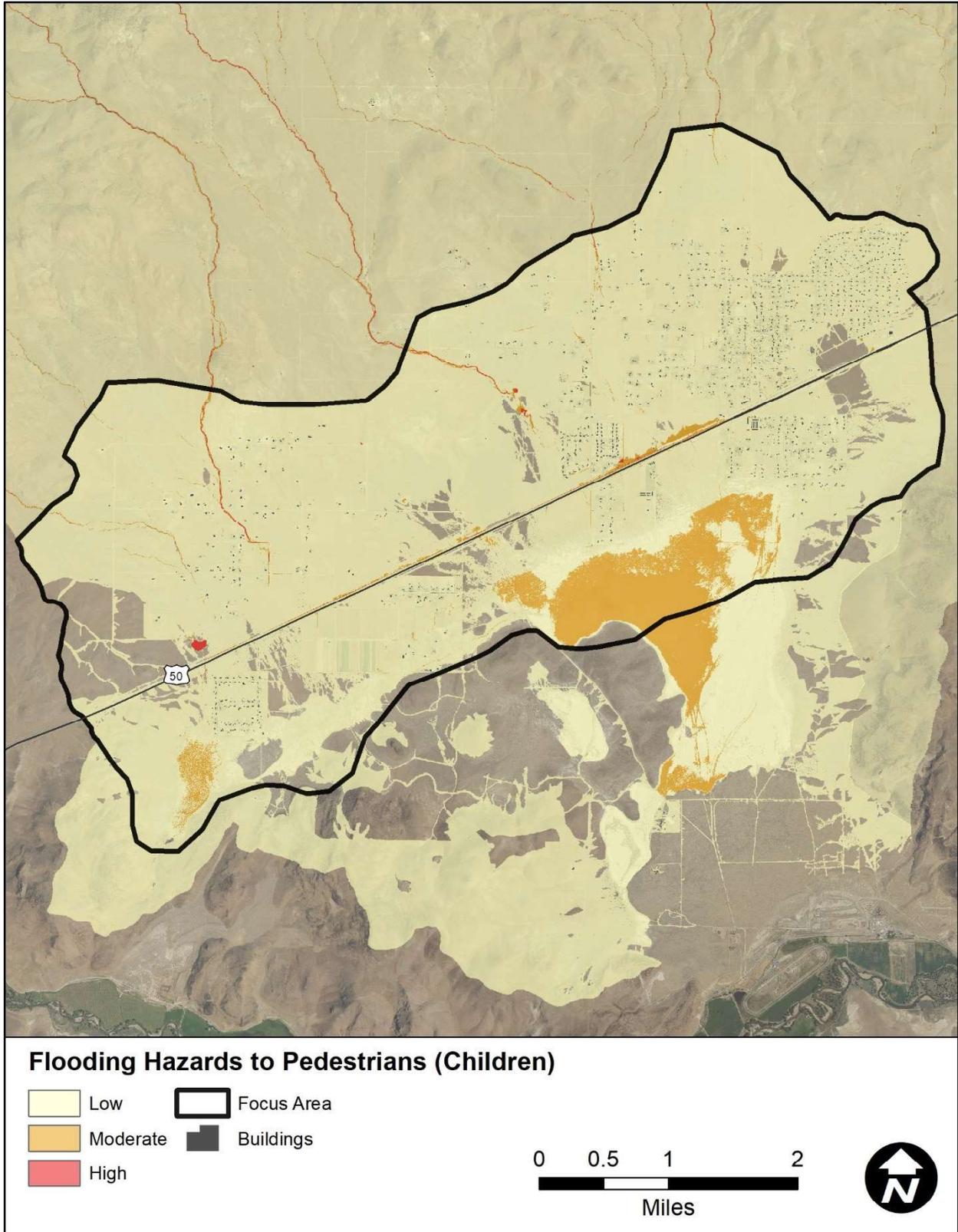


Figure 4-2. USBR criteria flooding hazards to pedestrians based on the 100-year, 24-hour results

### 4.3 FLOODING HAZARDS TO PASSENGER VEHICLES

Potential hazards to passenger vehicles were classified using a combination of minimum depth criteria and the depth-velocity relationship in TM 11 as shown in Figure 4-3. The following four categories exist for passenger vehicle flood hazards:

- **Low:** This hazard category is based solely on minimum depth criteria and is for roadway crossings with depths less than half a foot. Low passenger vehicle hazards are not displayed on the map exhibits because low hazard zones indicate areas where vehicles “are not seriously in danger” and, as such, almost any size passenger vehicle can safely pass. Also, this hazard classification covers all areas not classified with a higher flood hazard. This classification is not explicitly shown in the Figure 4-3.
- **Moderate:** This hazard category is based on a combination of minimum depth criteria and the depth-velocity relationship in TM 11. Specifically, these are roadway crossings with depths and velocities falling into the Low Danger Zone (as shown in Figure 4-3) that also have greater than a half a foot of depth. The threshold depth of half a foot was chosen because half a foot of water will reach the bottom of most passenger cars and can cause loss of control and possible stalling.
- **High:** Roadway crossings with depths and velocities corresponding to the Judgment Zone in have been labeled as having a high potential flood hazard for passenger vehicles.
- **Very High:** Roadway crossings with depths and velocities corresponding to the High Danger Zone in Figure 4-3 have been labeled as having a very high potential flood hazard for passenger vehicles.

The flood hazards to passenger vehicles have also been digitized in GIS in the form of a raster. The raster contains values of 1, 2, 3, and 4. These values correlate to low, moderate, high, and very high classification, respectively. The TM 11 flooding hazards to vehicles for the 100-year, 24-hour storm is shown in Figure 4-4.

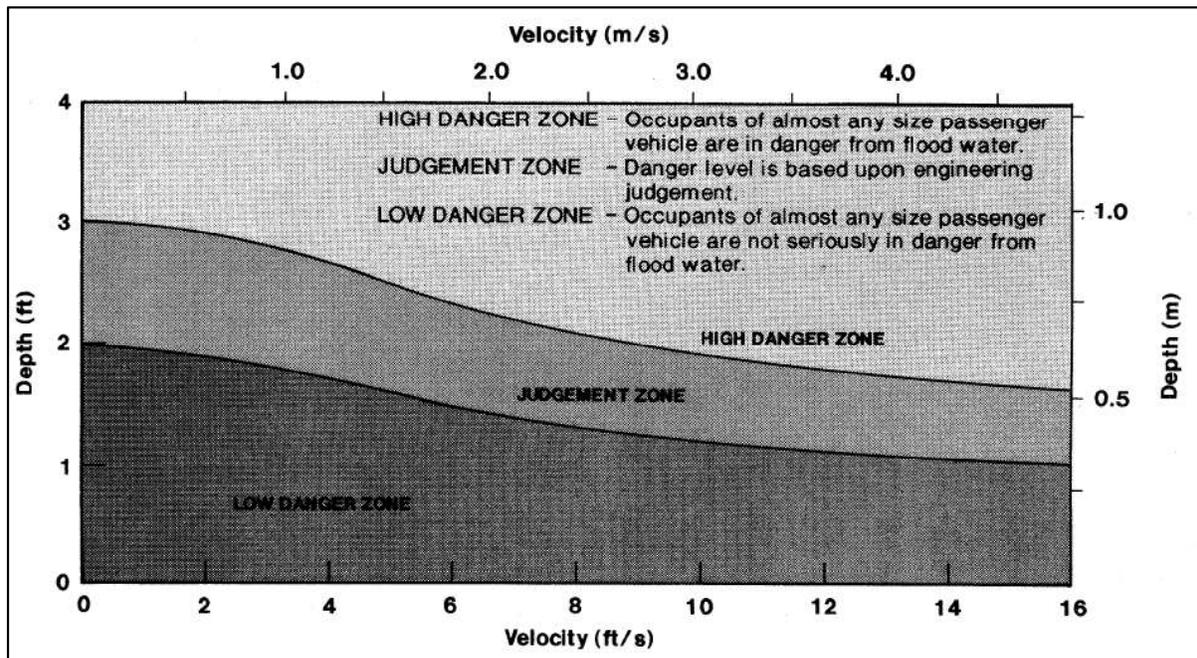


Figure 4-3. Depth-Velocity flood danger level relationship for passenger vehicles, from USBR (1988)

To isolate the actual risk to vehicles, the street centerlines GIS layer was intersected with the hazards zones to produce a “Potential Risk to Passenger Vehicles” map. This isolates the road crossings that pose a risk to vehicles during a storm event. The 100-year 24-hour produced conditions with both “High” and “Very High” risk, and the 25-year 24-hour storm produced conditions of “High” risk using the USBR criteria. The road crossing risk locations for the 100-year, 24-hour and 24-year, 24-hour storms are shown in Figure 4-5 and Figure 4-6, respectively.

For the 100-year storm there are 21 locations of “High” risk crossings, and one location of “Very High” risk, and for the 25-year storm there are eight locations of “High” risk crossings. The crossings are listed below. Note – some of the listing below contain multiple risk locations.

**100-Year, 24 Hour Road Crossing Risk Locations**

<u>Street/Intersection</u>	<u>Risk Level</u>
• Echo Ave and Iroquois Trl	High
• Broken Arrow Rd east of Iron Mountain Blvd	High
• Break A Heart Rd south of Highway 50 Frontage Rd	High
• Stallion Springs Cir south of Geurts Ln	High
• Highway 50 westbound between Boyer Ln and Roys Rd	High
• Highway 50 westbound between Crown Ln and Boyer Ln	High
• Highway 50 westbound between Black Hawk Rd and Crown Ln	High
• Highway 50 north Frontage Road between Boyer Ln and Roys Rd	High
• Highway 50 South Frontage Rd between Break A Heard Rd and Boyer Ln	High
• Highway 50 westbound between Black Hawk Rd and Crown Ln	Very High

**25-Year, 24 Hour Road Crossing Risk Locations**

<u>Street/Intersection</u>	<u>Risk Level</u>
• Broken Arrow Rd east of Iron Mountain Blvd	High
• Stallion Springs Cir south of Geurts Ln	High
• Highway 50 westbound between Boyer Ln and Roys Rd	High
• Highway 50 westbound between Crown Ln and Boyer Ln	High
• Highway 50 westbound between Black Hawk Rd and Crown Ln	High
• Highway 50 north Frontage Road between Boyer Ln and Roys Rd	High



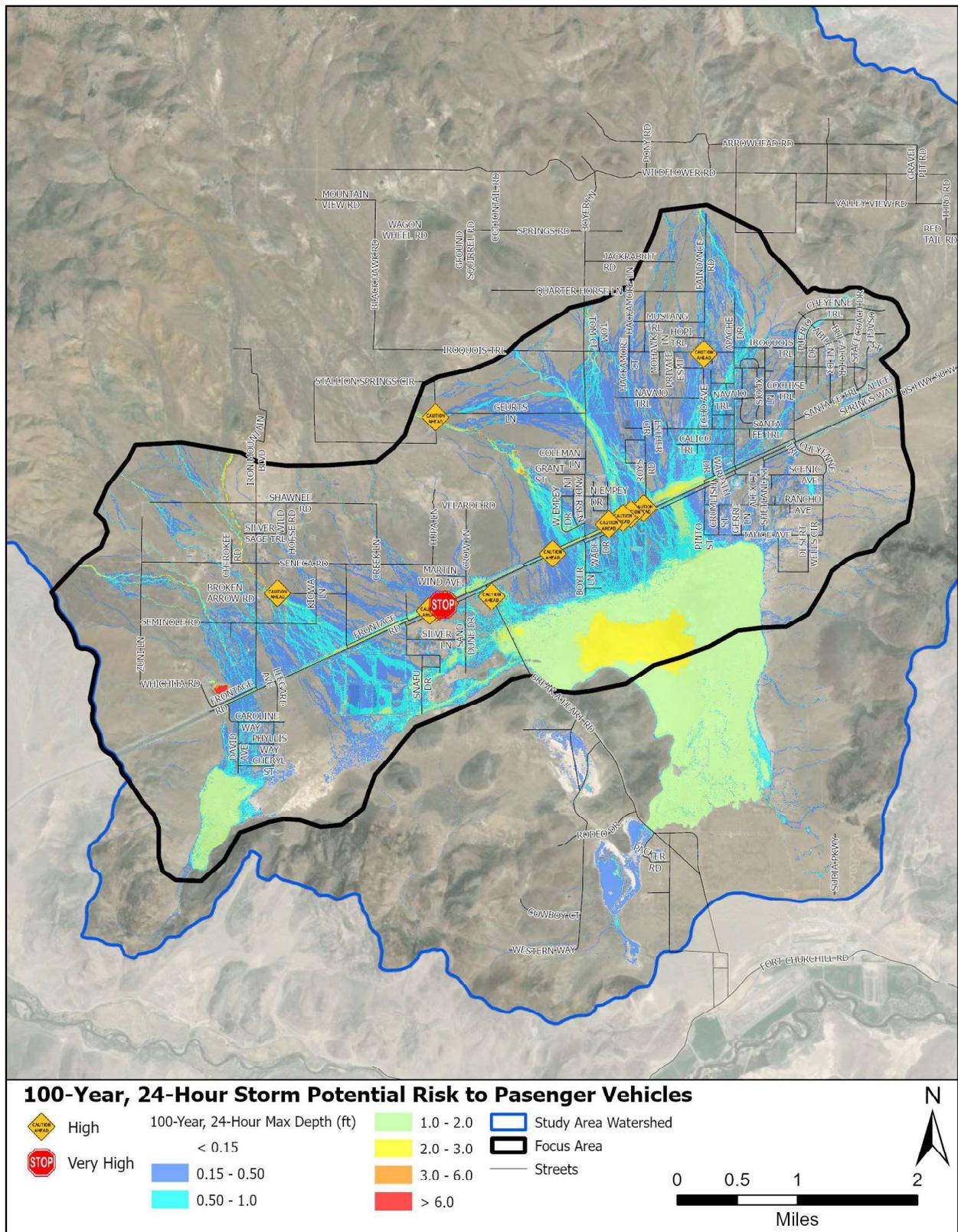
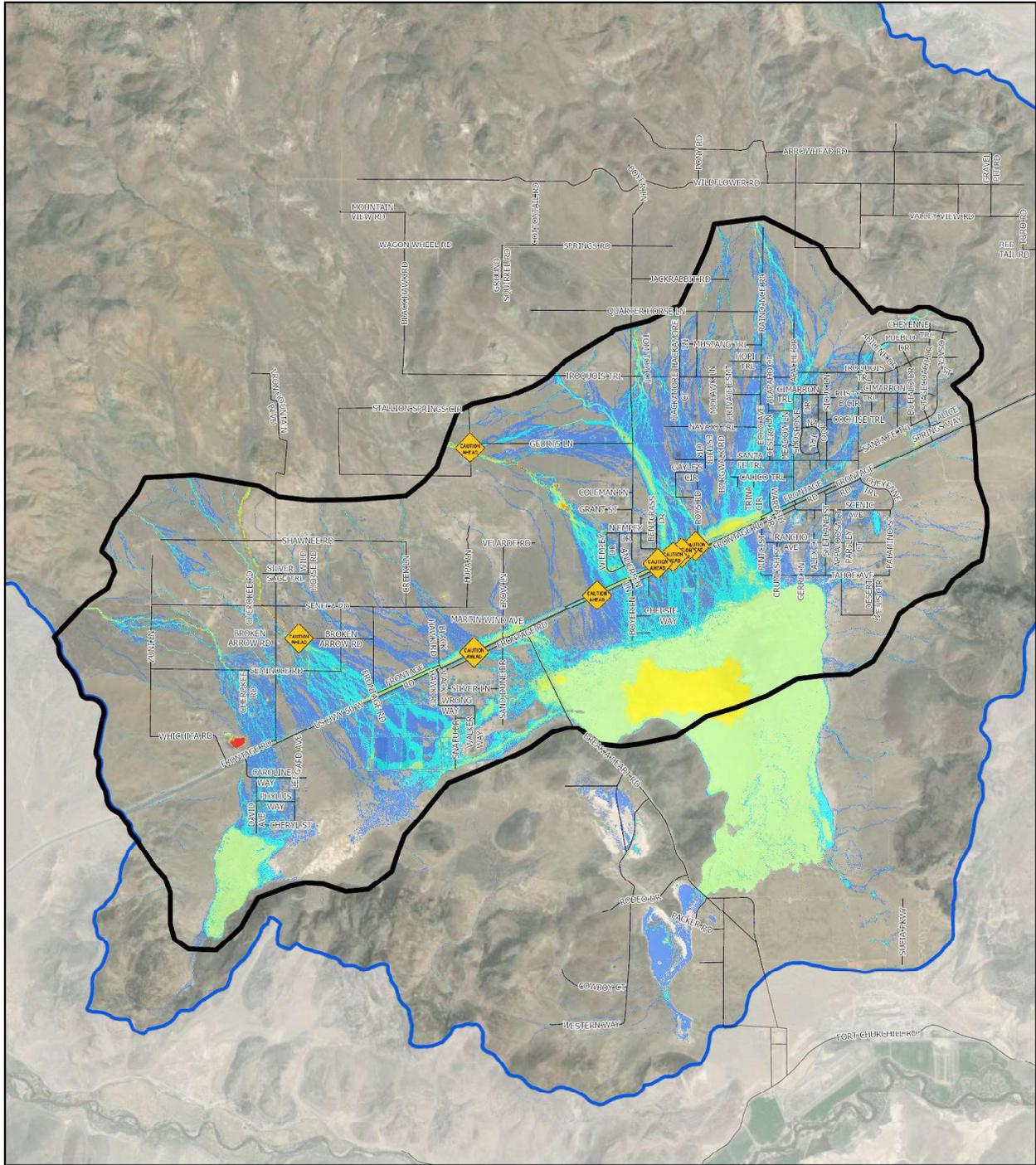


Figure 4-5. Hazardous road crossings during a 100-year, 24-hour storm (USBR criteria)



**25-Year, 24-Hour Storm Potential Risk to Passenger Vehicles**

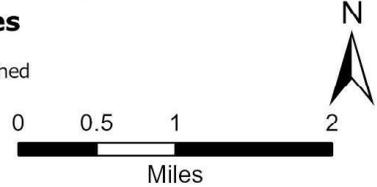


Figure 4-6. Hazardous road crossings during a 25-year, 24-hour storm (USBR criteria)

## 4.4 FLOODING HAZARDS TO BUILDINGS

### 4.4.1 USBR Methodology

Potential hazards to buildings were classified using the depth-velocity relationship from TM 11. The depth-velocity relationship from TM 11 is shown as Figure 4-7. The following three categories exist for potential flood hazards to structures:

- *Low*: Buildings that have contact with at least one FLO-2D grid element that has a depth-velocity relationship corresponding to the low danger zone in Figure 4-7 have been designated as having a low potential flood hazard.
- *Moderate*: Buildings that have contact with at least one FLO-2D grid element that has a depth-velocity relationship corresponding to the judgment danger zone in Figure 4-7 have been designated as having a moderate potential flood hazard.
- *High*: Buildings that have contact with at least one FLO-2D grid element that has a depth-velocity relationship corresponding to the high danger zone in Figure 4-7 have been designated as having a high potential flood hazard.

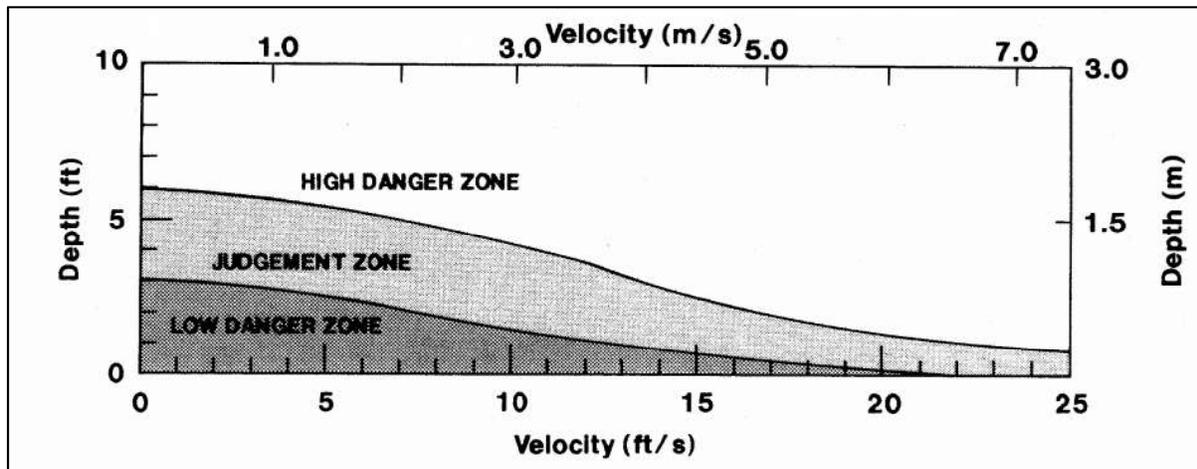


Figure 4-7. Depth-Velocity flood danger level relationship for structures built on foundations, from USBR (1988)

From the building footprint data there are 2,104 structures within the ADMP focus area (including a slight buffer); however, not all these structures are habitable. Buildings with less than 600 square feet (e.g., unattached garages or sheds) were not considered because they were assumed to be uninhabited due to their size. After applying this filter, there are 1,773 structures in the study area.

To create the building flood hazard classification, the building polygon shapefile is intersected with the flood hazard layer using GIS software tools. When multiple grid cells from the flood hazard layer intersect one building polygon, the maximum hazard classification is assigned to the building. The result is a building polygon shapefile with a hazard attribute classifying low, moderate, or high flood hazards.

The tabulated building hazard results are shown in Table 4-1. Due to the relatively shallow flooding in the project area, there are zero buildings with a moderate or high hazard based on the TM 11 criteria. The 100-year 24-hour flooding hazards to buildings raster for the ADMP focus area is shown in Figure 4-8.

Table 4-1. Buildings flooding hazard classification results (base conditions)

<b>Base Conditions</b>				
<b>Recurrence Interval</b>	<b>Building Count</b>	<b>Building Count</b>	<b>Building Count</b>	<b>Total Building Count</b>
	<b>Low</b>	<b>Moderate</b>	<b>High</b>	
<b>25Y24H</b>	2,070	0	0	2,070
<b>100Y24H</b>	2,070	0	0	2,070
<b>100Y6H</b>	2,070	0	0	2,070



#### 4.4.2 Alternate Building Impact Methodology

To verify and contrast the flooding hazards to building results, a separate building impact analysis was conducted using the building footprint data and the maximum depth results from the FLO-2D modeling for the base conditions. The maximum depth layers only consider the maximum depth that occurred during the simulation. For this analysis, the same 600 square foot filter that was discussed in Section 4.4.1 was applied.

In this section, the documentation will focus on the base conditions analyses, while the with-alternatives results will be presented later in Section 4.4.

#### 4.4.3 Base Conditions

Each building was classified based on the maximum flow depth that accumulated adjacent to the structure outline. The structures were tabulated into four groups:

- 1)  $0.25 \text{ ft} < \text{Depth (h)} \leq 0.5 \text{ ft}$  – Low
- 2)  $0.5 \text{ ft} \leq \text{Depth (h)} \leq 1.0 \text{ ft}$  – Moderate
- 3)  $1.0 \text{ ft} < \text{Depth (h)}$  – High
- 4)  $0.25 \text{ ft} < \text{Depth (h)}$  (inclusive of groups 1 through 3 above)

The results for existing conditions are tabulated in Table 4-2. From these results, the shorter duration 100-year 6-hour storm has the potential to impact a large number of structures, but the 100-year 24-hour storm still impacts the most structures. Due to the amount of rainfall volume, these results are expected. The flooding impact to individual buildings are shown in Figure 4-10 for the 100-year, 24-hour storm.

Table 4-2. Buildings that are impacted by various depths (base conditions)

<b>Base Conditions</b>				
<b>Recurrence Interval</b>	<b>Building Count Flow Depth</b>	<b>Building Count Flow Depth</b>	<b>Building Count Flow Depth</b>	<b>Total Building Count</b>
	<b>0.25' &lt; h ≤ 0.5'</b>	<b>0.5' &lt; h ≤ 1'</b>	<b>1' &lt; h</b>	<b>0.25' &lt; h</b>
<b>25Y24H</b>	381	208	15	604
<b>100Y24H</b>	474	443	71	988
<b>100Y6H</b>	424	214	10	648

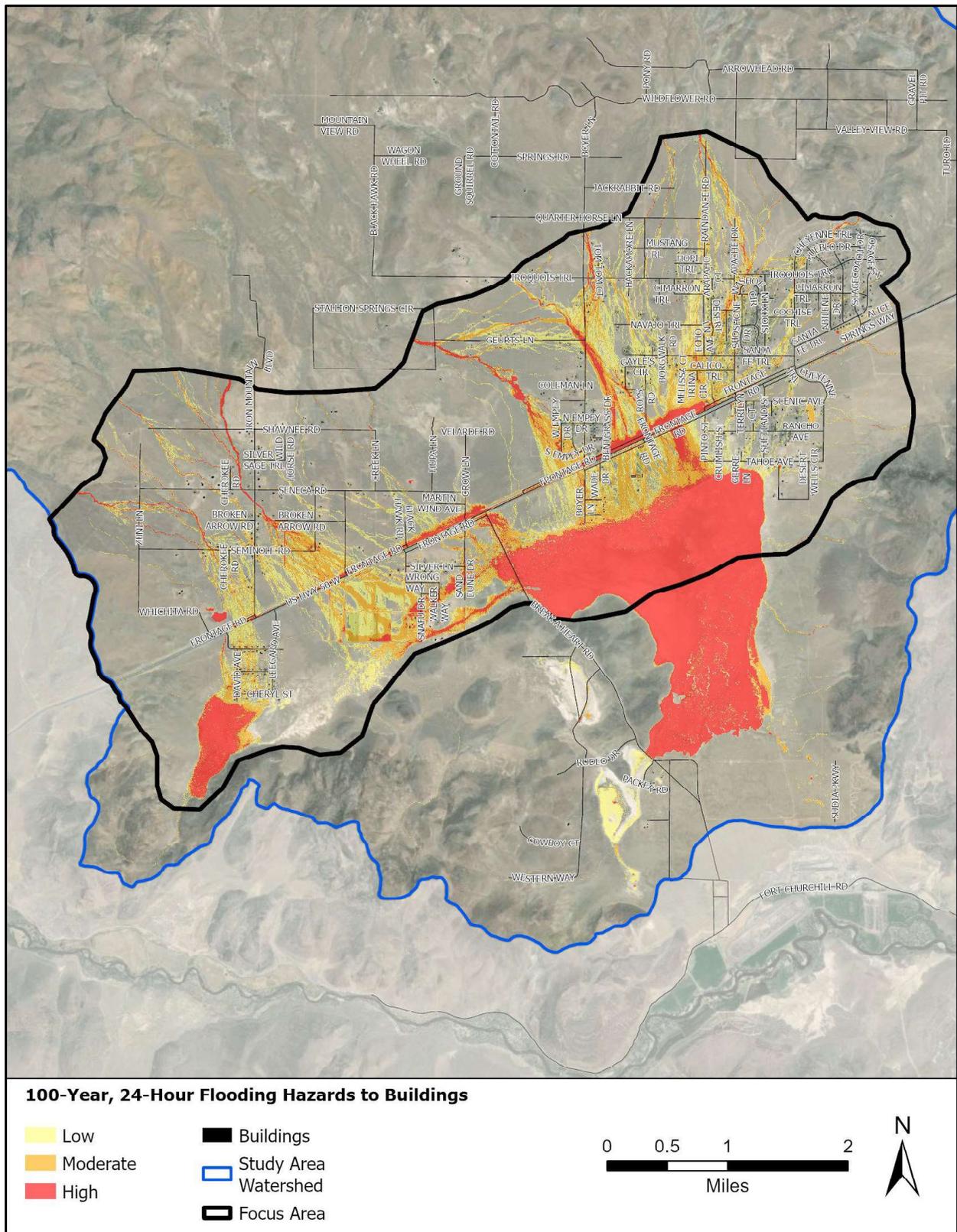


Figure 4-9. 100-year, 24-hour flood hazards to buildings

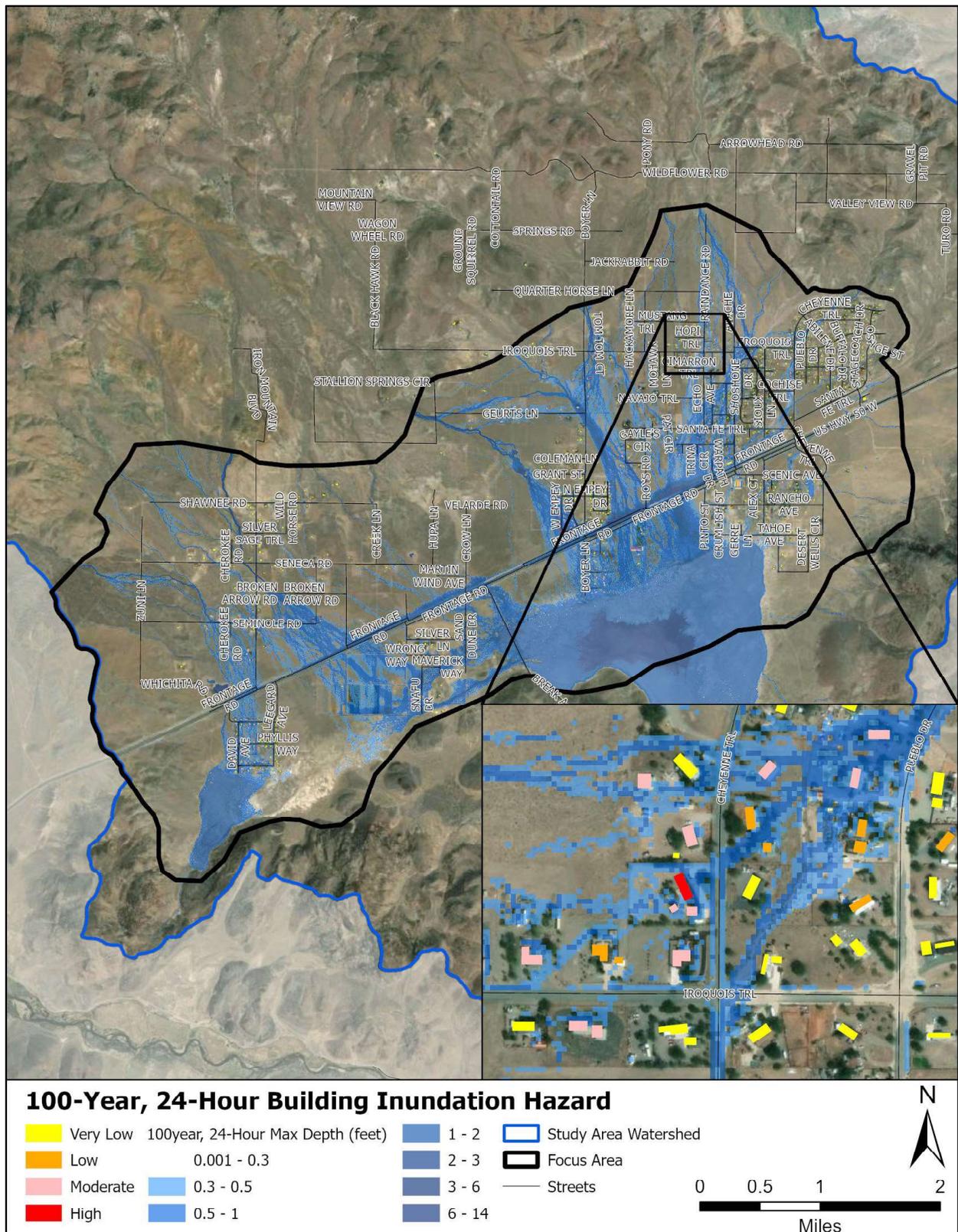


Figure 4-10. Building inundation assessment based on the 100-year, 24-hour FLO-2D results

# 5 FLOOD MITIGATION ALTERNATIVES

## 5.1 INTRODUCTION

The flood mitigation alternatives consisted of both regional and local concepts. Their development considered the following elements:

- 1) Flood hazard identification and evaluation
  - a. Data collection, Historic aerial review
  - b. Development of base conditions models
  - c. Sediment transport analysis
  - d. Flood hazard classification
- 2) Alternative formulation/evaluation
  - a. Brainstorming and potential structure identification
  - b. Coordination with Lyon County on acceptable types and locations of alternatives
  - c. Preliminary concept development and rough modeling
  - d. Refinement of beneficial concepts and more detailed modeling
- 3) Development of 15% design plans and cost estimates

JE Fuller (JEF) served as the lead on the flood hazard identification and alternative formulations with assistance from Lumos and Associates (Lumos), who were the lead in the development of the 15% design plans and cost estimates for the selected mitigation alternative. Figure 5-1 summarizes the process for developing the regional alternatives.

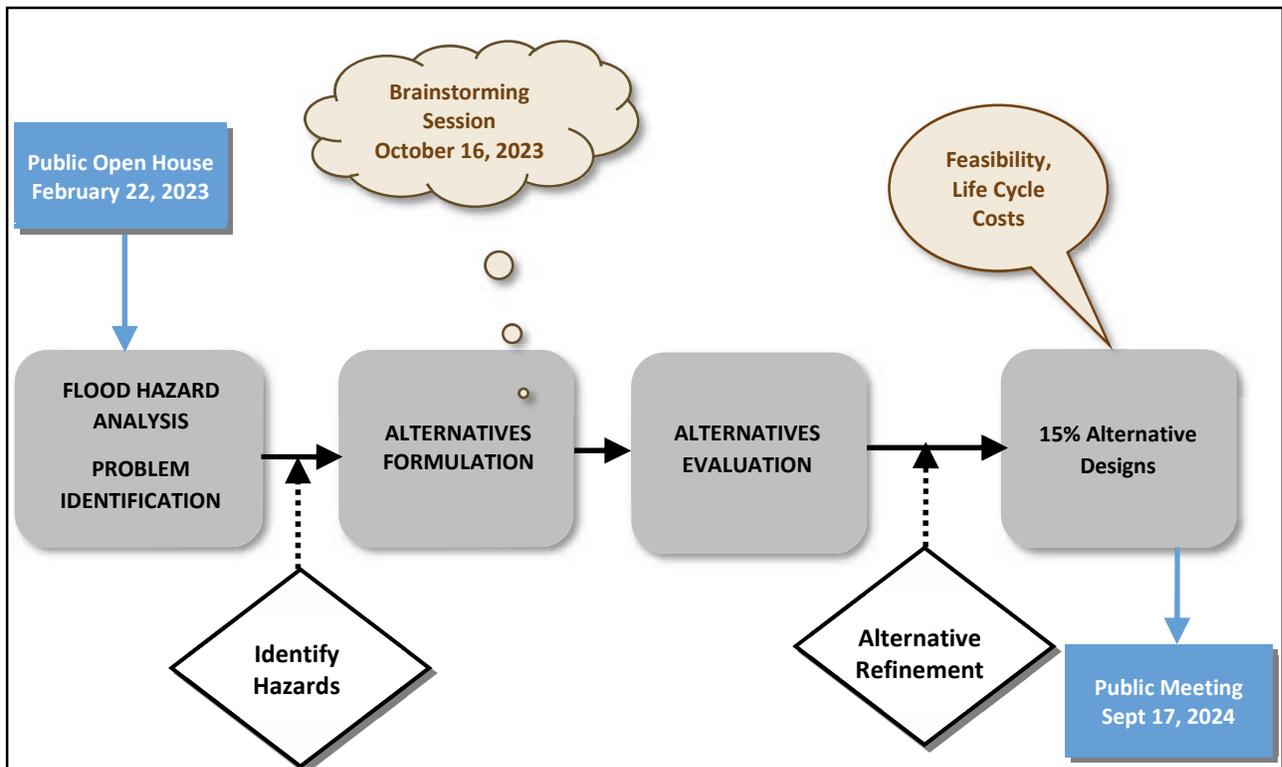


Figure 5-1. Development process for the regional flood mitigation alternatives for the ADMP

## 5.2 MITIGATION CHALLENGES

As described previously, much of the existing development in Stagecoach is located on a piedmont landform which is highly subject to flowpath uncertainty and distributary flow patterns. As illustrated in in Figure 2-9 through Figure 2-16, the depth and discharge modeling results indicate that offsite stormwater flow is generally shallow and widely distributed throughout the study area once it leaves the upper watershed mountain channels. This type of flood risk poses a challenge for developing both regional and local mitigation concepts. In addition to the flood risk, the study area is also subject to significant sedimentation impacts during storm events as was also described previously. Local solutions such as construction of roadway culverts or improvements to existing culverts often provide minimal benefit when distributary flow crosses roads in dozens of locations. Also, without proper control of sediment and ongoing maintenance, culverts are quickly clogged and overwhelmed during storm events. Combining these challenges with flowpath uncertainty due to upstream active alluvial fan flooding which could result in flow changing direction and magnitude, possibly leaving constructed culverts dry and ineffective, results in great difficulty in design projects that provide regional flood mitigation.

One of the primary goals of this ADMP was to develop regional mitigation concepts that would provide a benefit to the downstream community. Regional mitigation is often the most effective for active alluvial fan flooding. The combination of rainfall and hydrologic conditions of the study area watershed results in significant stormwater volumes being generated for the storm events evaluated. These large volumes also pose a challenge as regional flood mitigation structures must either have the capacity to store a significant amount of the runoff volume or have the capacity to convey the flows safely downstream to be effective. In addition to runoff volume, mitigation structures must also be designed to account for sediment being transported by the runoff. The combination of high runoff and large sediment volumes necessitates large mitigation structures, which are often costly and require large areas of land. Also, since the basins need to be placed before the flow becomes distributary, they are generally on high slope (> 3%) areas. When slopes are this high, basins tend to have an over excavation factor. That is, the amount of dirt that needs to be moved is much larger than the runoff volume effectively stored. This has the effect of greatly increasing the cost of the structure. While none of these challenges are insurmountable, they can pose constraints when attempting to develop a cost-benefit analysis for the structures. The ADMP project team encountered all these challenges during the development of the concept mitigation alternatives.

The ADMP technical team met with Lyon County officials early in the project schedule and discussed any potential constraints that the county might have regarding regional mitigation structures. Lyon County was not in favor of any potential mitigation structure that would result in the designation of a jurisdictional dam as defined by the Nevada Division of Water Resources. Additionally, Lyon County was not in favor of mitigation alternatives which required the purchase of existing developed property or construction of large culverts to cross US 50 since that road was recently widened and improved. The alternatives presented in the following sections are the result of collaboration between the project team and Lyon County and are the outgrowth of many discussions regarding the challenges and potential impacts of the concept mitigation structures.

## 5.3 MITIGATION ALTERNATIVES

Through the development process outlined above, the project team and Lyon County collaborated on a system of flood mitigation concepts that consisted of both regional and local structures that will help mitigate flooding and sedimentation problems within the study area. These alternatives are discussed in more detail in the sections below.

### 5.3.1 Regional Alternatives (Major Projects)

The regional alternatives consisted of six large regional basins with appurtenant collector and conveyance channels. Their design characteristics are summarized in Table 5-1, while their locations are shown in Figure 5-2.

All alternatives considered the 100-year and 25-year, 24-hour storms. However, since volumes for the 25-year storm were also large, basin excavation was designed for the 100-year storm volume to provide a conservative estimate of cost. Since each basin has its own unique characteristics, a brief description of each one is provided below.

- **Iron Mountain Basin** – This basin is on the western edge of the study area and uses an existing abandoned NDOT borrow pit to help minimize additional excavation volume. A collector channel gathers additional distributary flow to the east and routes this flow to the basin.
- **Shawnee Basin** – This basin is located higher in the watershed and collects flow before it exits the mountain front and becomes distributary. It is located upstream of existing structures.
- **Geurts Basin** – This basin is located on a BLM parcel near the center of the Focus Area. It includes a conveyance channel that contains a flow breakout to the east and routes the full flow to the basin. The basin itself is made up of an upper and lower basin that allows additional flow attenuation through the culverts and weirs between the two separate basins.
- **Quarter Horse Basin** – This basin is located along Quarter Horse Lane north of the Focus Area and existing development. The basin location would require the relocation of Boyer Lane to avoid the basin footprint and to continue to provide access to the water storage tank at the west end of Jackrabbit Road upstream of the basin.
- **Apache Basin** – This basin is located near the northern edge of the Focus Area just downstream of an existing house. An upstream conveyance channel is located upstream of the basin to contain the flow and route it past the home and into the basin. A culvert would be needed to maintain access to the home, but this culvert was not included at this time.
- **Stagecoach Basin** – This basin is located on the east side of the study area near the main flow path that crosses Cheyenne Trail. This system consists of a basin, channel, levee, and weir. The system fully contains the entire 25-year 24-hour storm, but during the 100-year 24-hour event, some flow is designed to overtop the weir and flow in the historic path of the downstream channel. The 100-year peak flow is reduced from 245 cfs to 133 cfs in the downstream channel. All low flows are routed and stored in the basin.

Table 5-1. Summary of design characteristics for regional basin structures

Regional Structures	Excavation Volume (ac-ft)	Effective Storage Capacity <i>includes freeboard</i> (ac-ft)	100Y Peak Inflow (cfs)	25Y Peak Inflow (cfs)	100Y Inflow Volume (ac-ft)	25Y Inflow Volume (ac-ft)	100Y Sediment Volume (ac-ft)
<b>Iron Mountain Basin</b>	334.1	131.2	700.0	248.8	236.5	91.2	0.2
<b>Shawnee Basin</b>	1,044.1	114.0	1,586.0	672.8	497.8	215.1	8.9
<b>Geurts Basin</b>	445.3	13.3 Upper, 83.5 Lower	1,940.0	898.7	555.9	366.7	4.6
<b>Quarter Horse Basin</b>	926.7	210.3	854.8	328.3	332.9	132.4	2.8
<b>Apache Basin</b>	516.7	93.9	836.6	379.4	309.7	139.0	3.2
<b>Stagecoach Basin</b>	233.5	52.9	106.2	50.0	55.6	21.3	0.6

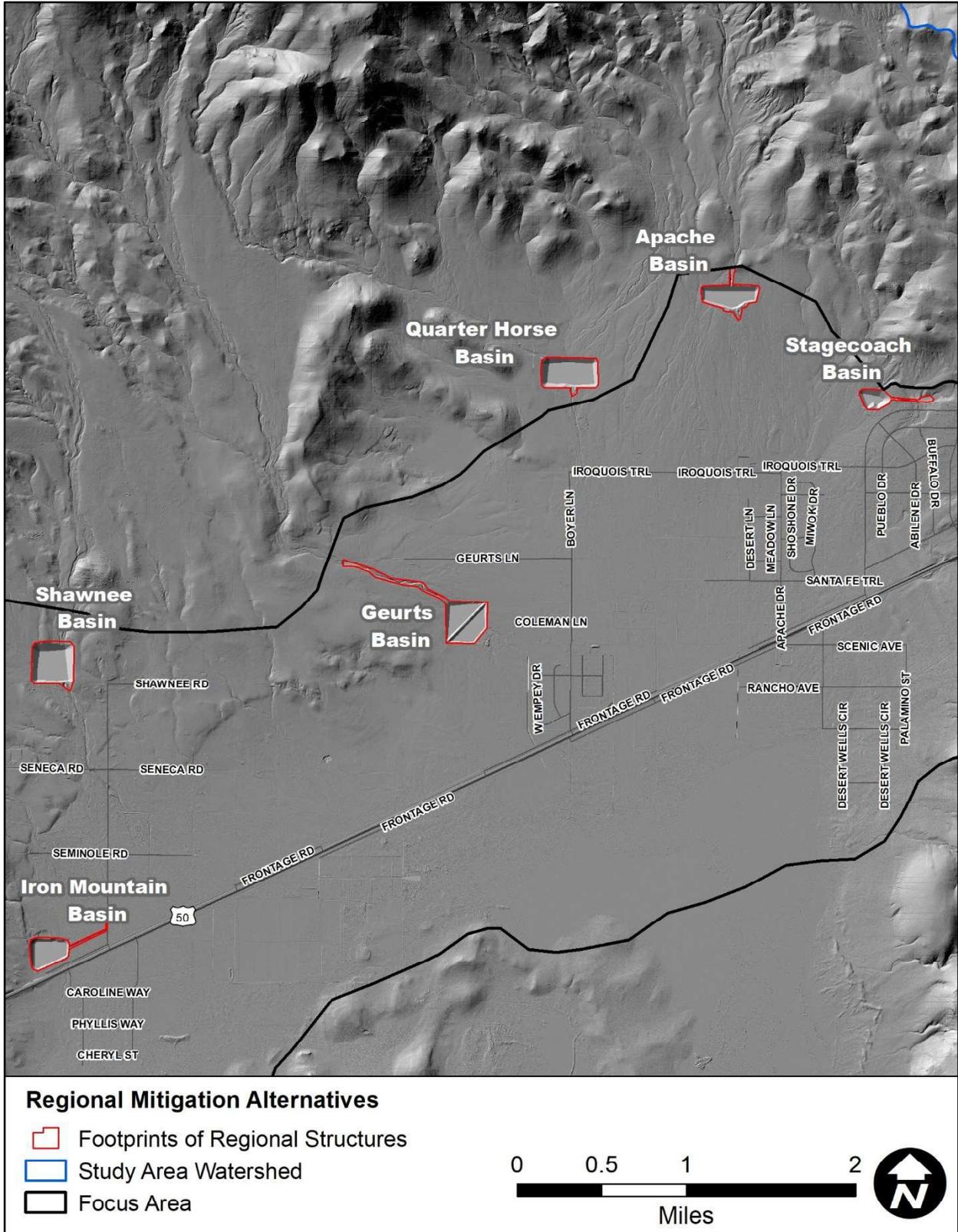


Figure 5-2. Locations of regional structures shown in hillshade

### 5.3.2 Local Alternatives (Minor Projects)

Since the basins are massive structures, four channels with more localized impacts (and lower construction costs) were investigated and proposed. The locations of these alternatives are shown in Figure 5-3, and each channel is briefly discussed below. The details of the design of the channels can be found in the Lumos Technical Memorandum in Appendix A.

- **Empey Channel** – These channels started as a part of the Geurts Basin alternative to capture and control outflow from the basin. The Empey Channel is located upstream (north) of US 50, with a channel segment downstream of the highway (Lincoln Channel). The two channels are connected by five 24-in pipe culverts. Note that these culverts do not pass the full flow in the channel for both the 100- and 25-year storms. Larger box culverts would be needed to pass the full channel flow, but the 24-in pipes are proposed since they are similar in size to the existing cross culverts on US 50 and would likely have a better chance of being approved by NDOT. Finally, the channel was designed to capture the full 100-year flow with or without the Guertz Basin in place. As such, these channels function independently of this basin.
- **Pinto/Rancho Channels** – These channels are minor roadside ditches located in an existing development on the east side of the Focus Area downstream of US 50. This development currently has minimal drainage infrastructure. One roadside ditch is proposed along the north side of Pinto Street, while the other is on the west side of Rancho Avenue. The channels combined as the intersection of Ranch Avenue and Pinto Street, and the combined flow is conveyed to the undeveloped area just north of the Playa. The current 15% design shows 18-in culverts for each driveway crossing, but this size of culvert does not pass the full flow in the ditch. As such, these culverts would reduce the efficiency of the channel, which will cause sediment to drop out at the culverts. Therefore, maintenance of both the channel and culverts are critical for the proper functioning of this alternative.
- **Blackhawk Channel** – This channel is located on the west side of Blackhawk Road downstream of US 50. The channel captures flow that overtops US 50 and impacts the existing house located east of Blackhawk Road. The channel routes this flow to the undeveloped area south of the homes.
- **Silver Lane Channel** – The Silver Lane Channel is not a defined channel. It is a minor grading project that will allow the ponded water in Silver Lane to flow out towards the east to the undeveloped area on the western edge of the Playa.

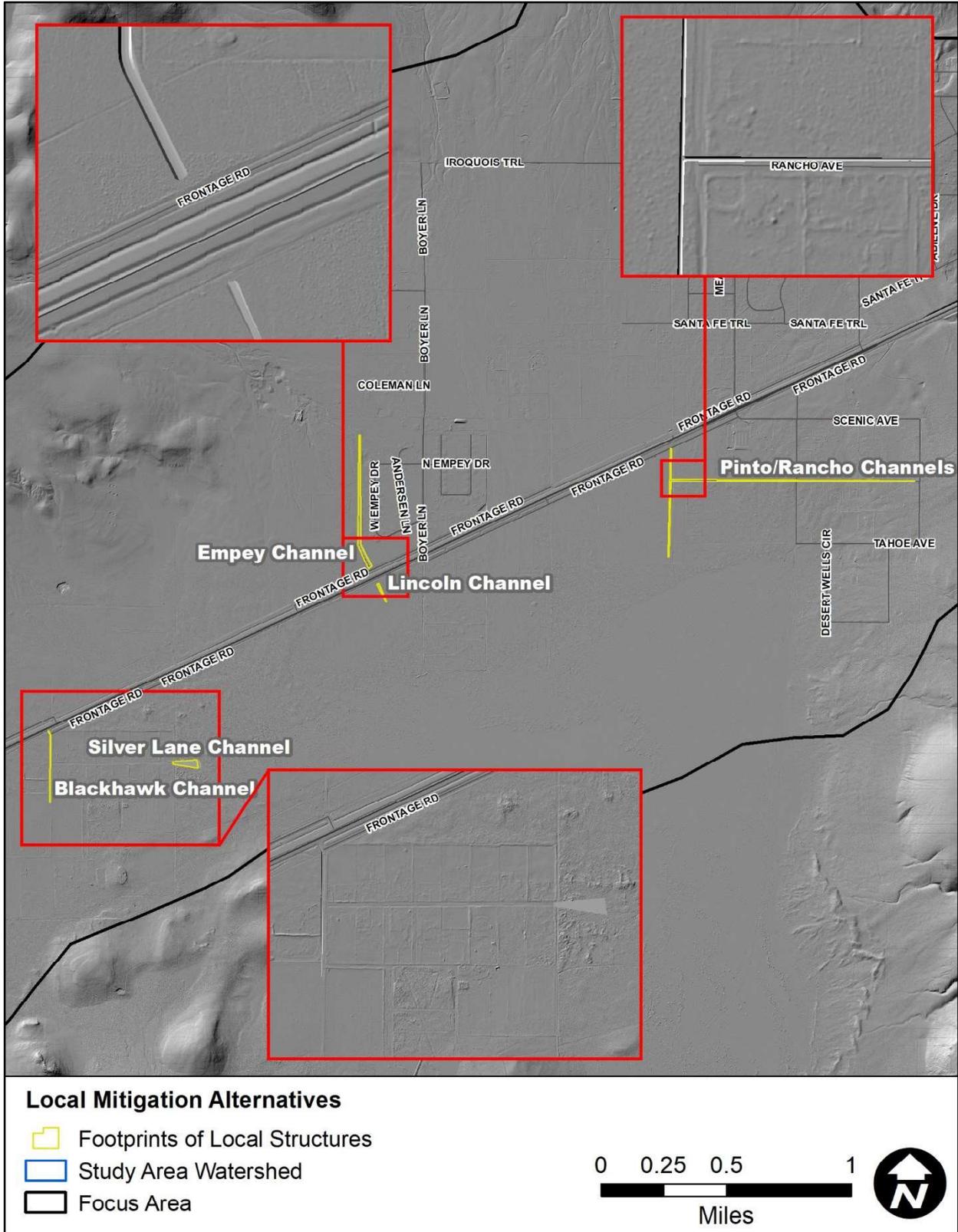


Figure 5-3. Locations of local channels shown in hillshade

### 5.3.3 Concept Plans

Lumos was tasked with developing conceptual, 15% design plans for the flood mitigation systems. The Lumos plan sets and accompanying technical report are included in Appendix A. An example design plan for a 100-year basin is shown in Figure 5-4, while an example of a local channel is shown in Figure 5-5. Note – Despite their detail, these design plans are still concept designs and are meant to outline costs and general characteristics of the proposed structures to allow for recommendation and prioritization. The design volumes, structure alignments, and other characteristics will be refined during final design based on constraints that are outside the scope of this study (e.g., utility conflicts).

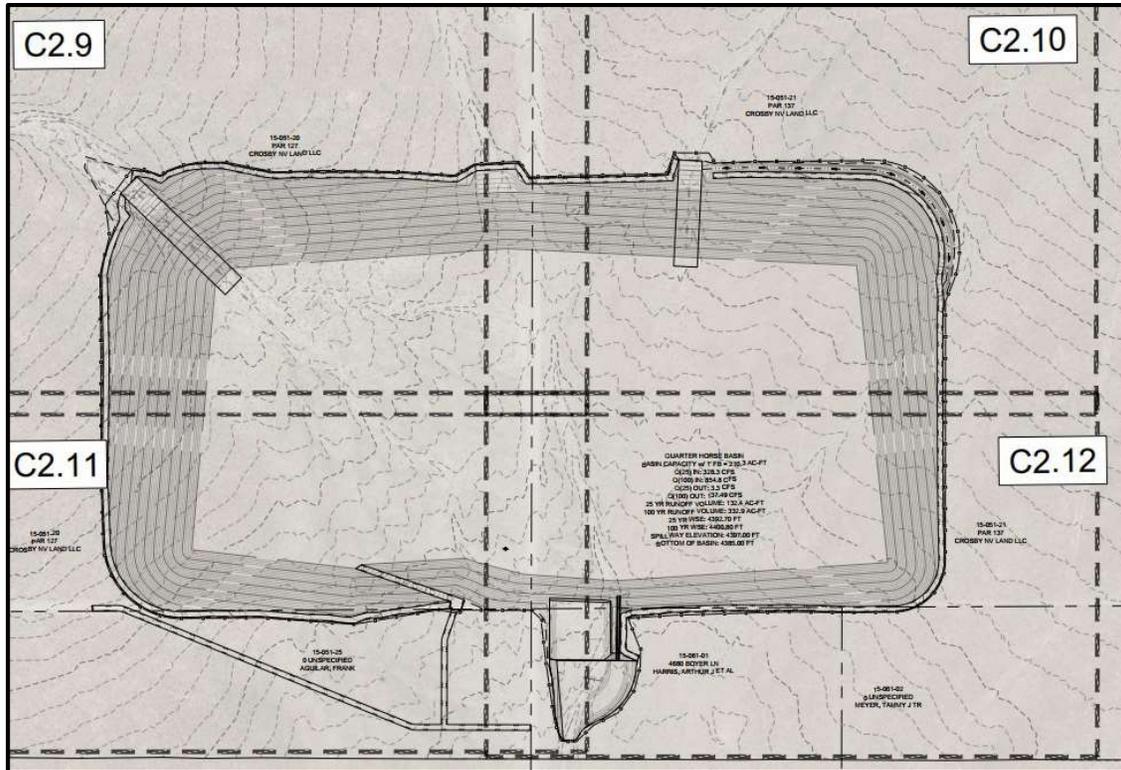


Figure 5-4. Example of 15% concept plans for a regional basin structure



Figure 5-5. Example of 15% concept plans for local channel structure

### 5.3.4 Summary of Costs

Since the slope of the watershed is steep where the basins need to be located, the excavation volume can be quite large when compared to the flood storage that needs to be provided (see Table 5-2). This, as well as the necessary erosion protection, causes the cost of the structures to be quite high.

Recognizing this, a detailed benefit analysis (Section 5.4) and a prioritization plan (Section 6.2) for implementation will be developed.

The 15% project costs are summarized in Table 5-2. Land acquisition (right-of-way or easements) costs are not included in these estimates and will need to be factored into the project costs during final design. Since these are 15% cost estimates, utility identification and other detailed analyses were not performed. As such, a 30% contingency was used in developing the estimates.

Table 5-2 15% project costs

Mitigation Alternative	Project Cost	O&M Cost (over 20 years)	20-year Net Present Values
<b>Regional Structures</b>			
Iron Mountain Basin	\$ 16,713,000	\$ 8,943,100	\$ 25,656,100
Shawnee Basin	\$ 51,500,500	\$ 9,669,400	\$ 61,169,900
Geurts Basin	\$ 28,234,800	\$ 9,835,900	\$ 38,070,700
Quarter Horse Basin	\$ 45,108,800	\$ 9,731,200	\$ 54,840,000
Apache Basin	\$ 28,753,500	\$ 9,123,400	\$ 37,876,900
Stagecoach Basin	\$ 30,178,300	\$ 8,453,800	\$ 38,632,100
<b>Regional Subtotal</b>	<b>\$ 200,488,900</b>	<b>\$ 55,756,800</b>	<b>\$ 256,245,700</b>
<b>Local Structures</b>			
Empey Channel	\$ 4,206,700	\$ 1,110,800	\$ 5,317,500
Black Hawk Channel	\$ 409,400	\$ 769,200	\$ 1,178,600
Silver Lane Channel	\$ 208,200	\$ 769,200	\$ 977,400
Rancho/Pinto Channels	\$ 2,108,000	\$ 769,200	\$ 2,877,200
<b>Local Subtotal</b>	<b>\$ 6,932,300</b>	<b>\$ 3,418,400</b>	<b>\$ 10,350,700</b>
<b>Total</b>	<b>\$ 207,421,200</b>	<b>\$ 59,175,200</b>	<b>\$ 266,596,400</b>

## 5.4 BENEFITS SUMMARY

### 5.4.1 Buildings Benefit

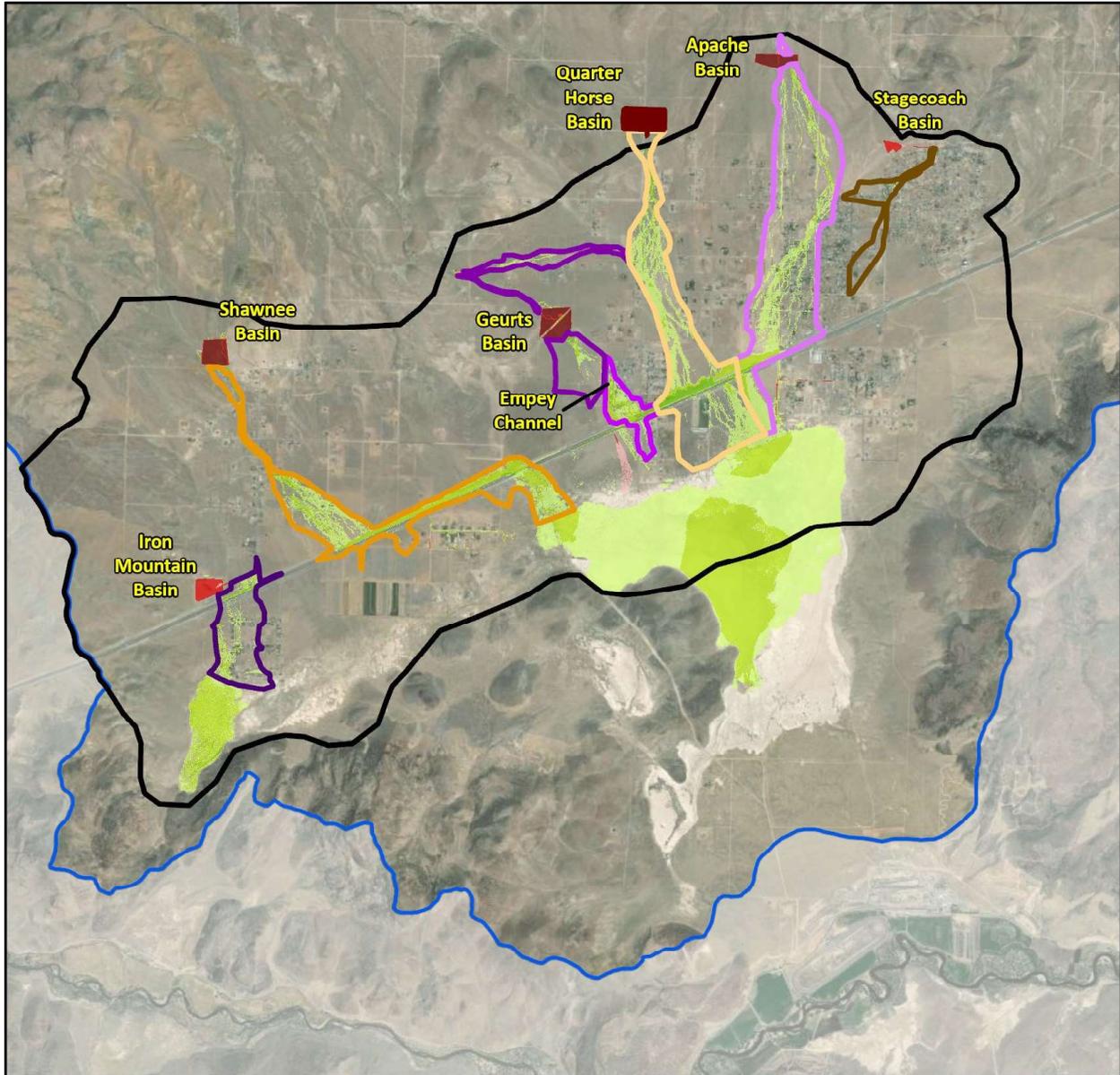
The depth analyses that were performed for existing conditions (Section 4.4) were repeated for the proposed conditions. The analyses were run for the 25-year, 24-hour, and 100-year, 24-hour project storm events. The proposed conditions analyses are summarized in Table 5-3 by Regional Alternative Project. The last column in the tables shows the estimated benefit for each storm event when compared to existing base conditions for that same storm event.

Table 5-3. Summary of depth analyses for buildings

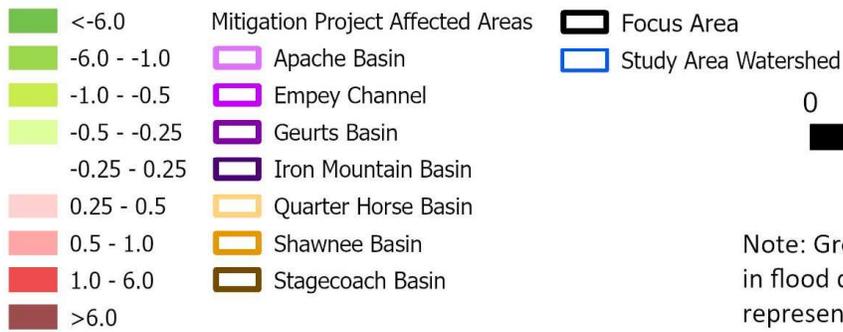
Regional Alternative Project	25-Year, 24-Hour Storm Existing Conditions			25-Year, 24-Hour Storm With Mitigation Alternatives			Total Buildings Removed
	Number of High Hazard Buildings <sup>1</sup>	Number of Moderate Hazard Buildings <sup>2</sup>	Number of Low Hazard Buildings <sup>3</sup>	Number of High Hazard Buildings <sup>1</sup>	Number of Moderate Hazard Buildings <sup>2</sup>	Number of Low Hazard Buildings <sup>3</sup>	
Apache Basin	2	51	75	0	4	32	92
Empey Channel	5	17	14	0	0	1	35
Geurts Basin	0	2	3	0	0	1	4
Iron Mountain Basin	2	8	15	1	2	4	18
Quarter Horse Basin	3	22	20	0	2	12	31
Shawnee Basin	1	3	1	0	0	2	3
Stagecoach Basin	0	10	13	0	0	8	15
Regional Alternative Project	100-Year, 24-Hour Storm Existing Conditions			100-Year, 24-Hour Storm With Mitigation Alternatives			Total Buildings Removed
	Number of High Hazard Buildings <sup>1</sup>	Number of Moderate Hazard Buildings <sup>2</sup>	Number of Low Hazard Buildings <sup>3</sup>	Number of High Hazard Buildings <sup>1</sup>	Number of Moderate Hazard Buildings <sup>2</sup>	Number of Low Hazard Buildings <sup>3</sup>	
Apache Basin	12	96	50	2	49	71	36
Empey Channel	15	37	20	0	3	14	55
Geurts Basin	1	13	11	0	2	8	15
Iron Mountain Basin	7	34	16	1	4	7	45
Quarter Horse Basin	13	33	13	1	12	24	22
Shawnee Basin	2	5	2	1	4	3	1
Stagecoach Basin	1	9	6	1	8	3	4
1. Depth: > 1' 2. Depth: 0.5' < h ≤ 1' 3. Depth: 0.25' < h ≤ 0.5'							

### 5.4.2 Floodplain Benefit

The regional alternatives would also result in a significant reduction in flood depths throughout the study area. To illustrate this benefit, Figure 5-6 and Figure 5-7 show the reduction in flood depth for the 25-year and 100-year, 24-hour storms. Overall, the alternatives provide a significant benefit to the community by reducing the flood depths. Note that the area downstream of the Empey Channel alternative shows an increase in flood depths. This is because the alternative includes a series of new culverts beneath Highway 50 to convey the flows beneath the highway and downstream to the playa. Because this alternative results in higher flood depths downstream of US 50, the county will likely need to acquire right-of-way or easements for the areas with higher depths.

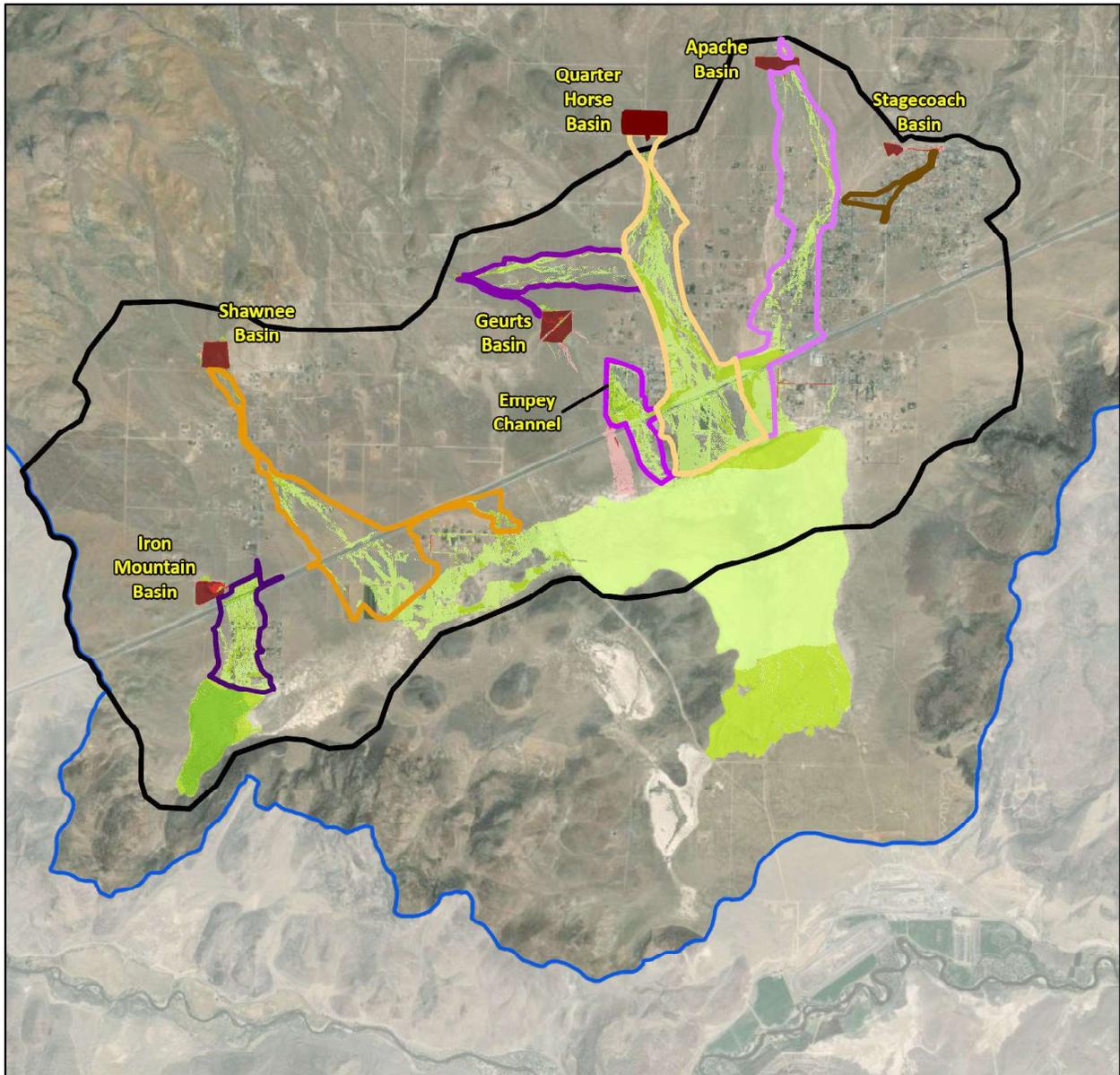


**Depth Change with Alternatives (25-year, 24-hour storm)**

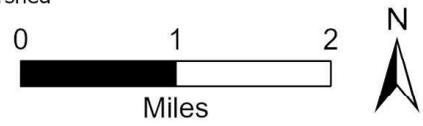
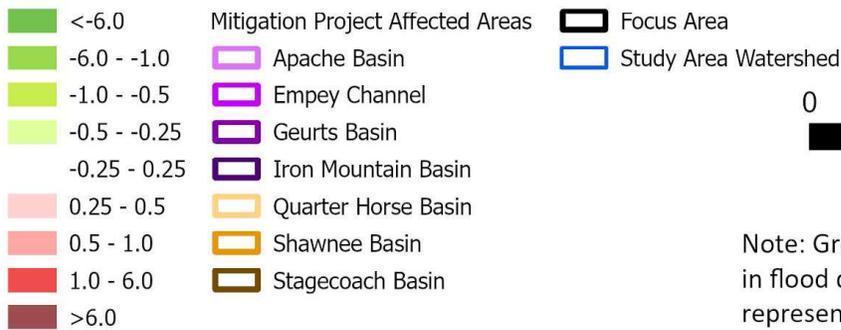


Note: Green colors represent a reduction in flood depths (benefit). Red colors represent an increase in flood depths.

Figure 5-6. Potential flood risk area benefit from the 25-year, 24-hour regional alternatives



**Depth Change with Alternatives (100-year, 24-hour storm)**



Note: Green colors represent a reduction in flood depths (benefit). Red colors represent an increase in flood depths.

Figure 5-7. Potential flood risk area benefit from the 100-year, 24-hour regional alternatives

In addition to a depth reduction benefit analysis, a floodplain area reduction benefit analysis was conducted. The total acreage that would experience a reduction in flood inundation for depths greater than 0.25 feet is **690 acres for the 25-year, 24-hour storm**, and **1,054 acres for the 100-year, 24-hour storm**. Note that this calculation excludes the playa areas which actually experience a significant reduction in inundation area but are unlikely to be considered for future development.

## 5.5 FUTURE DESIGN CONSIDERATIONS

Preliminary FLO-2D modeling of the 15% design alternatives highlighted several considerations for the final design:

- **Dynamic Nature of Upstream Topography:** The upstream terrain is dynamic, with channels potentially shifting. This may require the identification of new inlet locations.
- **Regional Sediment Calculations:** The sediment calculations used in this study were regional and generally applicable to channels with sandy bedload sediment. However, some channels in the watershed were observed to have coarser bedload sediment. Therefore, a more site-specific sediment analysis is recommended for each structure during the final design process.
- **Adverse Impacts and Emergency Spillways:** If any mitigation system is implemented, the emergency spillways need to be carefully evaluated and designed to ensure they do not cause adverse impacts during larger events.
- **Percolation Testing:** The basins retain large volumes of water, and percolation tests should be performed to verify drain time requirements for vector control requirements.
- **Design Refinement:** When each regional structure goes into the final design process, it is recommended that design refinement and a value analysis be performed to identify areas where the 15% design could be optimized. For example, the Stagecoach Basin routes all low flows to the basin, but a culvert could be added that discharges low flows to the historic channel to allow the basin to capture more of the 100-year volume.

## 5.6 FLOOD RISK QUICK REFERENCE

An existing conditions flood risk dataset was developed from the ADMP 100-year, 24-hour flowpath uncertainty combined maximum flow depth data. The flood risk dataset is a simplified GIS polygon layer representing maximum flow depth. Our goal in developing this dataset was to provide Lyon County with a quick reference tool to evaluate potential flood risk against future development permit requests. Figure 5-8 shows the flood risk dataset. Note – the figure is intended to illustrate the dataset; it is recommended that the digital GIS layer be used during regulatory analyses. It is recommended that this dataset be used in conjunction with the Development Guidelines that were drafted for both the Dayton Valley ADMP and South Dayton Valley ADMP studies when evaluating future development within the Stagecoach area.

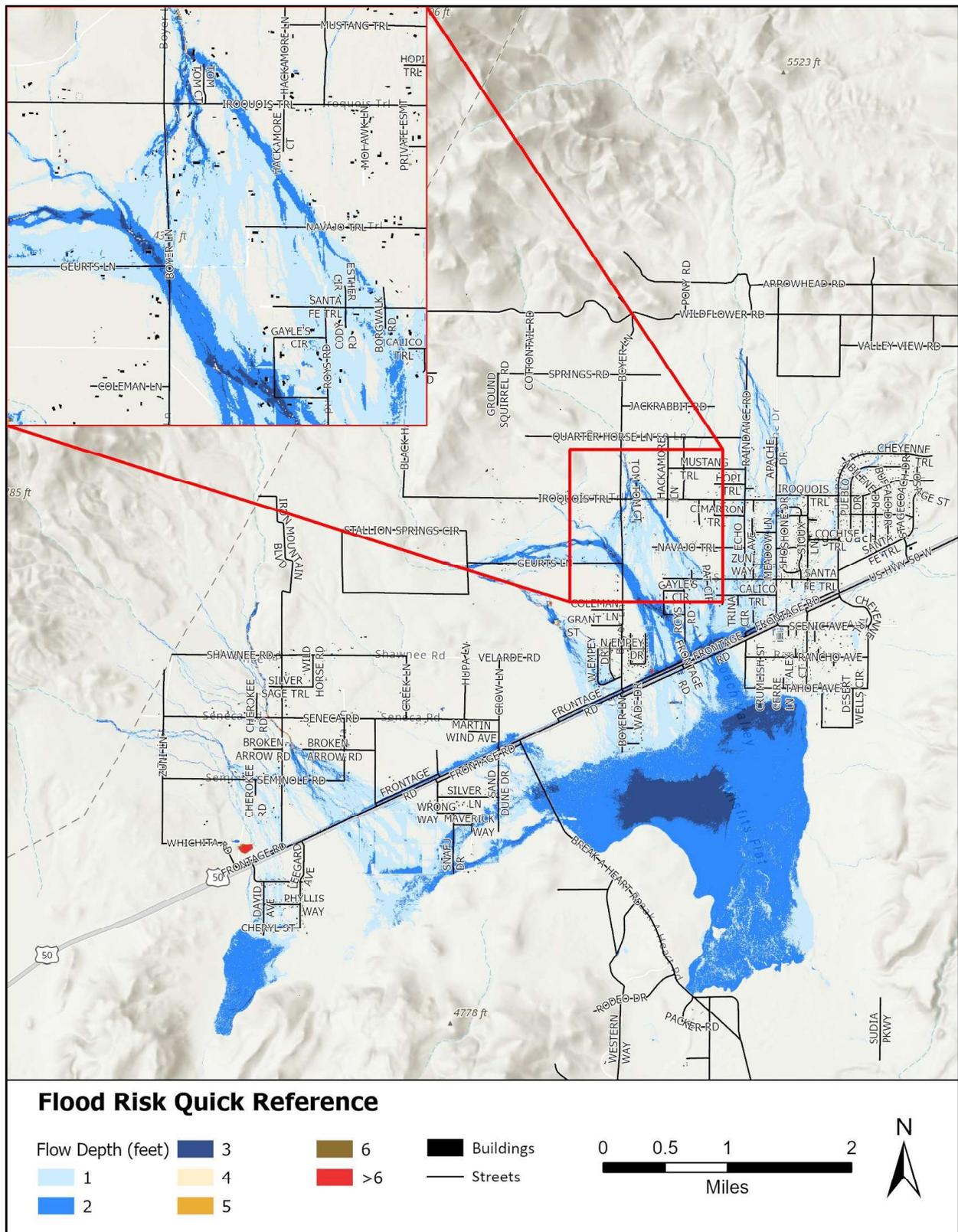


Figure 5-8. Flood risk quick reference dataset

## 6 ADMP SUMMARY AND RECOMMENDATIONS

### 6.1 SUMMARY

Through the development of the base (or existing) conditions hydrology and hydraulics and the classification of existing flood hazards, flood prone areas were identified. To protect these areas, a total of ten mitigation alternatives were proposed, designed (to a 15% level), and evaluated for their benefit. These alternatives consisted of six regional structures and four more localized structures (i.e., their benefits fall with a small area close to the improvement and their design is on a much smaller level). The estimated benefits were weighed with each structure’s project cost, and a preliminary prioritization plan is proposed. This prioritization plan and structures that could not be recommended are outlined in the next sections.

### 6.2 PROJECT PRIORITIZATION

The project team evaluated the overall cost and benefits (buildings removed from flood risk) of each of the potential mitigation projects presented previously with the goal of recommending a prioritization plan for Lyon County. Lyon County is under no obligation to conform to this recommendation and may ultimately elect to implement some (or none) of the mitigation projects described in the ADMP report. Table 6-1 lists the recommended project priority.

*Table 6-1. Recommended mitigation project priority*

Priority	Mitigation Project	Project Type	Total Cost <sup>1</sup>
1	Empey Channel	Local	\$ 5,317,500
2	Pinto/Rancho Channels	Local	\$ 2,877,200
3	Blackhawk/Silver Lane Channels	Local	\$ 2,156,000
4	Iron Mountain Basin	Regional	\$ 25,656,100
5	Apache Basin	Regional	\$ 37,876,900
6	Quarter Horse Basin	Regional	\$ 54,840,000
7	Stagecoach Basin	Regional	\$ 38,632,100

<sup>1</sup>Includes Initial Project Cost and 10-year O&M Costs

### 6.3 ALTERNATIVES NOT RECOMMENDED

After careful consideration, the project team is not recommending the following mitigation projects due to their overall cost and relative benefit to the community:

- Geurts Basin – the site for this basin was selected primarily because it is located within a BLM parcel, thus potentially reducing the overall project cost by not having to purchase the property. Several design alternatives were considered for this basin, and the final recommended design optimizes the volume storage within the stie. Even with this optimization, the overall basin size is not highly effective in storing enough runoff volume to substantially reduce downstream impacts. In fact, some adverse impacts were observed downstream of the basin during the 100-year event. In addition, the downstream Empey Channel provides more direct benefit at a lower cost.
- Shawnee Basin – the benefits analysis indicates that the Shawnee Basin removes the fewest number of buildings from the flood risk than any of the other basins. This is primarily because the downstream impact area is largely undeveloped as of the date of this study. The overall cost of this basin does not justify its minimal benefit at this time. Should future development be proposed in the downstream impact area, this basin may become a higher priority.

### 6.4 STUDY LIMITATIONS

While the results are based on detailed topography, hydrology, and hydraulic modeling, they represent the conditions as of the date of the LiDAR mapping. Because of the landform and sediment characteristics of the watershed, the topography and distribution of flow can be very dynamic. Therefore, during final design of any of the alternatives or prior to any future development within the project area, an assessment of current flow conditions should be performed to ensure the applicability of the ADMP modeling results.

Furthermore, this study did not analyze rain on snow events, flooding recurrence intervals greater than 100-year, or post-wildfire flooding events. These types of events are considered outside the scope of the typical area drainage master plan process. The hydrology used in this study is state of the art, engineering design storms based on recent NDOT research. These atypical events could create hydraulic conditions that exceed these design storms.

## 7 REFERENCES

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Digital Aerial Solutions, LLC (DAS), 2018a, NV Reno Carson Urban Lidar 2017 B17 Survey Report USGS Contract: G16PC00044 Task Order Number: G17PD01257, modified date of March 23, 2018.

\_\_\_\_\_, 2018b, LiDAR Project Report G17PD01257, NV Reno Carson City Urban, modified date of March 29, 2018.

Field, John, 2001, "Channel avulsion on alluvial fans in southern Arizona," *Geomorphology*, Vol. 37, p. 93-104.

FLO-2D Software, Inc., 2021, FLO-2D Data Input Manual, October 2021 – Build 21.

Floyd, B., 2017, Alluvial Fan Mapping for the Carson River Watershed Methodology. U.S. Army Corps of Engineers, Sacramento District.

Jarrett, R. D., 1985. Determination of Roughness Coefficients for Streams in Colorado, USGS Water Resources Investigations Report 85-4004.

JE Fuller, Inc. (JEF), 2019, Dayton Valley Area Drainage Master Plan Technical Support Data Notebook. Lyon and Storey Counties, Nevada.

\_\_\_\_\_, 2020a, Sun Valley ADMPU Roughness Determination. Memorandum prepared for the Flood Control District of Maricopa County, dated April 30, 2020.

\_\_\_\_\_, 2020b, South Dayton Valley Area Drainage Master Plan Technical Support Data Notebook. Lyon County, Nevada.

\_\_\_\_\_, 2020b, Modernize Hydrologic Prediction Processes by Creating Custom Statewide SSURGO Green and Ampt Parameter Database, Green and Ampt Rainfall Loss Parameters. NDOT Project No. P674-19-803. December 2020.

Nevada Department of Transportation (NDOT), 2015, Streamlining Hydrologic Prediction Processes Using New and More Accurate Techniques and Methods, NDOT Research Report, Report No. 530-13-803.

NV5 Geospatial, 2022. EarthMRI\_2020\_D20 Lidar Processing Report. Work Unit 300013 Lidar Acquisition Task Order, issued by USGS under Contract G16PC00016.

Slingerland, R and Smith, N.D., 2004, River Avulsions and Their Deposits, Annual Review of Earth and Planetary Science, Vol. 32:257-285.

US Army Corps of Engineers (USACE), 2016. HEC-RAS, River Analysis System Hydraulic Reference Manual. Version 5.0. February 2016.

US Bureau of Reclamation (USBR), 1988, Downstream Hazard Classification Guidelines, Acer Technical Memorandum No. 11.

Yang, C. T., 1973, Incipient motion and sediment transport, *Journal of the Hydraulics Division, ASCE*, 99 (HY10), pp. 1670-1704.

\_\_\_\_\_, 1984, Unit stream power equation for gravel. *Journal of Hydraulic Engineering, ASCE*, 110(HY12), pp. 1783-1797.

Yochum, S.E., Comiti, F., Wohl, E., David, G.C.L., Mao, L. 2014. Photographic Guidance for Selecting Flow Resistance Coefficients in High-Gradient Channels. USDA, Forest Service, Rocky Mountain Research Station, General Technical Report, RMRS-GTR-323

## **APPENDIX A**

- **Concept Design Technical Memorandum**
- **Concept Design Sheets**
- **Construction Cost Estimates**
- **Life-Cycle Cost Estimates**

**STAGECOACH ADMP  
DESIGN SUMMARY &  
LIFE CYCLE COST ANALYSIS**

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## Attachments

- Attachment A – Opinions of Probable Construction Costs
- Attachment B – Preliminary Hydraulic Calculations
- Attachment C – 15% Design Drawings

## **I. INTRODUCTION**

Lumos & Associates has completed the preliminary designs and grading of stormwater improvements for the Stagecoach area in Lyon County, Nevada. This analysis was performed as part of the Stagecoach Area Drainage Master Plan (ADMP) being provided by JE Fuller to the Carson Water Subconservancy District and Lyon County. This memorandum presents an overview of the 15% preliminary designs and schematic grading, cost estimate, and life cycle cost analysis for the 25 and 100-year storm events based on data provided by JE Fuller.

### **Preliminary Design**

Channel, basin grading, and preliminary design of the storm drain structures were completed in accordance with 2018 Lyon County Drainage Guidelines. Each alternative design is based on conveying flows for the 25 and 100-year storm events. Costs for each option can be found in Attachment A. Preliminary hydraulic calculations for each option are included in Attachment B.

It should be noted that all basins impound over 20 acre-feet, but have been designed to hold the impounded water below existing grade, and therefore do not meet the criteria for being a jurisdictional dam in the state of Nevada.

In the context of this report, the definition of a minor versus major project is the difference between solving minor flooding that is seen during most storm events, versus large stormwater systems that mitigate impacts from the 25 and 100-year storm events. For example, roadside ditches have been provided in various locations to help flush flows out of neighborhoods.

Below is a summary of each project proposed. In total, 10 projects were evaluated to address the stormwater runoff issues identified as a part of the ADMP. Each proposed project is an independent project that can be constructed as an overall phased approach to address the stormwater needs of the area. Preliminary design level plans are also provided for reference as Attachment C.

## **II. MAJOR PROJECTS**

### **1. Common Design Elements**

Several design elements are common between all projects. The basin projects all have a similar intent: to capture and detain flows and release flows at a metered rate. This is achieved by grading a stormwater basin into the existing ground on available open space, utilizing inlet spillways, low flow outlet culverts, and outlet spillways. Each basin includes an access road into the bottom of the pond for access and maintenance. Additionally, fencing has been specified around the basins for public safety and to protect the longevity of each stormwater basin.

To protect against general soil erosion from sheet flows, a geotextile (Pyramat-75) has been provided on all disturbed slopes where rip rap or concrete is not included. This will protect the soils against sheet flow and allow for revegetation to take hold on the steeper slopes. For the sake of being conservative, Pyramat-75 was chosen for all slopes, however, may not be necessary

following a geotechnical investigation. Additional considerations for slopes may be considered during final design.

Erosion control and stabilization have been preliminarily designed for all basins and included with the plans. Final design will need to provide and include rock riprap, scour and other required calculations for all erosion control and stabilization.

Three times the annual sediment load plus the sediment load from one 100-year storm, 24-hour event was used to estimate the reduced volume for each detention.

## 2. Stagecoach Basin

The Stagecoach Basin intercepts flow from the wash to the northeast at the intersection of Stagecoach Drive and Cheyenne Trail. The basin contains a cut-off channel that collects the water and directs it westerly, parallel to Cheyenne Trail. The basin is proposed to act as a retention basin and will require percolation tests during final design to determine the time to drain per vector control requirements.

The project includes a realignment of approximately 1,700 linear feet of the existing channel. The proposed realignment redirects the runoff from its natural flow path and retains it slightly north in APN 015-071-31. The basin is designed to fully retain 100% of the 25-year event. A channel with a 20-foot bottom width, four-foot maximum depth, and 2:1 side slopes was designed to convey the 100-year event to the proposed Stagecoach Basin. An average velocity of 2.7-fps is expected for a longitudinal slope of 0.003 ft/ft and a rip-rap class of 300. It should be noted that a portion of the channel acts as a levee due to the raised nature of the channel side slopes. Additional consideration will need to be given during final design.

A 64 foot concrete weir has been provided just north of APN 019-411-07 to release flows above the 25-year event into the existing drainage path. This weir has been provided to reduce the required size of the Stagecoach Basin for the 100-year storm event. During the 25-year event, flows do not overtop the weir, and contribute to the Stagecoach Basin without overtopping. During the 100-year event, a portion of the runoff overtops the weir, and runs downstream to the existing flow path. The flows that do not overtop the weir, contribute to the Stagecoach Basin and are retained completely without overtopping. The basin was sized to fully retain the flows contributing from the 100-year storm event in order to not negatively impact downstream properties from the Stagecoach Basin. This volume was estimated running a HEC-RAS model to determine the overtopping flows from the weir, leaving the remaining flows and volume to contribute to the basin.

The channel discharges the runoff about 53 vertical feet above the bottom of the proposed basin. A 4:1 USBR Type IX baffled spillway is proposed to control the flows down the slope into the basin. A concrete stilling basin is proposed at the toe of each spillway to minimize erosive forces at the bottom of the basin.

The basin has a proposed volume of 52.8 ac-ft with a depth of 14 feet and 1 feet of freeboard. The basin has a 50' wide concrete spillway to release flows in excess of the 100-year storm. The spillway has a rip rap outfall to dissipate energy.

The estimated sediment loading for the basin is approximately 0.599 acre-feet. This volume is approximately 1.1% of the overall volume of the basin.

Based on the preliminary design, approximately 387,242 cubic yards of cut and 10,518 cubic yards of fill are expected during construction, leaving a net export of 376,714 cubic yards, respectively. The total area of disturbance is approximately 15.0 acres.

Drainage easement or property acquisition will be required on APNs 019-429-01 and 015-017-31.

### 3. Empey Channel

The Empey Channel is designed to intercept flow from upstream runoff areas and protect the houses along Empey Drive, Anderson Lane, and Boyer Lane. The channel is proposed to collect and convey the 100-year storm event.

The channel has a minimum longitudinal slope of 1%, a maximum longitudinal slope of 1.2%, has a 30-foot wide bottom width, a maximum channel depth of 6-feet, and 2:1 side slopes. During the 100-year storm event, maximum flow depth is modeled to be 5.6 feet. The channel is proposed to be lined with Class 300 rip-rap. Other linings such as an engineered geotextile could be considered during final design if the soils are sufficient. Empey Channel has been designed to work in conjunction with Geurts Basin and convey the 100-year discharge of the basin. However, in the event that Empey Channel gets built prior to Geurts Basin, the channel is capable of conveying 100% of the un-mitigated 100-year storm event.

The channel discharges to five 24-inch RCP culverts underneath Highway 50. An inlet and outlet headwall is provided. A 25-foot bottom width channel is provided at the downstream end of the culverts. This channel tapers to zero foot height to release the flows back into the existing conditions. Five 24-inch RCP culverts were chosen to provide the hydraulic capacity while minimizing cover requirements. With the 24-inch culverts, a 0.2% longitudinal sloped, 400 foot long channel is required to daylight to existing conditions downstream.

Based on the preliminary design, approximately 22,170 cubic yards of cut and 4 cubic yards of fill are expected during construction, leaving a net export of 22,166 cubic yards, respectively. The total area of disturbance is approximately 3.3 acres.

Drainage easement or property acquisition will be required on APNs 015-391-04 and 015-391-05. Coordination with NDOT will be required in order to construct the culverts across the Highway 50 corridor.

#### 4. Apache Basin

The Apache Basin intercepts flow from the wash between the intersection of Wildflower Road and Apache Drive, and the eastern end of Boyer Lane. This project also includes channelizing the flow coming out of the mouth of the canyon before it can start to spread due to the alluvial fanning and direct those flows to the proposed basin. The basin is proposed to act retain runoff and will require percolation tests during final design to determine the time to drain per vector control requirements.

A non-typical channel starts at the mouth of the canyon and necks down to a channel with a 20-foot wide bottom width 10-foot (max) depth, and 2:1 side slopes, overall, channelizing runoff for approximately 620 feet. The channel was designed to convey the 100-year event. An average velocity of 10.7-fps is expected for a slope of 0.053 ft/ft and a rip-rap class of 300. The channel discharges the runoff about 35 vertical feet above the bottom of the basin. A 4:1 USBR Type IX baffled inlet spillway is proposed to control the flows down the slope into the basin. A concrete stilling basin is proposed at the toe of the spillway to minimize erosive forces at the bottom of the basin.

The basin has a proposed volume of 93.9 ac-ft with a maximum depth of 12 feet and 1 feet of freeboard. The basin has three concrete outlet spillways with varying widths: two 50-foot and one 100-foot. The spillways were placed to minimize potential downstream impacts to properties with the concentrated discharges. In order to not increase any downstream flows, wide concrete spillways were provided to replicate the existing conditions. Additionally, the basin has two 18-inch culverts that act as low flow releases and to keep the accumulated detained volume moderately low prior to the peak of the storm event arriving. The culverts are placed two feet above the bottom of the basin to allow for two feet of retention volume to alleviate minor flooding during smaller storm events and to reduce total volume of runoff arriving at the playa.

The estimated sediment loading for the basin is approximately 3.21 acre-feet. This volume is approximately 3.4% of the overall volume of the basin.

Based on the preliminary design, approximately 834,125 cubic yards of cut and 462 cubic yards are expected during construction, leaving a net export of 833,662 cubic yards, respectively. The total area of disturbance is approximately 27.5 acres.

Drainage easement or property acquisition will be required on APNs 015-051-01, 015-081-10 and 015-041-45.

#### 5. Quarter Horse Basin

The Quarter Horse Basin intercepts flow from the wash at the intersection of Boyer Lane and Quarter Horse Lane. The basin requires the relocation of a portion of Quarter Horse Lane, and Boyer Lane to access the water tank located on APN 015-051-43. The basin is proposed to act as a retention basin and will require percolation tests during final design to determine the time to drain per vector control requirements.

There are two main entry points to the basin: the west and the east inlet spillways. 4:1 USBR Type IX baffled spillways are proposed to control the flows down the slope into the basin. A concrete stilling basin is proposed at the toe of each spillway to minimize erosive forces at the bottom of the basin.

The basin has a proposed volume of 210.3 ac-ft with a maximum depth of 12 feet and 2 feet of freeboard. The basin has one 100-foot wide concrete outlet spillway. Additionally, the basin is equipped with two 18-inch outlet culverts. The culverts are placed eight-feet above the bottom of the basin to allow for eight feet of retention volume to alleviate minor flooding during smaller storm events and to reduce total volume of runoff arriving at the playa. The existing downstream hydraulic conditions of Quarter Horse Basin area are more channelized than the adjacent basins, and therefore requires less spillway length to mimic existing conditions.

The estimated sediment loading for the basin is approximately 2.81 acre-feet. This volume is approximately 1.3% of the overall volume of the basin.

Based on the preliminary design, approximately 1,495,903 cubic yards of cut and 877 cubic yards of fill are expected during construction, leaving a net export of 1,495,025 cubic yards, respectively. The total area of disturbance is approximately 39.6 acres.

Drainage easement or property acquisition will be required on APNs 015-051-20, 015-051-21 and 051-061-01.

## 6. Geurts Basin

The Geurts Basin project channelizes flow from the wash just west of the intersection of Geurts Lane and Stallion Springs Circle. A channel is proposed to channelize flow from this wash and direct it to the proposed basin to be located on APN 015-382-10. Currently, flows from the 25 and 100-year storm events break out from the existing natural drainage channel and head easterly parallel to Geurts Lane. The proposed channel prevents this breakout and ensures all flow up to the 100-year event is discharged into the Geurts Basin. The basin is proposed to act as a retention basin and will require percolation tests during final design to determine the time to drain per vector control requirements. The basin has two tiers, separated by a spillway.

A non-typical channel starts at APN 015-342-01 and necks down to a typical channel with a 20-foot bottom width, 6-foot (max) depth, and 2:1 side slopes, overall, channelizing runoff for approximately 3,800 feet. The channel was designed to convey the 100-year event. An average velocity of 10.7-fps is expected for a slope of 0.053 ft/ft and a rip-rap class of 300. The channel is lined with rip rap for the expected max water surface elevation, the additional slope to catch to existing is proposed to be lined with Pyramat-75 to protect against sheet flow conditions. The channel discharges the runoff approximately 21 vertical feet above the bottom of the basin. The basin has a spillway separating the lower basin from the upper basin. A 4:1 USBR Type IX baffled spillway is proposed to control the flows down the slope into the upper tier of the basin. A concrete stilling basin is proposed at the toe of each spillway to minimize erosive forces at the bottom of the basin.

Together, both tiers of the basin combined have a proposed maximum storage capacity of 97 ac-ft with 1 feet of freeboard. Individually, the upper basin retains a volume of 62.9 ac-ft and the lower basin retains 83.5 ac-ft. Depths in the upper and lower tiers respectively are 7 feet and 8 feet. The basin has two spillways, one connecting the upper to the lower tier, and one as a discharge out of the proposed lower tier. The upper tier also is proposed to utilize four 18-inch culverts to attenuate and detain the flows prior to the peak. The 18-inch culverts are discharged into the lower tier. Both the upper and lower spillways have a width of 300 feet.

The estimated sediment loading for the basin is approximately 4.6 acre-feet. This volume is approximately 35% of the upper basin's volume, and 5.5% of the lower basin's volume.

Based on the preliminary design, approximately 722,552 cubic yards of cut and 4,076 cubic yards of fill are expected during construction, leaving a net export of 718,475 cubic, respectively. The total area of disturbance is approximately 41.6 acres.

Drainage easement or property acquisition will be required on APNs 015-342-01, 0154-341-16, 015-382-02, and 015-382-10.

## 7. Iron Mountain Basin

The Iron Mountain Basin is located within the NDOT Right-of-Way and extends into the adjacent private parcels. Two different sections of cut-off channels are proposed to intercept flows from the upstream distributary flow and fully convey the 100-year storm event to the proposed basin. The purpose of the basin serves the combined purpose of retention of a volume of water to prevent the downstream playa (Misfits Flat) from overflowing and flooding adjacent homes and decreasing peak flows inundating a section of Highway 50 and the downstream subdivision just south along Caroline Way.

The first channel segment is a road side ditch that has an eight foot wide bottom width with 2:1 side slopes, and a maximum depth of three feet. As the channel diverges from Iron Mountain Boulevard, it transitions to a 20-foot-wide bottom width with 2:1 side slopes, and a maximum depth of six feet. An average velocity of 3.3-fps is expected for a slope of 0.0022 ft/ft and a rip-rap class of 300. The channel is proposed to withstand the erosive forces of large amounts of runoff entering laterally, due to the channel being placed perpendicular to the upstream distributary flows, therefore acting as a cut-off ditch. This is accomplished by utilizing rip rap all the way to the top of slope, instead of only the max water surface elevation.

The basin is proposed to fully retain the 25-year event, and partially retain the 100-year storm event with a 270-foot concrete stabilized spillway proposed to allow the discharge of flows from larger storm events.

The basin has a proposed volume of 131.2 ac-ft with a maximum depth of 12 feet and 1 feet of freeboard.

The estimated sediment loading for the basin is approximately 0.24 acre-feet. This volume is approximately 0.2% of the overall volume of the basin.

Based on the preliminary design, approximately 539,055 cubic yards of cut and 2,519 cubic yards of fill are expected during construction, leaving a net export of 536,536 cubic, respectively. The total area of disturbance is approximately 24.4 acres.

Coordination and permitting with NDOT to utilize their parcel (APN 015-362-08) for stormwater storage will be required. Additionally, drainage easements or property acquisition will be required on APN 015-362-23.

## 8. Shawnee Basin

The Shawnee Basin intercepts flow from the wash just west of Iron Mountain Boulevard and north of Shawnee Road. The proposed basin captures flow from several sub-catchments while still channelized.

The Shawnee Basin, unlike the other basins, does not retain any flow, but attenuates the flow with restricted outlets only. There are two main entry points to the basin: the west and the north inlet spillways. Flows directed to other areas along the top of slope of the basin will be redirected to one of the two entry points via top-of-slope cut off ditches that will channelize the flows to the main entry spillways. 4:1 USBR Type IX baffled spillways are proposed to control the flows down the slope into the basin. A concrete stilling basin is proposed at the toe of each spillway to minimize erosive forces at the bottom of the basin.

The basin has a proposed volume of 114.0 ac-ft with a maximum depth of 12 feet with 3 feet of freeboard. The basin has one 100-foot-wide concrete outlet spillway. Additionally, the basin is equipped with two 24-inch outlet culverts. The culverts are placed at the bottom of the pond to allow for no retention volume. The existing downstream hydraulic conditions of Shawnee Basin is more channelized than the adjacent basins, and therefore requires less spillway length to mimic existing conditions.

The estimated sediment loading for the basin is approximately 8.85 acre-feet. This volume is approximately 7.8% of the overall volume of the basin.

Based on the preliminary design, approximately 1,685,453 cubic yards of cut and 907 cubic yards of fill are expected during construction, leaving a net export of 1,684,453 cubic yards, respectively. The total area of disturbance is approximately 38.6 acres.

Drainage easement or property acquisition will be required on APNs 015-351-02 and 015-354-05.

Table 1: Stormwater Basin Summary

Stormwater Basin	Storage w/ 1' freeboard (ac-ft)	Max Depth		Disturbed Area (ac)	Cut	Fill (cubic-yards)	Net
		25 YR (ft)	100 YR (ft)				
Stagecoach	52.8	5.3	12.9	15.0	387,233	10,519	376,714
Apache	93.9	7.9	13.3	27.5	834,125	463	833,662
Iron Mountain	131.2	5.5	17.0	24.4	539,056	2,519	536,537
QuarterHorse	210.3	7.7	15.8	39.6	1,495,903	878	1,495,025
Geurts Upper	62.9	7.0	7.9	41.6	722,552	4,077	718,476
Geurts Lower	83.5	10.0	10.5	41.6	722,552	4,077	718,476
Shawnee	114.0	7.9	13.2	38.6	1,685,453	908	1,684,545

**III. MINOR PROJECTS**

**1. Silver Lane Drainage Improvements**

The residents along Silver Lane have expressed concerns with the drainage issues along Silver Lane. Storm runoff is directed through the yards and along the road, generally draining easterly. However, the road lacks proper drainage infrastructure and therefore ponds water. During winter months, the ponded water freezes over and creates impassable situations. Many residents complained of a sand dune at the eastern edge of the road that prevents water from draining out of the corridor.

Grading to allow for positive drainage from Silver Lane towards the playa to the east will alleviate the ponding conditions and allow runoff to be conveyed away from the yards and roadway. Attachment C designates this project as a “channel”; however, this area correlates more with a grading project that allows for ponding mitigation.

Based on the preliminary design, approximately 225 cubic yards of cut and fill 8 cubic yards of fill are expected during construction, leaving a net export of 217 cubic yards, respectively. The total area of disturbance is approximately 0.32 acres.

Drainage and/or construction easements will be required on APN 015-311-17.

**2. Black Hawk Channel**

The Black Hawk Channel is proposed as a roadside ditch along Blackhawk Road. This channel is proposed to collect runoff from culverts beneath Highway 50 and discharge the upstream flows south of Turf Farm Road, on APN 015-371-07. This culvert collects the flows, channelizes them, and releases them where they were being released in the existing conditions.

The channel has a 6' base, 2:1 side slopes, and a max depth of 2.5 feet. 18-inch RCP driveway culverts have been placed where current driveways exist. 18-inch culverts were chosen for depth of cover reasons. The 18-inch culverts do not fully pass the 25 or 100-year storm events due to imposing a decrease in efficiency; however, in general, the channel combined with the culverts will improve runoff conditions during storm events. An average velocity of 2.4-fps is expected for a longitudinal slope of .22% and a rip-rap class of 150.

Maintenance is a concern for the roadside ditch and culverts. Regular flushing of the culverts and removing sediment from the channel will be required to maintain the hydraulic efficiency.

Based on the preliminary design, approximately 1,303 cubic yards of cut and 0 cubic yards of fill are expected during construction, leaving a net export of 1,303 cubic yards, respectively. The total area of disturbance is approximately 0.50 acres.

No Right-of-Way acquisition is anticipated for this project.

### 3. Pinto/Rancho Channels

Similarly to the Silver Lane drainage improvements, there is minimal roadside ditches to convey runoff out of the subdivision leading to ponding conditions.

Two roadside channels are proposed: One that parallels Pinto Street on the North side, and one that parallels Rancho Avenue on the west side. The channels combine at the intersection of Rancho Avenue and Pinto Street. The channel terminates just south of Tahoe Avenue allow for flows to dissipate into the undeveloped area.

The channels both have a section of four-foot-wide bottom width, 2:1 side slopes, and a maximum depth of 2.5 feet. The channels are proposed to convey the 100-year storm event. The channels have a maximum longitudinal slope of 1.5%, and therefore have a maximum expected velocity of 6.0 fps. 18-inch RCP driveway culverts have been placed where current driveways exist. The 18-inch culverts do not fully pass the 25 or 100-year storm events due to imposing a decrease in efficiency; however, in general, the channel combined with the culverts will improve runoff conditions during storm events.

Maintenance is a concern for the roadside ditch and culverts. Regular flushing of the culverts and removing sediment from the channel will be required to maintain the hydraulic efficiency.

Based on the preliminary design, approximately 8,318 cubic yards of cut and 0 cubic yards of fill are expected during construction, leaving a net export of 8,318 cubic yards, respectively. The total area of disturbance is approximately 2.5 acres.

No Right-of-Way acquisition is anticipated for this project.

#### **IV. PRELIMINARY OPINION OF PROBABLE CONSTRUCTION COSTS**

Preliminary Opinion of Probable Construction Costs (OPCC) were prepared for each of the proposed drainage facilities. In addition to OPCC, non-construction costs were also considered which include additional environmental and BLM permitting, design, and construction services. OPCC estimates for each alternative are provided in Appendix A. Grant funding and loan interest calculations were not included as part of this analysis. However, as discussed in the Introduction, each project can be constructed independently of the others to allow for phasing of the project as funds become available. A detailed OPCC for each alternative can be found in Attachment A. Right-of-way and/or easement acquisition costs are not included in these estimates and will need to be factored into the overall project costs as each project moves forward.

#### **V. ANNUAL OPERATIONS AND MAINTENANCE COSTS**

It is anticipated that the basins and channels will require routine maintenance including debris removal, repairs to the lining and slopes, and revegetation after work is complete. Maintenance costs were annualized from the future value using a discounted rate from Circular A94 Appendix C from the Office of Management and Budget for an assumed maintenance schedule of once every 10 years for a period of 20 years. Maintenance costs were derived from the annualized equipment and labor costs required to remove the 25 or 100-year sediment volume and replace the basin line. Table 2 shows the estimated yearly maintenance costs.

Table 2: Annual Operations and Maintenance Costs

Expense Type	Maintenance/ Riprap Repairs <sup>1,2</sup>	Debris Removal/ Disposal <sup>1,2</sup>	Revegetate <sup>3</sup>	Total
Iron Mountain Basin	\$ 43,800	\$ 403,200	\$ 73,900	\$ <b>520,900</b>
Shawnee Basin	\$ 43,800	\$ 403,200	\$ 116,200	\$ <b>563,200</b>
Geurts Basin	\$ 43,800	\$ 403,200	\$ 125,900	\$ <b>572,900</b>
Quarterhorse Basin	\$ 43,800	\$ 403,200	\$ 119,800	\$ <b>566,800</b>
Apache Basin	\$ 43,800	\$ 403,200	\$ 84,400	\$ <b>531,400</b>
Stagecoach Basin	\$ 43,800	\$ 403,200	\$ 45,400	\$ <b>492,400</b>
Empey Channel	\$ 4,700	\$ 60,000	\$ -	\$ <b>64,700</b>
Black Hawk Channel	\$ 1,600	\$ 43,200	\$ -	\$ <b>44,800</b>
Silver Lane Channel	\$ 1,600	\$ 43,200	\$ -	\$ <b>44,800</b>
Rancho/Pinto Channel	\$ 1,600	\$ 43,200	\$ -	\$ <b>44,800</b>
<b>Total</b>	\$ <b>272,300</b>	\$ <b>2,608,800</b>	\$ <b>565,600</b>	\$ <b>3,446,700</b>

<sup>1</sup> Assume maintenance and debris removal in each pond and channel every year.

<sup>2</sup> Costs based on 2023 FEMA Schedule of Equipment Rates.

<sup>3</sup> Average cost to reseed each basin based on \$2.50/SY for entire basin area with 20% markup. Assumes 25% of basin to be reseeded each year

**VI. LIFECYCLE COST ANALYSIS**

A life cycle present worth analysis was prepared for a 20-year period considering the estimated total project costs and the annual O&M costs. Salvaged and valuable materials such as soils excavated during construction and during routine maintenance were not considered in the analysis. The life cycle cost analysis and net present value (NPV) for each alternative are presented in Table 3.

Table 3: Life Cycle Costs

Alternative	Present Value		
	Total Project Cost <sup>1</sup>	O&M <sup>2</sup> (for 20 Yrs)	20-Yr NPV <sup>4</sup>
Iron Mountain Basin	\$ 16,713,000	\$ 8,943,100	\$ 25,656,100
Shawnee Basin	\$ 51,500,500	\$ 9,669,400	\$ 61,169,900
Geurts Basin	\$ 28,234,800	\$ 9,835,900	\$ 38,070,700
Quarterhorse Basin	\$ 45,108,800	\$ 9,731,200	\$ 54,840,000
Apache Basin	\$ 28,753,500	\$ 9,123,400	\$ 37,876,900
Stagecoach Basin	\$ 30,178,300	\$ 8,453,800	\$ 38,632,100
Empey Channel	\$ 4,206,700	\$ 1,110,800	\$ 5,317,500
Black Hawk Channel	\$ 409,400	\$ 769,200	\$ 1,178,600
Silver Lane Channel	\$ 208,200	\$ 769,200	\$ 977,400
Rancho/Pinto Channel	\$ 2,108,000	\$ 769,200	\$ 2,877,200

<sup>1</sup> Total project cost includes construction and non-construction costs.

<sup>2</sup> Considers 20-year real discount rate of 1.5 percent per Circular A94 Appendix C from the Office of Management and Budget, revised November 2015 ([https://www.whitehouse.gov/omb/circulars\\_a094/a94\\_appx-c](https://www.whitehouse.gov/omb/circulars_a094/a94_appx-c)).

<sup>3</sup> Salvage Values are based on useful material (sand) excavated collected within the detention ponds.

<sup>4</sup> Total 20-YR NPV = Total Project Cost + O&M (uniform series present worth) – Salvage (single payment present worth).

All improvements shown within the 15% design phase plan set are preliminary with the quantities obtained from the designs used to provide the OPCC for the project. Estimated quantities for bid items for each project will be adjusted based on final design of each project.

## **VII. REFERENCES**

- ❑ Drainage Guidelines for Lyon County, Dated September, 2018.
- ❑ Truckee Meadows Regional Drainage Manual (TMRDM), Dated April, 2009
- ❑ U.S. Department of Agriculture, Soil Conservation Service, Engineering Division, *Urban Hydrology for Small Watersheds*, Technical Release 55, Second Edition June 1986
- ❑ United States Department of the Interior, Bureau of Reclamation, Engineering and Research Center, *A Baffled Apron As a Spillway Energy Dissipator*, PAP-0340
- ❑ 2023 FEMA Schedule of Equipment Rates
- ❑ Circular A94 Appendix C from the Office of Management and Budget, revised November 2018.

# Appendix A

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*Opinion of Probable Construction Costs*

**Lyon County  
Iron Mountain Basin and Spillway  
June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	\$ 450,000.00	\$ 450,000
2	Clearing and Grubbing	AC	24.4	\$ 7,100.00	\$ 173,240
3	Cut/Fill Onsite	CY	539,055	\$ 7.00	\$ 3,773,385
4	Cut Offhaul	CY	536,536	\$ 7.00	\$ 3,755,752
5	Pyramat-75	SF	300,963	\$ 3.00	\$ 902,889
6	Class 300 Rip Rap w/ Geotextile	CY	5,779	\$ 150.00	\$ 866,850
7	Wire Fencing/ W Gates	LF	3,877	\$ 60.00	\$ 232,620
8	Concrete Stillling Basin	SF	2,400	\$ 30.00	\$ 72,000
9	Concrete Access Road	SF	503	\$ 30.00	\$ 15,090
10	Agg. Base Access Road	SF	44,607	\$ 4.00	\$ 178,428
11	Outlet Concrete Spillway	SF	4,097	\$ 35.00	\$ 143,395
12	USBR Type IX Concrete Spillway	SF	5,677	\$ 80.00	\$ 454,177
13	Rock Chute	CY	448	\$ 100.00	\$ 44,837
14	Sediment Gauge	EA	1	\$ 5,000.00	\$ 5,000
15	Revegetation	AC	24.4	\$ 2,500.00	\$ 61,000
16	Temporary Erosion Control	LS	1	\$ 50,000.00	\$ 50,000
17	Contingency, 30%	LS	1	\$ -	\$ 3,353,599
				<b>Subtotal Construction Costs</b>	\$ 14,533,000
<b>Non-Construction Costs</b>					
18	Permitting, Engineering, Construction Management, Administration, Testing, and Inspection, 15%			\$	\$ 2,180,000
				<b>Subtotal Non-Construction Costs</b>	\$ 2,180,000
<b>Total Project Costs</b>					\$ 16,713,000

**Lyon County  
Empey Channel  
June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	100,000.00	\$ 100,000
2	Clearing and Grubbing	AC	3.7	7,100.00	\$ 25,986
3	Cut/Fill Onsite	CY	22,170	7.00	\$ 155,190
4	Cut Offhaul	CY	22,170	7.00	\$ 155,190
5	24" Storm Drain Pipe	LF	2,000	250.00	\$ 500,000
6	Class 300 Rip Rap w/ Geotextile	CY	11,586	150.00	\$ 1,737,900
7	Revegetation	AC	3.7	2,500.00	\$ 9,150
8	Headwall	EA	2	10,000.00	\$ 20,000
9	Temporary Erosion Control	LS	1	30,000.00	\$ 30,000
10	HWY 50 AC Patch	LS	1	80,000.00	\$ 80,000
11	Contingency, 30%	LS	1	-	\$ 844,025
				<b>Subtotal Construction Costs</b>	\$ 3,658,000
<b>Non-Construction Costs</b>					
12	Permitting, Engineering, Construction Management, Administration, Testing, and Inspection, 15%				\$ 548,700
				<b>Subtotal Non-Construction Costs</b>	\$ 548,700
<b>Total Project Costs</b>					<b>\$ 4,206,700</b>

**Lyon County  
Shawnee Basin  
June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	\$ 1,000,000.00	\$ 1,000,000
2	Clearing and Grubbing	AC	38.4	\$ 7,100.00	\$ 272,640
3	Cut/Fill Onsite	CY	1,685,453	\$ 7.00	\$ 11,798,168
4	Cut Offhaul	CY	1,684,545	\$ 7.00	\$ 11,791,815
5	Pyramat-75	SF	674,346	\$ 3.00	\$ 2,023,000
6	Class 300 Rip Rap w/ Geotextile	CY	3,097	\$ 150.00	\$ 464,600
7	Wire Fencing/ W Gates	LF	5,066	\$ 60.00	\$ 303,960
8	Concrete Stilling Basin	SF	11,096	\$ 30.00	\$ 332,880
9	Concrete Access Road	SF	4,164	\$ 30.00	\$ 124,920
10	Agg. Base Access Road	SF	54,132	\$ 4.00	\$ 216,500
11	Outlet Concrete Spillway	SF	10,234	\$ 35.00	\$ 358,190
12	USBR Type IX Concrete Spillway	SF	67,545	\$ 80.00	\$ 5,403,600
13	Rock Chute	CY	1,389	\$ 100.00	\$ 138,852
14	Sediment Gauge	EA	1	\$ 5,000.00	\$ 5,000
15	24" Storm Drain Pipe	LF	272	\$ 250.00	\$ 68,000
16	Revegetation	AC	38.4	\$ 2,500.00	\$ 96,000
17	Temporary Erosion Control	LS	1	\$ 50,000.00	\$ 50,000
18	Contingency, 30%	LS	1	\$ -	\$ 10,334,437
				<b>Subtotal Construction Costs</b>	\$ 44,783,000
<b>Non-Construction Costs</b>					
19	Permitting, Engineering, Construction Management, Administration, Testing, and Inspection, 15%			\$	\$ 6,717,500
				<b>Subtotal Non-Construction Costs</b>	\$ 6,717,500
<b>Total Project Costs</b>					\$ 51,500,500

**Lyon County  
Blackhawk Channel  
June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	\$ 20,000.00	\$ 20,000
2	Clearing and Grubbing	AC	0.5	\$ 7,100.00	\$ 3,550
3	Cut/Fill Onsite	CY	1,185	\$ 7.00	\$ 8,295
4	Cut Offhaul	CY	1,184	\$ 7.00	\$ 8,288
5	Class 150 Rip Rap w/ Geotextile	CY	1,436	\$ 120.00	\$ 172,300
6	18" RCP Culvert	LF	129	\$ 200.00	\$ 25,800
7	Headwall	EA	6	\$ 5,000.00	\$ 30,000
8	Revegetation	AC	0.5	\$ -	\$ -
9	Temporary Erosion Control	LS	1	\$ 5,000.00	\$ 5,000
10	Contingency, 30%	LS	1	\$ -	\$ 81,970
				<b>Subtotal Construction Costs</b>	<b>\$ 356,000</b>
<b>Non-Construction Costs</b>					
11	Permitting, Engineering, Construction Management, Administration, Testing, and Inspection, 15%			\$	\$ 53,400
				<b>Subtotal Non-Construction Costs</b>	<b>\$ 53,400</b>
<b>Total Project Costs</b>					<b>\$ 409,400</b>

**Lyon County  
Silverlane Channel  
June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	10,000.00	\$ 10,000
2	Clearing and Grubbing	AC	0.3	7,100.00	\$ 2,272
3	Cut/Fill Onsite	CY	228	7.00	\$ 1,597
4	Cut Offhaul	CY	222	7.00	\$ 1,556
5	Class 150 Rip Rap w/ Geotextile	CY	981	120.00	\$ 117,720
6	Temporary Erosion Control	LS	1	5,000.00	\$ 5,000
7	Revegetation	AC	0.3	2,500.00	\$ 800
8	Contingency, 30%	LS	1	-	\$ 41,683
				<b>Subtotal Construction Costs</b>	<b>\$ 181,000</b>
<b>Non-Construction Costs</b>					
8	Permitting, Engineering, Construction Management, Administration, Testing, and Inspection, 15%			\$	27,200
				<b>Subtotal Non-Construction Costs</b>	<b>\$ 27,200</b>
<b>Total Project Costs</b>					<b>\$ 208,200</b>

**Lyon County  
Pinto/Rancho Channel  
June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	\$ 100,000.00	\$ 100,000
2	Clearing and Grubbing	AC	2.1	\$ 7,100.00	\$ 14,910
3	Cut/Fill Onsite	CY	8,318	\$ 7.00	\$ 58,226
4	Cut Offhaul	CY	8,318	\$ 7.00	\$ 58,226
5	Class 150 Rip Rap w/ Geotextile	CY	8,043	\$ 120.00	\$ 965,160
6	18" RCP Culvert	LF	1,036	\$ 200.00	\$ 207,200
7	Revegetation	AC	2.1	\$ 2,500.00	\$ 5,250
8	Temporary Erosion Control	LS	1	\$ 1,000.00	\$ 1,000
9	Contingency, 30%	LS	1	\$ -	\$ 422,992
				<b>Subtotal Construction Costs</b>	<b>\$ 1,833,000</b>
<b>Non-Construction Costs</b>					
10	Permitting, Engineering, Construction Management, Administration, Testing, and Inspection, 15%			\$	275,000
				<b>Subtotal Non-Construction Costs</b>	<b>\$ 275,000</b>
<b>Total Project Costs</b>					<b>\$ 2,108,000</b>

**Lyon County**  
**Geurts Basin-Channel**  
**June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	\$ 850,000.00	\$ 850,000
2	Clearing and Grubbing	AC	41.6	\$ 7,100.00	\$ 295,360
3	Cut/Fill Onsite	CY	722,552	\$ 7.00	\$ 5,057,864
4	Cut Offhaul	CY	718,475	\$ 7.00	\$ 5,029,325
5	Pyramat-75	SF	456,427	\$ 3.00	\$ 1,369,281
6	Class 300 Rip Rap w/ Geotextile	CY	19,580	\$ 150.00	\$ 2,937,000
7	Wire Fencing/ W Gates	LF	4,892	\$ 60.00	\$ 293,520
8	Concrete Stillling Basin	SF	900	\$ 30.00	\$ 27,000
9	Concrete Access Road	SF	364	\$ 30.00	\$ 10,920
9	Agg. Base Access Road	SF	84,675	\$ 4.00	\$ 338,700
10	Outlet Concrete Spillway	SF	44,017	\$ 35.00	\$ 1,540,595
11	USBR Type IX Concrete Spillway	SF	9,526	\$ 80.00	\$ 762,080
12	Rock Chute	CY	191	\$ 100.00	\$ 19,059
13	Sediment Gauge	EA	1	\$ 5,000.00	\$ 5,000
14	36" Storm Drain Pipe	LF	560	\$ 350.00	\$ 196,000
15	Revegetation	AC	41.6	\$ 2,500.00	\$ 104,000
16	Temporary Erosion Control	LS	1	\$ 50,000.00	\$ 50,000
17	Contingency, 30%	LS	1	\$ -	\$ 5,665,711
				<b>Subtotal Construction Costs</b>	\$ 24,552,000
<b>Non-Construction Costs</b>					
18	Permitting, Engineering, Construction Management, Administration, Testing, and Inspection, 15%			\$	\$ 3,682,800
				<b>Subtotal Non-Construction Costs</b>	\$ 3,682,800
<b>Total Project Costs</b>					\$ 28,234,800

**Lyon County  
Quarterhorse Basin  
June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	\$ 1,300,000.00	\$ 1,300,000
2	Clearing and Grubbing	AC	39.6	\$ 7,100.00	\$ 281,160
3	Cut/Fill Onsite	CY	1,495,903	\$ 7.00	\$ 10,471,321
4	Cut Offhaul	CY	1,495,025	\$ 7.00	\$ 10,465,175
5	Pyramat-75	SF	597,038	\$ 3.00	\$ 1,791,114
6	Class 300 Rip Rap w/ Geotextile	CY	4,338	\$ 150.00	\$ 650,700
7	Wire Fencing/ W Gates	LF	5,820	\$ 60.00	\$ 349,200
8	Concrete Stilling Basin	SF	6,377	\$ 30.00	\$ 191,300
9	Agg. Base Access Road	SF	86,402	\$ 4.00	\$ 345,600
10	Concrete Access Road	SF	5,849	\$ 35.00	\$ 204,700
11	Outlet Concrete Spillway	SF	16,730	\$ 35.00	\$ 585,600
12	USBR Type IX Concrete Spillway	SF	40,819	\$ 80.00	\$ 3,265,500
13	Rock Chute	CY	624	\$ 100.00	\$ 62,378
14	Sediment Gauge	EA	1	\$ 5,000.00	\$ 5,000
15	18" Storm Drain Pipe	LF	276	\$ 200.00	\$ 55,200
16	Revegetation	AC	39.6	\$ 2,500.00	\$ 99,000
17	Temporary Erosion Control	LS	1	\$ 50,000.00	\$ 50,000
18	Contingency, 30%	LS	1	\$ -	\$ 9,051,884
				<b>Subtotal Construction Costs</b>	\$ 39,225,000
<b>Non-Construction Costs</b>					
19	Permitting, Engineering, Construction Management, Administration, Testing, and Inspection, 15%			\$	\$ 5,883,800
				<b>Subtotal Non-Construction Costs</b>	\$ 5,883,800
<b>Total Project Costs</b>					\$ 45,108,800

**Lyon County**  
**Apache Basin and Channel**  
**June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	\$ 850,000.00	\$ 850,000
2	Clearing and Grubbing	AC	27.9	\$ 7,100.00	\$ 198,090
3	Cut/Fill Onsite	CY	834,125	\$ 7.00	\$ 5,838,875
4	Cut Offhaul	CY	833,662	\$ 7.00	\$ 5,835,634
5	Pyramat-75	SF	444,380	\$ 3.00	\$ 1,333,140
6	Class 300 Rip Rap w/ Geotextile	CY	2,077	\$ 150.00	\$ 311,550
7	Class 900 Rip Rap w/ Geotextile	CY	3,253	\$ 500.00	\$ 1,626,500
8	Wire Fencing/ W Gates	LF	5,808	\$ 60.00	\$ 348,480
9	Concrete Stillling Basin	SF	1,866	\$ 30.00	\$ 55,980
10	Agg. Base Access Road	SF	85,898	\$ 4.00	\$ 343,592
11	Outlet Concrete Spillway	SF	35,363	\$ 35.00	\$ 1,237,705
12	USBR Type IX Concrete Spillway	SF	12,469	\$ 80.00	\$ 997,520
13	Rock Chute	CY	624	\$ 100.00	\$ 62,378
14	Sediment Gauge	EA	1	\$ 5,000.00	\$ 5,000
15	18" Storm Drain Pipe	LF	344	\$ 200.00	\$ 68,800
16	Revegetation	AC	27.9	\$ 2,500.00	\$ 69,750
17	Temporary Erosion Control	LS	1	\$ 50,000.00	\$ 50,000
18	Contingency, 30%	LS	1	\$ -	\$ 5,769,898
				<b>Subtotal Construction Costs</b>	\$ 25,003,000
<b>Non-Construction Costs</b>					
19	Permitting, Engineering, Construction Management, Administration, Testing, and Inspection, 15%			\$	\$ 3,750,500
				<b>Subtotal Non-Construction Costs</b>	\$ 3,750,500
<b>Total Project Costs</b>					\$ 28,753,500

**Lyon County**  
**Stagecoach Basin and Channel**  
**June, 2024**

<b>Item</b>	<b>Description</b>	<b>Unit</b>	<b>Quantity</b>	<b>Unit Price</b>	<b>Total Price</b>
<b>Construction Costs</b>					
1	Mobilization and Demobilization	LS	1	\$ 850,000.00	\$ 850,000
2	Clearing and Grubbing	AC	15.0	\$ 7,100.00	\$ 106,500
3	Cut/Fill Onsite	CY	387,232	\$ 7.00	\$ 2,710,624
4	Cut Offhaul	CY	376,714	\$ 7.00	\$ 2,636,998
5	Pyramat-75	SF	307,026	\$ 3.00	\$ 921,078
6	Class 300 Rip Rap w/ Geotextile	CY	8,307	\$ 150.00	\$ 16,821,150
7	Wire Fencing/ W Gates	LF	2,959	\$ 60.00	\$ 177,540
8	Concrete Stilling Basin	SF	1,768	\$ 30.00	\$ 53,040
9	Concrete Access Road	SF	583	\$ 35.00	\$ 20,400
10	Agg. Base Access Road	SF	32,559	\$ 4.00	\$ 130,236
11	Outlet Concrete Spillway	SF	2,163	\$ 35.00	\$ 75,705
12	USBR Type IX Concrete Spillway	SF	14,208	\$ 80.00	\$ 1,136,640
13	Rock Chute	CY	5,090	\$ 100.00	\$ 509,000
14	Sediment Gauge	EA	1	\$ 5,000.00	\$ 5,000
15	Revegetation	AC	15.0	\$ 2,500.00	\$ 37,500
16	Temporary Erosion Control	LS	1	\$ 50,000.00	\$ 50,000
17	Contingency, 30%	LS	1	\$ -	\$ -
	<b>Subtotal Construction Costs</b>			\$ 26,242,000	\$ 26,242,000
<b>Non-Construction Costs</b>					
18	Permitting, Engineering, Construction Management, Administration, Testing, and			\$	\$ 3,936,300
	<b>Subtotal Non-Construction Costs</b>			\$	\$ 3,936,300
<b>Total Project Costs</b>				\$	\$ 30,178,300

# **Attachment B**

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*Preliminary Hydraulic Calculations*

# Hydraulic Analysis Report

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## Project Data

Project Title: Stagecoach ADMP

Designer:

Project Date: Sunday, April 28, 2024

Project Units: U.S. Customary Units

Notes:

## Channel Analysis: Stagecoach Channel

Notes:

### Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width 20.00 ft

Longitudinal Slope: 0.0030 ft/ft

Manning's n: 0.0666

Flow 240.0000 cfs

### Result Parameters

Depth 3.6614 ft

Area of Flow 100.0402 ft<sup>2</sup>

Wetted Perimeter 36.3743 ft

Hydraulic Radius 2.7503 ft

Average Velocity 2.3990 ft/s

Top Width 34.6457 ft

Froude Number: 0.2488

Critical Depth 1.5603 ft

Critical Velocity 6.6528 ft/s

Critical Slope: 0.0603 ft/ft

Critical Top Width 26.24 ft

Calculated Max Shear Stress 0.6854 lb/ft<sup>2</sup>

Calculated Avg Shear Stress 0.5149 lb/ft<sup>2</sup>

## **Channel Analysis: Empey Channel**

Notes:

### **Input Parameters**

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width 30.00 ft

Longitudinal Slope: 0.0100 ft/ft

Manning's n: 0.0560

Flow 1600.0000 cfs

### **Result Parameters**

Depth 5.6162 ft

Area of Flow 231.5683 ft<sup>2</sup>

Wetted Perimeter 55.1163 ft

Hydraulic Radius 4.2014 ft

Average Velocity 6.9094 ft/s

Top Width 52.4647 ft

Froude Number: 0.5796

Critical Depth 4.0498 ft

Critical Velocity 10.3697 ft/s

Critical Slope: 0.0323 ft/ft

Critical Top Width 46.20 ft

Calculated Max Shear Stress 3.5045 lb/ft<sup>2</sup>

Calculated Avg Shear Stress 2.6217 lb/ft<sup>2</sup>

## **Channel Analysis: Apache Channel**

Notes:

### **Input Parameters**

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width 20.00 ft

Longitudinal Slope: 0.0350 ft/ft

Manning's n: 0.0792

Flow 836.0000 cfs

### **Result Parameters**

Depth 4.0697 ft

Area of Flow 114.5192 ft<sup>2</sup>

Wetted Perimeter 38.2003 ft

Hydraulic Radius 2.9979 ft

Average Velocity 7.3001 ft/s

Top Width 36.2788 ft

Froude Number: 0.7241

Critical Depth 3.3632 ft

Critical Velocity 9.3008 ft/s

Critical Slope: 0.0699 ft/ft

Critical Top Width 33.45 ft

Calculated Max Shear Stress 8.8882 lb/ft<sup>2</sup>

Calculated Avg Shear Stress 6.5473 lb/ft<sup>2</sup>

## Channel Analysis: Geurts Channel

Notes:

### Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width 30.00 ft

Longitudinal Slope: 0.0275 ft/ft

Manning's n: 0.0609

Flow 1932.0000 cfs

### Result Parameters

Depth 4.9359 ft

Area of Flow 196.8014 ft<sup>2</sup>

Wetted Perimeter 52.0739 ft

Hydraulic Radius 3.7793 ft

Average Velocity 9.8170 ft/s

Top Width 49.7435 ft

Froude Number: 0.8698

Critical Depth 4.5395 ft

Critical Velocity 10.8908 ft/s

Critical Slope: 0.0371 ft/ft

Critical Top Width 48.16 ft

Calculated Max Shear Stress 8.4699 lb/ft<sup>2</sup>

Calculated Avg Shear Stress 6.4852 lb/ft<sup>2</sup>

## Channel Analysis: Iron Mountain Channel

Notes:

### Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width 20.00 ft

Longitudinal Slope: 0.0022 ft/ft

Manning's n: 0.0584

Flow 700.0000 cfs

### Result Parameters

Depth 6.5777 ft

Area of Flow 218.0882 ft<sup>2</sup>

Wetted Perimeter 49.4166 ft

Hydraulic Radius 4.4133 ft

Average Velocity 3.2097 ft/s

Top Width 46.3110 ft

Froude Number: 0.2607

Critical Depth 3.0235 ft

Critical Velocity 8.8885 ft/s

Critical Slope: 0.0391 ft/ft

Critical Top Width 32.09 ft

Calculated Max Shear Stress 0.9030 lb/ft<sup>2</sup>

Calculated Avg Shear Stress 0.6059 lb/ft<sup>2</sup>

## Channel Analysis: Black Hawk Channel

Notes:

### **Input Parameters**

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width 6.00 ft

Longitudinal Slope: 0.0022 ft/ft

Manning's n: 0.0350

Depth 2.0000 ft

### **Result Parameters**

Flow 48.3685 cfs

Area of Flow 20.0000 ft<sup>2</sup>

Wetted Perimeter 14.9443 ft

Hydraulic Radius 1.3383 ft

Average Velocity 2.4184 ft/s

Top Width 14.0000 ft

Froude Number: 0.3566

Critical Depth 1.1096 ft

Critical Velocity 5.3038 ft/s

Critical Slope: 0.0199 ft/ft

Critical Top Width 10.44 ft

Calculated Max Shear Stress 0.2746 lb/ft<sup>2</sup>

Calculated Avg Shear Stress 0.1837 lb/ft<sup>2</sup>

### **Channel Analysis: PintoRancho Channel**

Notes:

#### **Input Parameters**

Channel Type: Trapezoidal

Side Slope 1 (Z1): 2.0000 ft/ft

Side Slope 2 (Z2): 2.0000 ft/ft

Channel Width 4.00 ft

Longitudinal Slope: 0.0152 ft/ft

Manning's n: 0.0350

Depth 2.0000 ft

### **Result Parameters**

Flow 96.4615 cfs

Area of Flow 16.0000 ft<sup>2</sup>

Wetted Perimeter 12.9443 ft

Hydraulic Radius 1.2361 ft

Average Velocity 6.0288 ft/s

Top Width 12.0000 ft

Froude Number: 0.9201

Critical Depth 1.9147 ft

Critical Velocity 6.4345 ft/s

Critical Slope: 0.0181 ft/ft

Critical Top Width 11.66 ft

Calculated Max Shear Stress 1.8970 lb/ft<sup>2</sup>

Calculated Avg Shear Stress 1.1724 lb/ft<sup>2</sup>

### **Channel Lining Analysis: Apache Channel Lining**

Notes:

#### **Lining Input Parameters**

Channel Lining Type: Riprap, Cobble, or Gravel

D50: 900.00 mm

Riprap Specific Weight: 165 lb/ft<sup>3</sup>

Water Specific Weight: 62.4 lb/ft<sup>3</sup>

Riprap Shape is Angular

Safety Factor: 1

Calculated Safety Factor: 1.50016

### Lining Results

Angle of Repose: 42.1 degrees

Relative Flow Depth: 1.06905 ft

Manning's n method: Bathurst

Manning's n: 0.0791781

### Channel Bottom Shear Results

V\*: 2.14163

Reynold's Number: 519614

Shield's Parameter: 0.15

Shear stress on channel bottom: 8.88824 lb/ft<sup>2</sup>

Permissible shear stress for channel bottom: 45.4429 lb/ft<sup>2</sup>

Channel bottom is stable

Stable D50: 264.076 mm

### Channel Side Shear Results

K1: 0.802

K2: 0.745006

Kb: 0

Shear stress on side of channel: 8.88824 lb/ft<sup>2</sup>

Permissible shear stress for side of channel: 33.8552 lb/ft<sup>2</sup>

Stable Side D50: 0.93267 lb/ft<sup>2</sup>

Side of channel is stable

### Channel Lining Stability Results 2

The channel is stable

## Channel Summary

Name of Selected Channel: Apache Channel

## Channel Lining Analysis: Stagecoach Channel Lining

Notes:

### Lining Input Parameters

Channel Lining Type: Riprap, Cobble, or Gravel

D50: 300.00 mm

Riprap Specific Weight: 165 lb/ft<sup>3</sup>

Water Specific Weight: 62.4 lb/ft<sup>3</sup>

Riprap Shape is Angular

Safety Factor: 1

Calculated Safety Factor: 1.02546

### Lining Results

Angle of Repose: 41.7 degrees

Relative Flow Depth: 2.93373 ft

Manning's n method: Blodgett

Manning's n: 0.0665973

### Channel Bottom Shear Results

V\*: 0.594721

Reynold's Number: 48098.2

Shield's Parameter: 0.0522132

Shear stress on channel bottom: 0.685417 lb/ft<sup>2</sup>

Permissible shear stress for channel bottom: 5.27271 lb/ft<sup>2</sup>

Channel bottom is stable

Stable D50: 39.9909 mm

Channel Side Shear Results

K1: 0.802

K2: 0.740307

Kb: 0

Shear stress on side of channel: 0.685417 lb/ft<sup>2</sup>

Permissible shear stress for side of channel: 3.90343 lb/ft<sup>2</sup>

Stable Side D50: 0.142138 lb/ft<sup>2</sup>

Side of channel is stable

Channel Lining Stability Results 2

The channel is stable

### Channel Summary

Name of Selected Channel: Stagecoach Channel

### Channel Lining Analysis: Geurts Channel Lining

Notes:

#### Lining Input Parameters

Channel Lining Type: Riprap, Cobble, or Gravel

D50: 300.00 mm

Riprap Specific Weight: 165 lb/ft<sup>3</sup>

Water Specific Weight: 62.4 lb/ft<sup>3</sup>

Riprap Shape is Angular

Safety Factor: 1

Calculated Safety Factor: 1.40354

#### Lining Results

Angle of Repose: 41.7 degrees

Relative Flow Depth: 4.01975 ft

Manning's n method: Blodgett

Manning's n: 0.0609057

#### Channel Bottom Shear Results

V\*: 2.09066

Reynold's Number: 169083

Shield's Parameter: 0.130097

Shear stress on channel bottom: 8.47024 lb/ft<sup>2</sup>

Permissible shear stress for channel bottom: 13.1377 lb/ft<sup>2</sup>

Channel bottom is stable

Stable D50: 271.469 mm

Channel Side Shear Results

K1: 0.802

K2: 0.740307

Kb: 0

Shear stress on side of channel: 8.47024 lb/ft<sup>2</sup>

Permissible shear stress for side of channel: 9.72597 lb/ft<sup>2</sup>

Stable Side D50: 0.964868 lb/ft<sup>2</sup>

Side of channel is stable

Channel Lining Stability Results 2

The channel is stable

### Channel Summary

Name of Selected Channel: Geurts Channel

## Channel Lining Analysis: Iron Mountain Channel Lining

Notes:

### Lining Input Parameters

Channel Lining Type: Riprap, Cobble, or Gravel

D50: 300.00 mm

Riprap Specific Weight: 165 lb/ft<sup>3</sup>

Water Specific Weight: 62.4 lb/ft<sup>3</sup>

Riprap Shape is Angular

Safety Factor: 1

Calculated Safety Factor: 1.04768

### Lining Results

Angle of Repose: 41.7 degrees

Relative Flow Depth: 4.78456 ft

Manning's n method: Blodgett

Manning's n: 0.0584271

### Channel Bottom Shear Results

V\*: 0.682618

Reynold's Number: 55206.9

Shield's Parameter: 0.0567894

Shear stress on channel bottom: 0.902992 lb/ft<sup>2</sup>

Permissible shear stress for channel bottom: 5.73484 lb/ft<sup>2</sup>

Channel bottom is stable

Stable D50: 49.4893 mm

### Channel Side Shear Results

K1: 0.802

K2: 0.740307

Kb: 0

Shear stress on side of channel: 0.902992 lb/ft<sup>2</sup>

Permissible shear stress for side of channel: 4.24554 lb/ft<sup>2</sup>

Stable Side D50: 0.175897 lb/ft<sup>2</sup>

Side of channel is stable

### Channel Lining Stability Results 2

The channel is stable

### Channel Summary

Name of Selected Channel: Iron Mountain Channel

**Project:** Detention Hydrographs

**Simulation Run:** 25 Year Run

**Simulation Start:** 31 December 2022, 24:00

**Simulation End:** 2 January 2023, 24:00

**HMS Version:** 4.12

**Executed:** 11 June 2024, 18:19

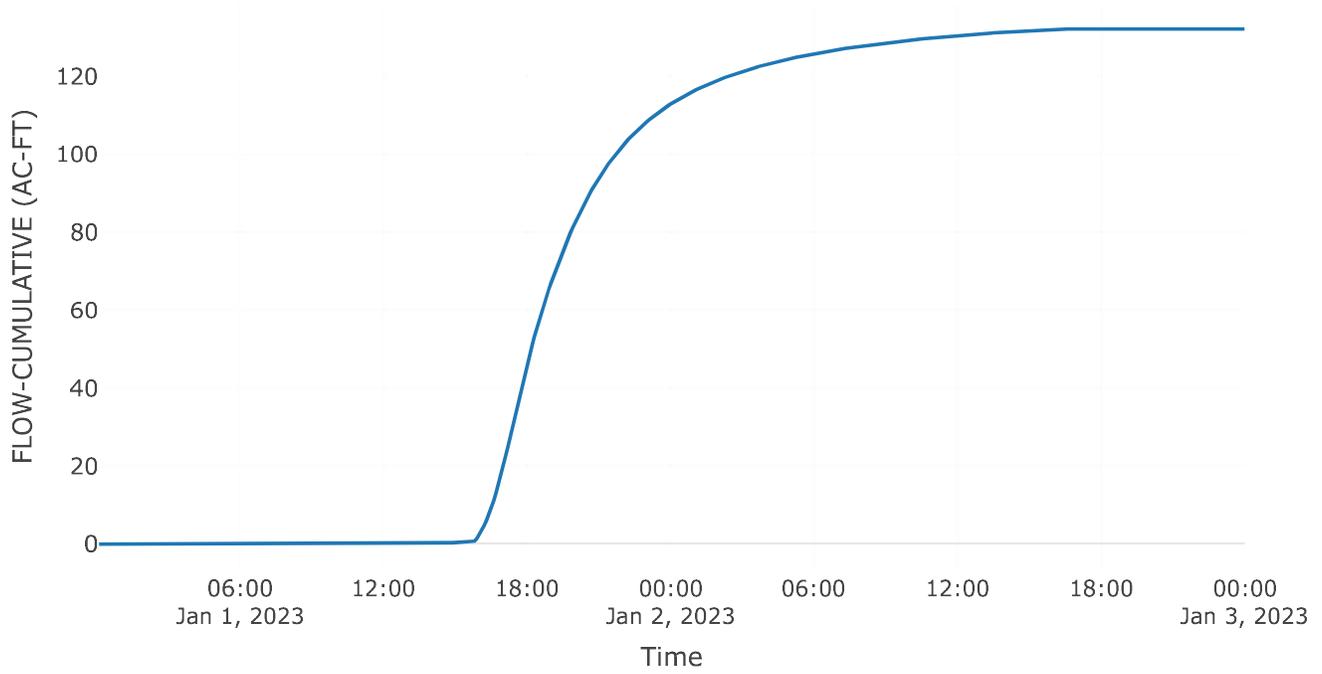
## Global Results Summary

Hydrologic Element	Drainage Area (MI <sup>2</sup> )	Peak Discharge (CFS)	Time of Peak	Volume ( )
XS ID 4 - EX_25	Not specified	327.44	01Jan2023, 17:18	Not specified
Quater Horse Basin	Not specified	0	31Dec2022, 24:00	Not specified
XS ID 79 - EX_25	Not specified	378.54	01Jan2023, 16:24	Not specified
Apache Basin	Not specified	24.7	02Jan2023, 00:18	Not specified
XS ID 17 - EX_25	Not specified	671.78	01Jan2023, 15:54	Not specified
Shawnee Basin	Not specified	99.07	01Jan2023, 22:00	Not specified
XS ID 72 - EX_25	Not specified	246.35	01Jan2023, 17:36	Not specified
Iron Mountain Basin	Not specified	0	31Dec2022, 24:00	Not specified
XS ID 74 - EX_25	Not specified	898.97	01Jan2023, 16:06	Not specified
Geurts Basin_Upper	Not specified	705.89	01Jan2023, 17:06	Not specified
Geurts Basin_Lower	Not specified	371.74	01Jan2023, 19:12	Not specified

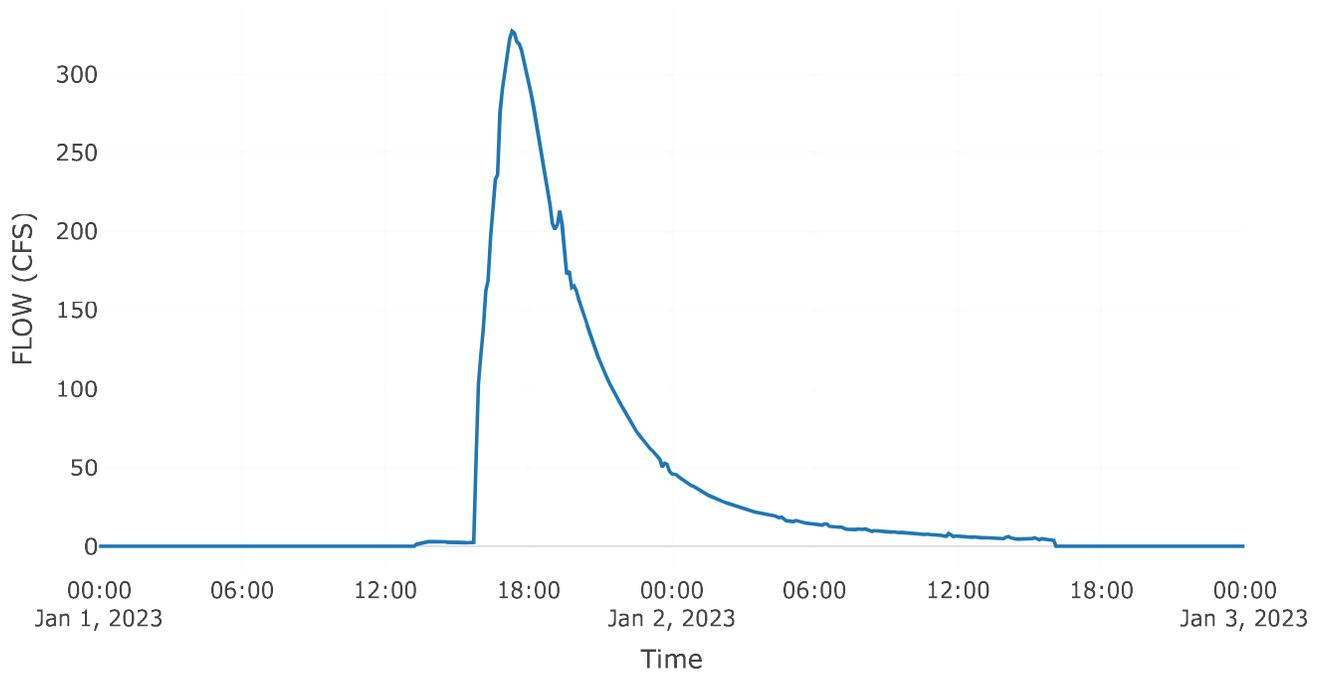
**Source: XS ID 4 - EX\_25****Downstream** : Quater Horse Basin**Flow Method** : Gage Flow**Flow Gage** : XS ID 4 - EX\_25**Results: XS ID 4 - EX\_25**

Peak Discharge (CFS)	327.44
Time of Peak Discharge	01Jan2023, 17:18

### Cumulative Outflow



### Outflow

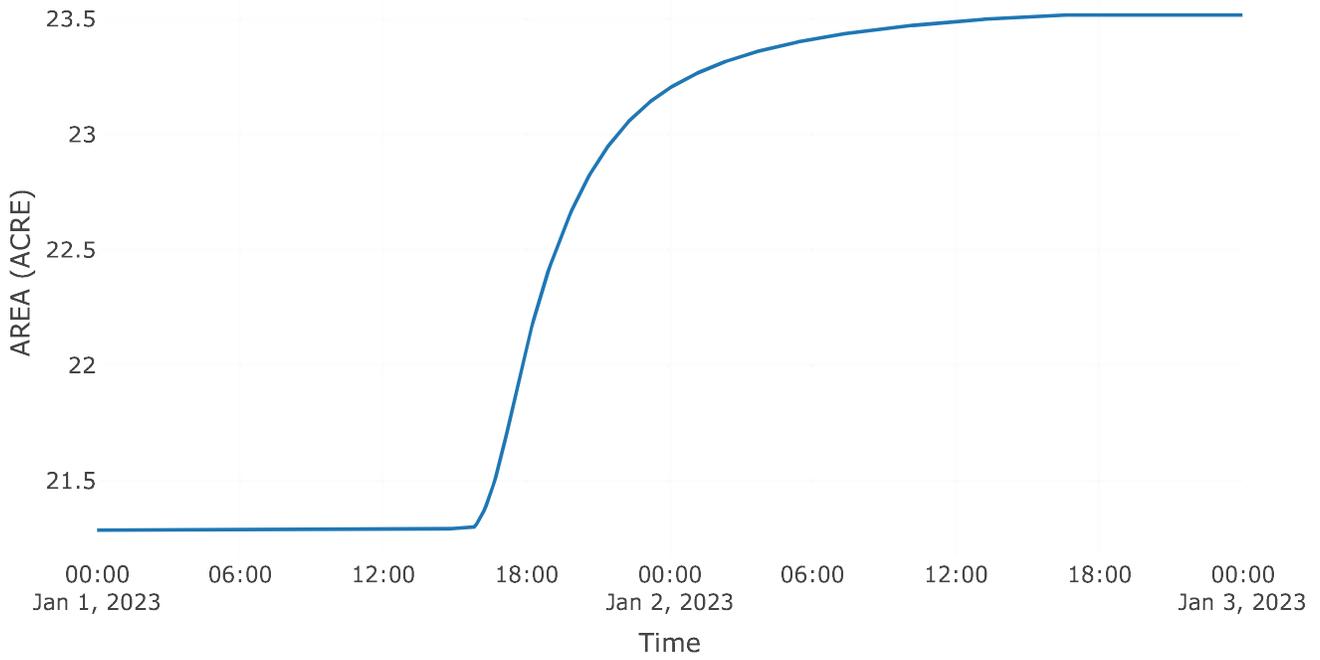


## Reservoir: Quater Horse Basin

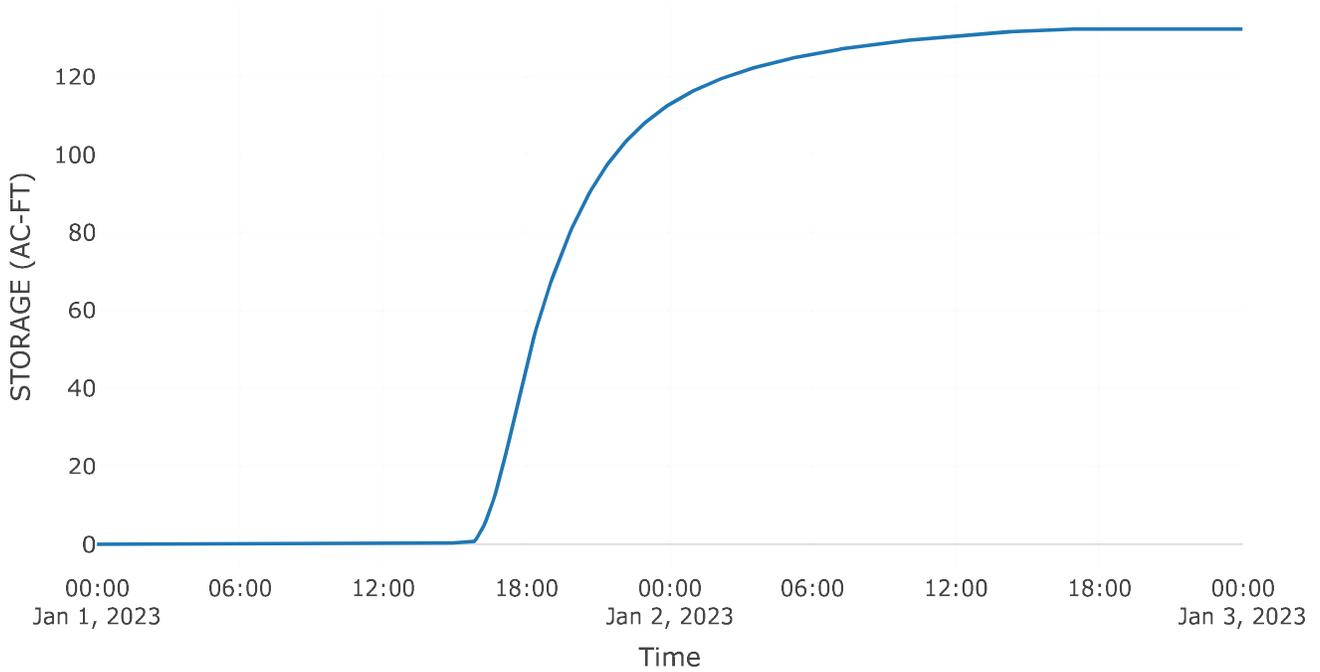
### Results: Quater Horse Basin

Peak Discharge (CFS)	0
Time of Peak Discharge	31Dec2022, 24:00
Peak Inflow (CFS)	327.44
Time of Peak Inflow	01Jan2023, 17:18
Inflow Volume (AC - FT)	132.21
Maximum Storage (AC - FT)	132.21
Peak Elevation (FT)	4390.9
Discharge Volume (AC - FT)	0

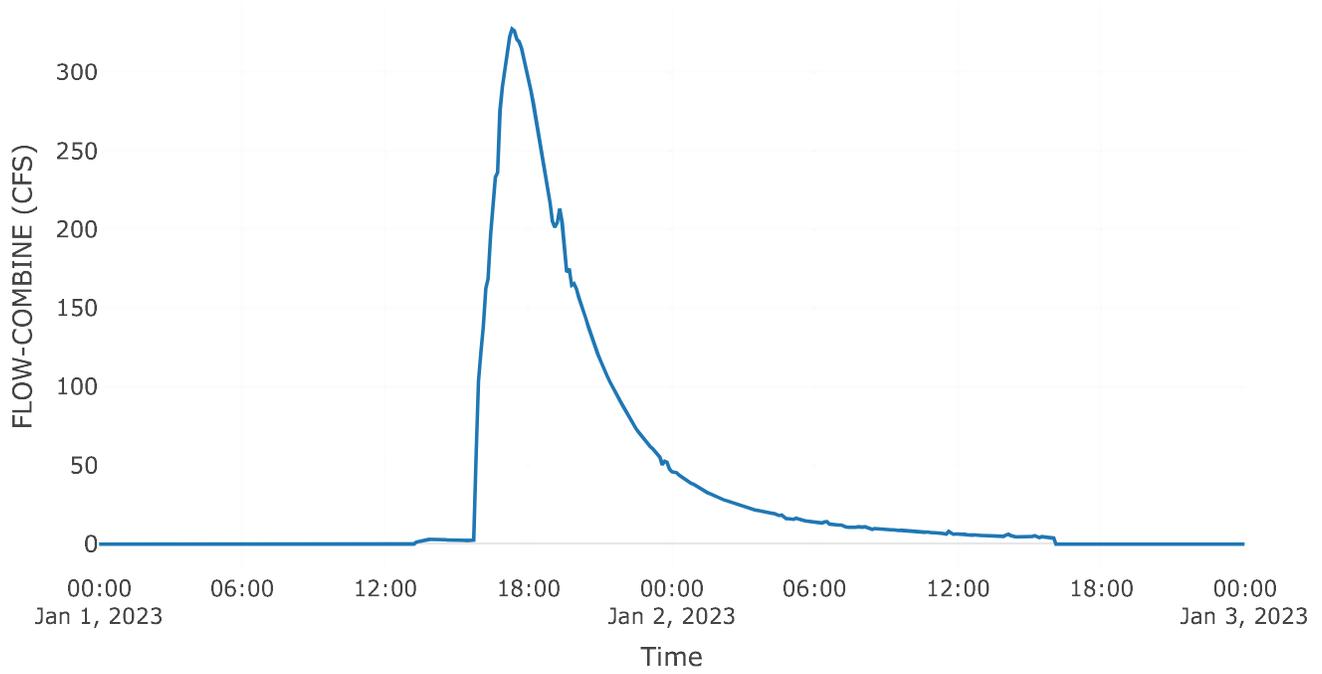
### Reservoir Area



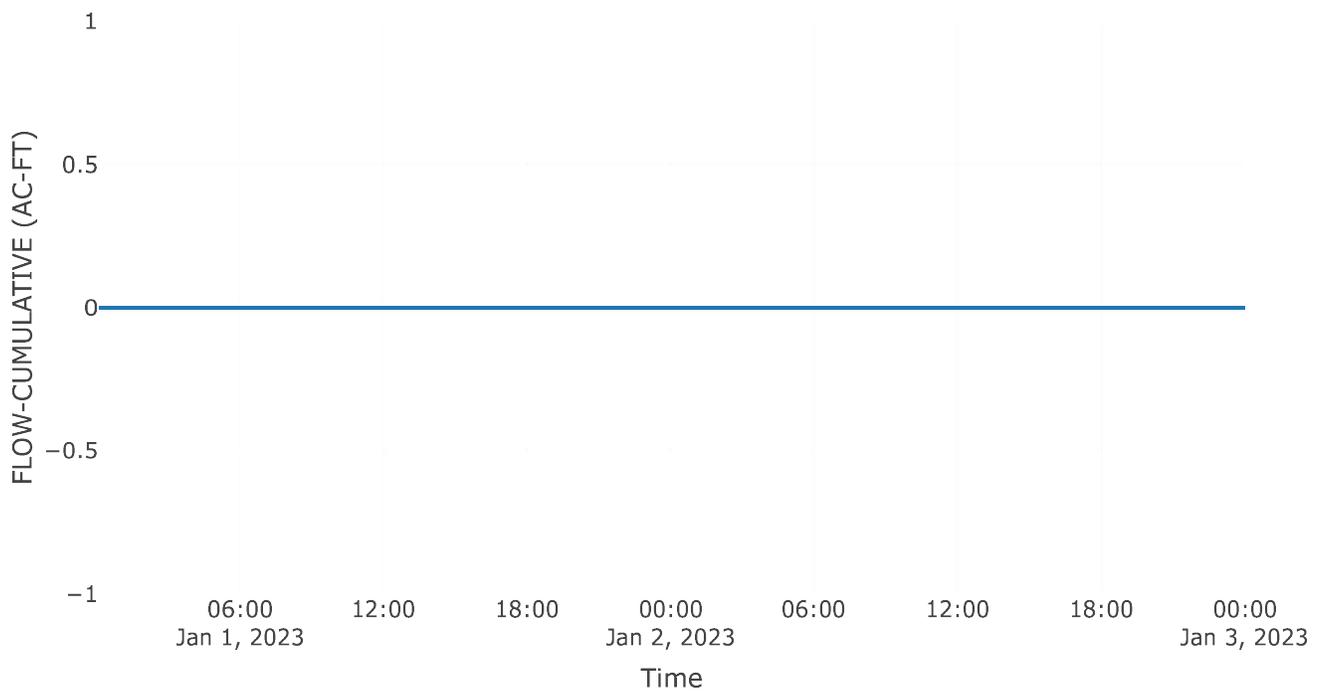
### Storage



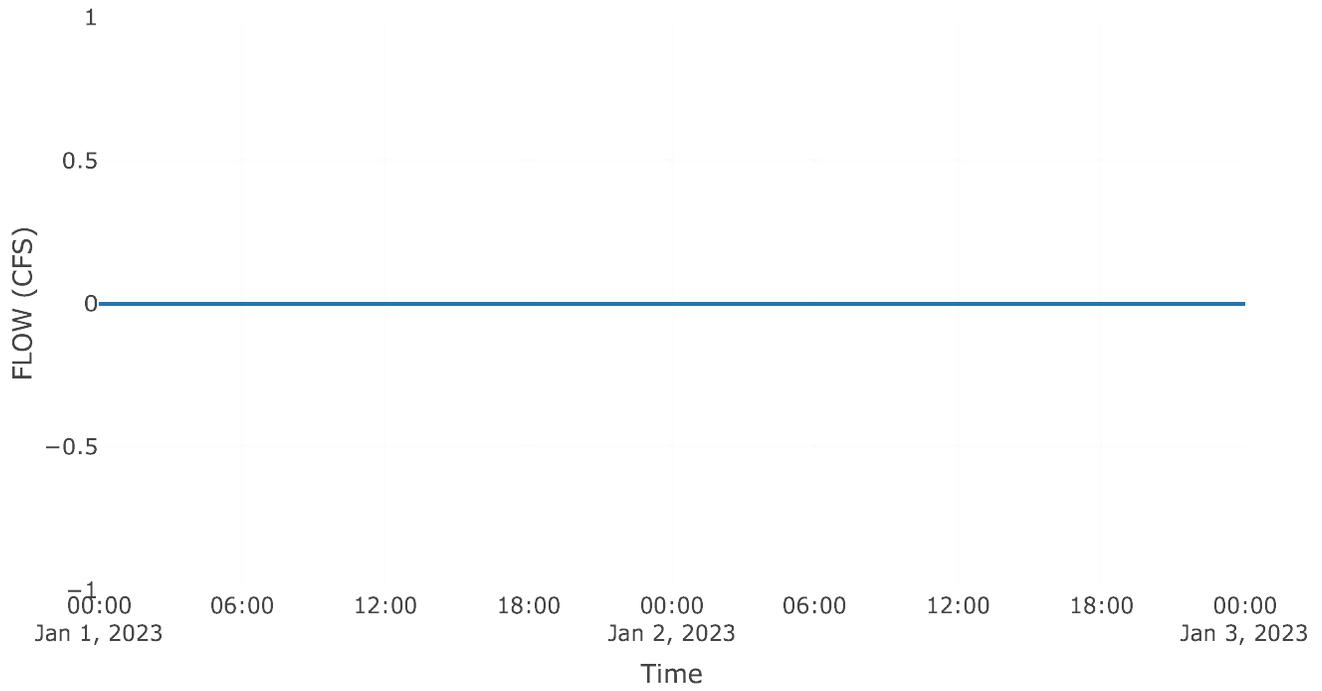
### Combined Inflow



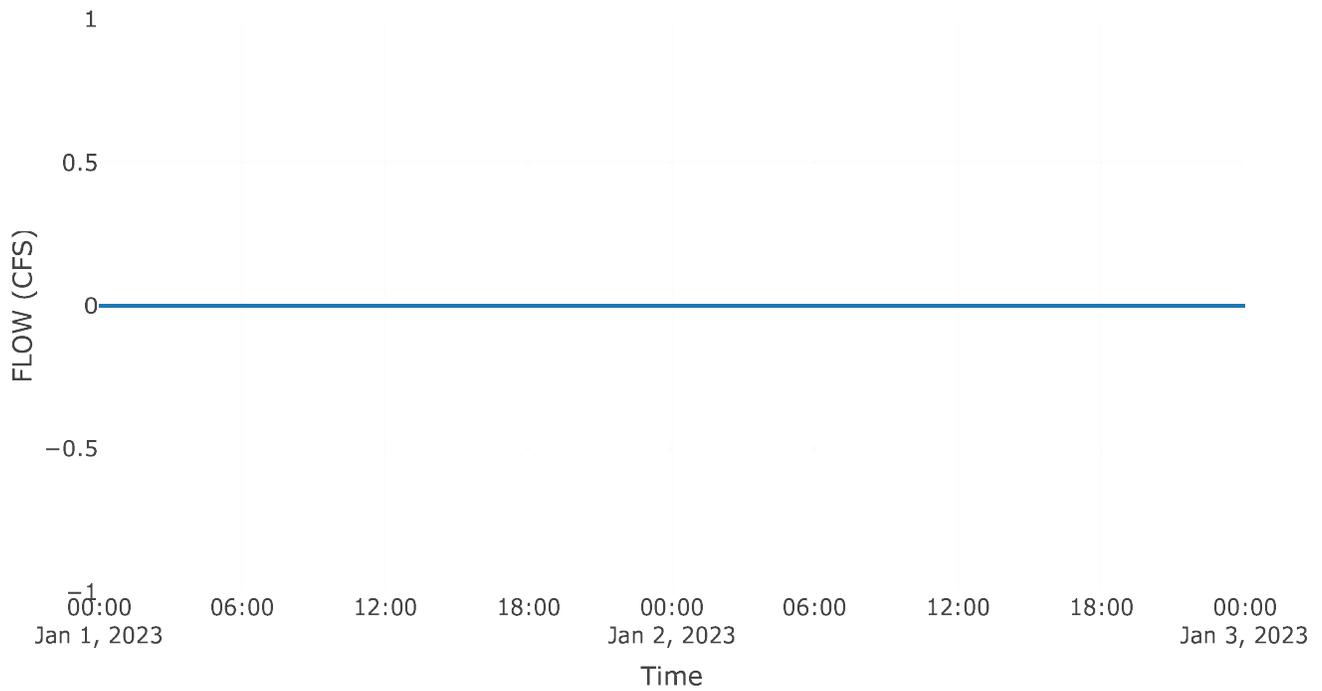
### Cumulative Outflow



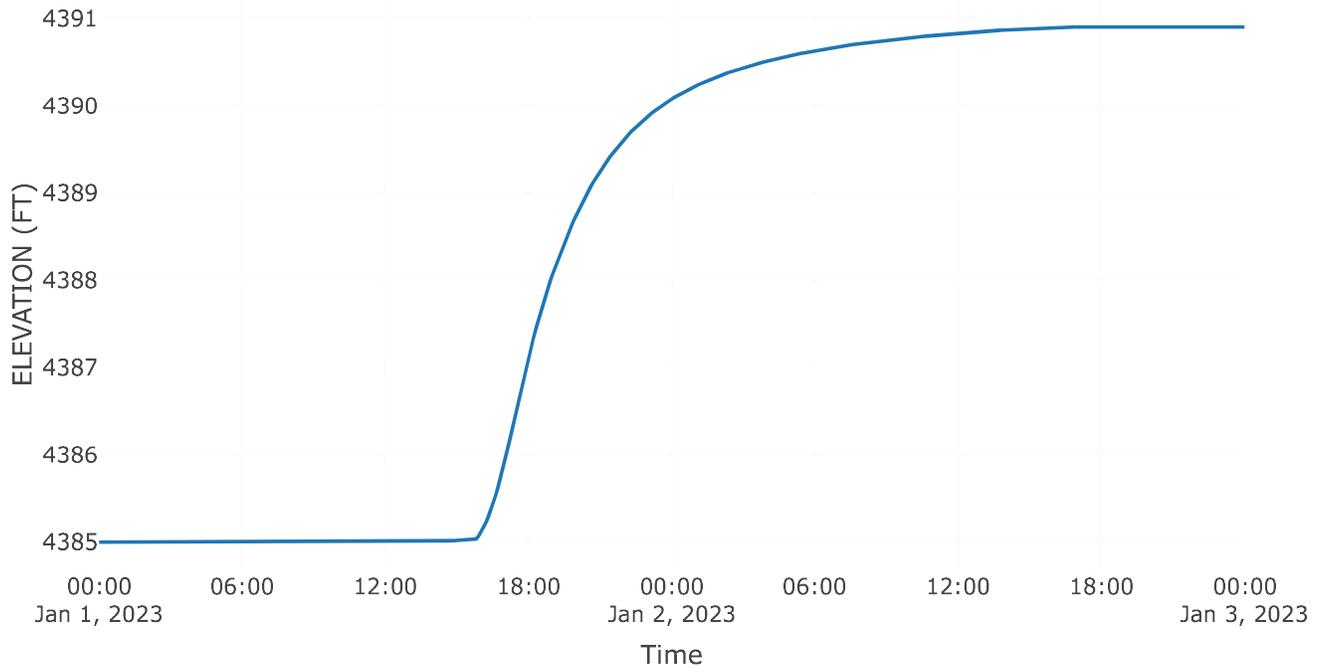
### Spillway 1



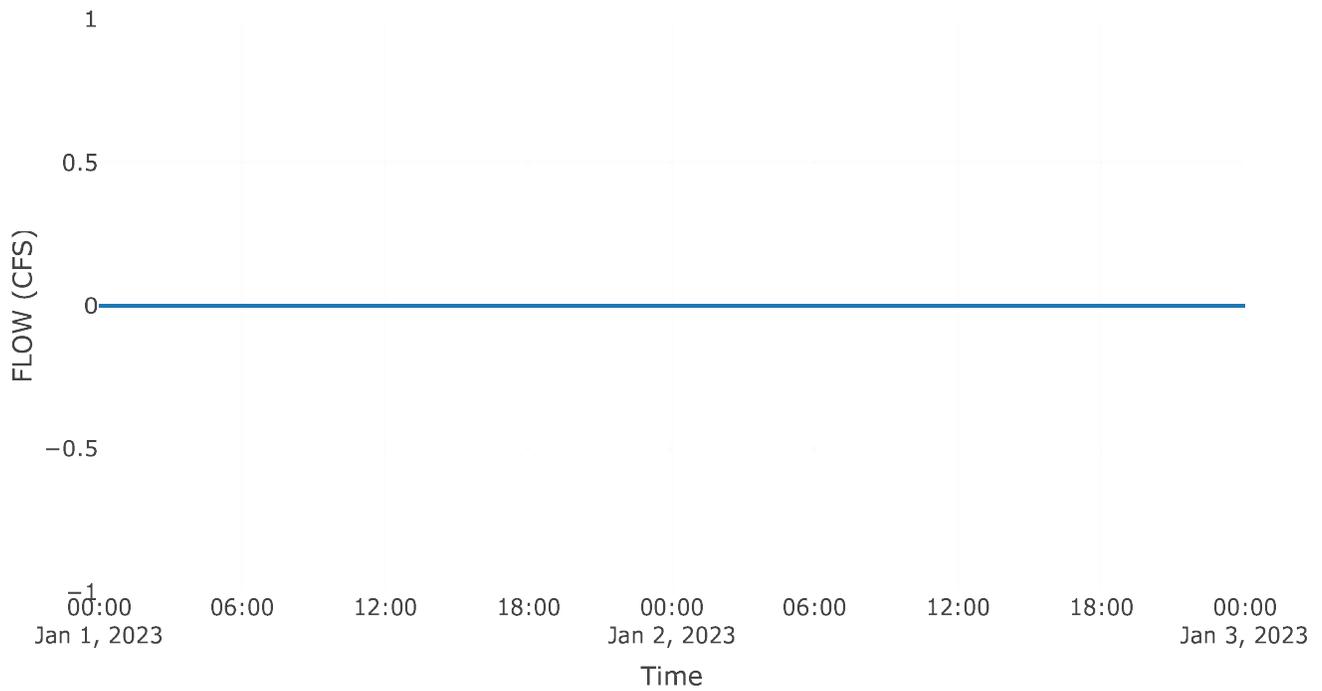
### Outlet 1



### Pool Elevation



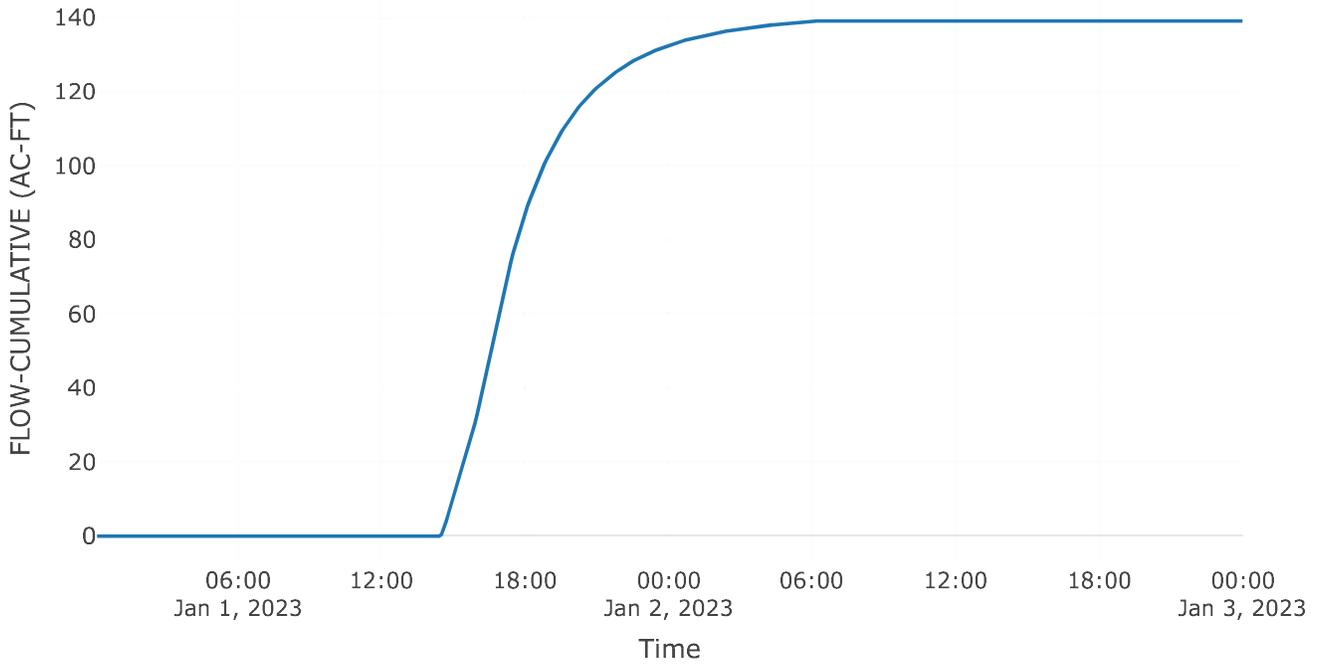
### Outflow



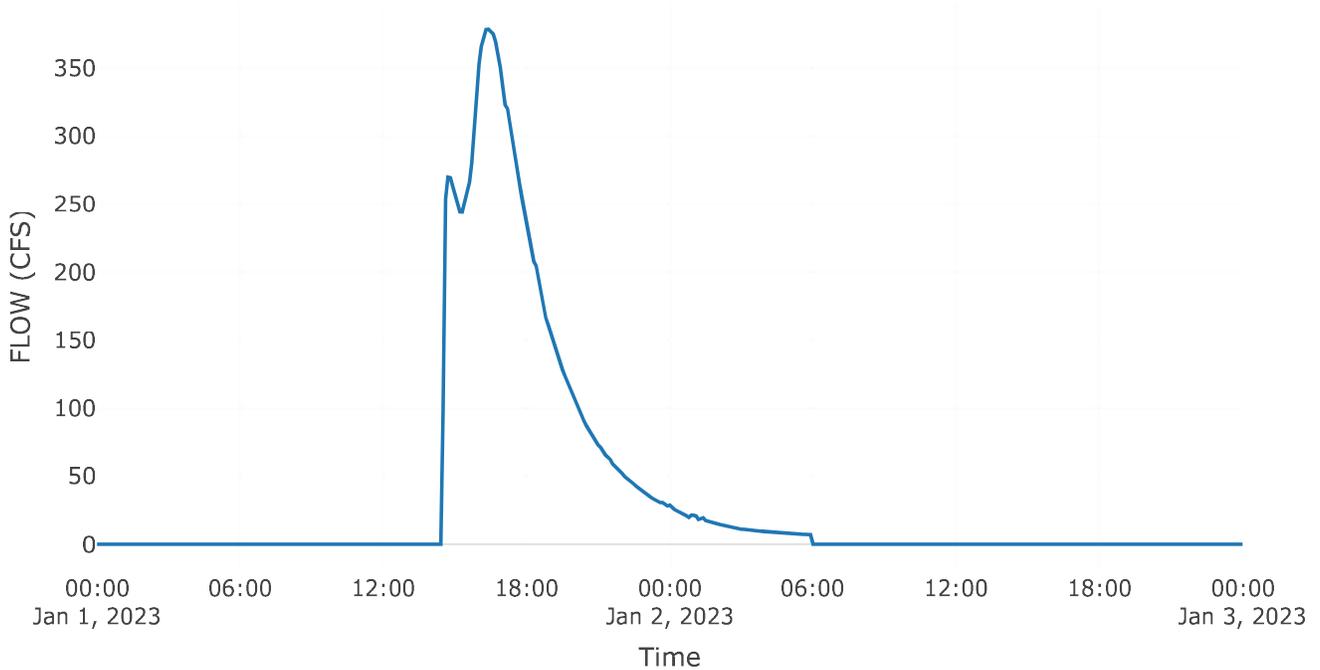
**Source: XS ID 79 - EX\_25****Downstream** : Apache Basin**Flow Method** : Gage Flow**Flow Gage** : XS ID 79 - EX\_25**Results: XS ID 79 - EX\_25**

Peak Discharge (CFS)	378.54
Time of Peak Discharge	01Jan2023, 16:24

### Cumulative Outflow



### Outflow

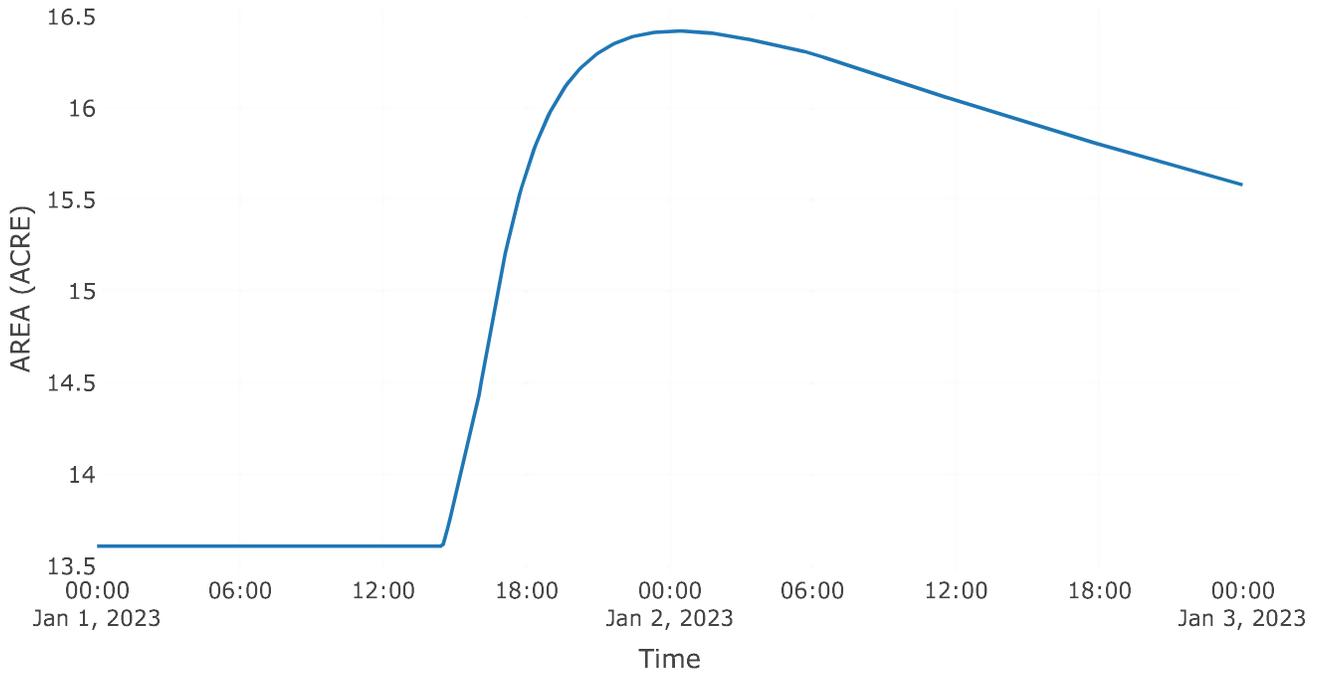


## Reservoir: Apache Basin

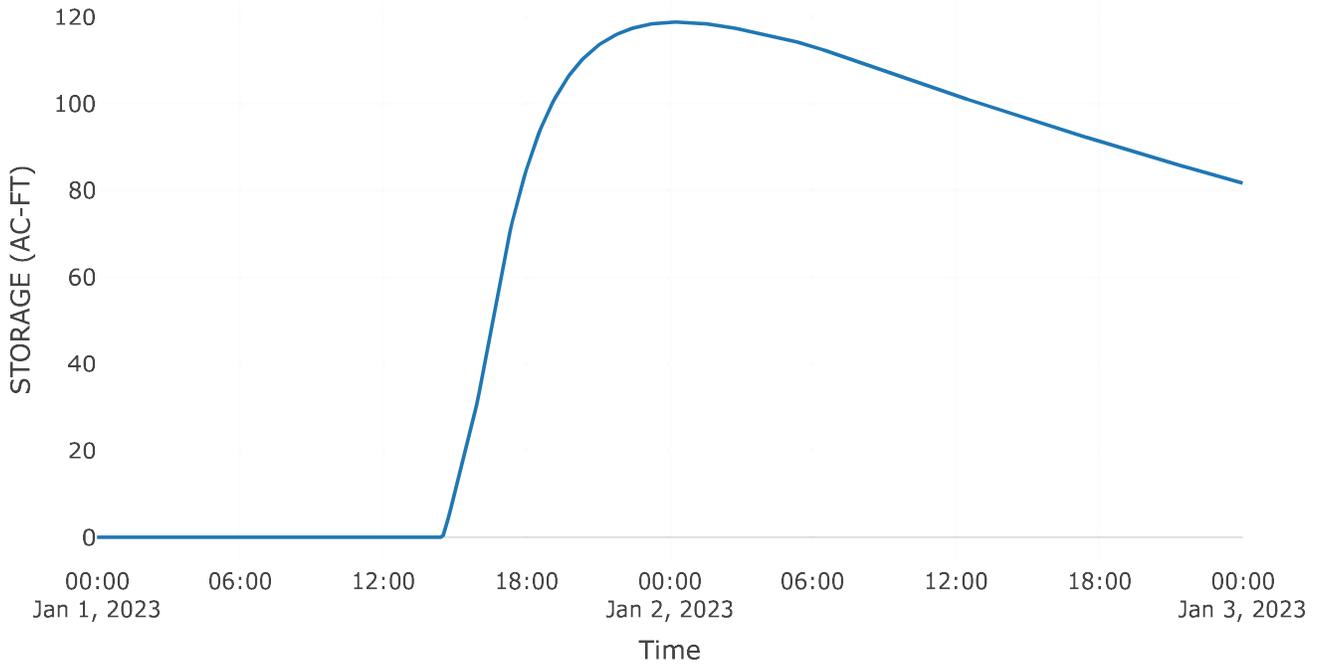
### Results: Apache Basin

Peak Discharge (CFS)	24.7
Time of Peak Discharge	02Jan2023, 00:18
Peak Inflow (CFS)	378.54
Time of Peak Inflow	01Jan2023, 16:24
Inflow Volume (AC - FT)	139.09
Maximum Storage (AC - FT)	118.85
Peak Elevation (FT)	4532.92
Discharge Volume (AC - FT)	57.38

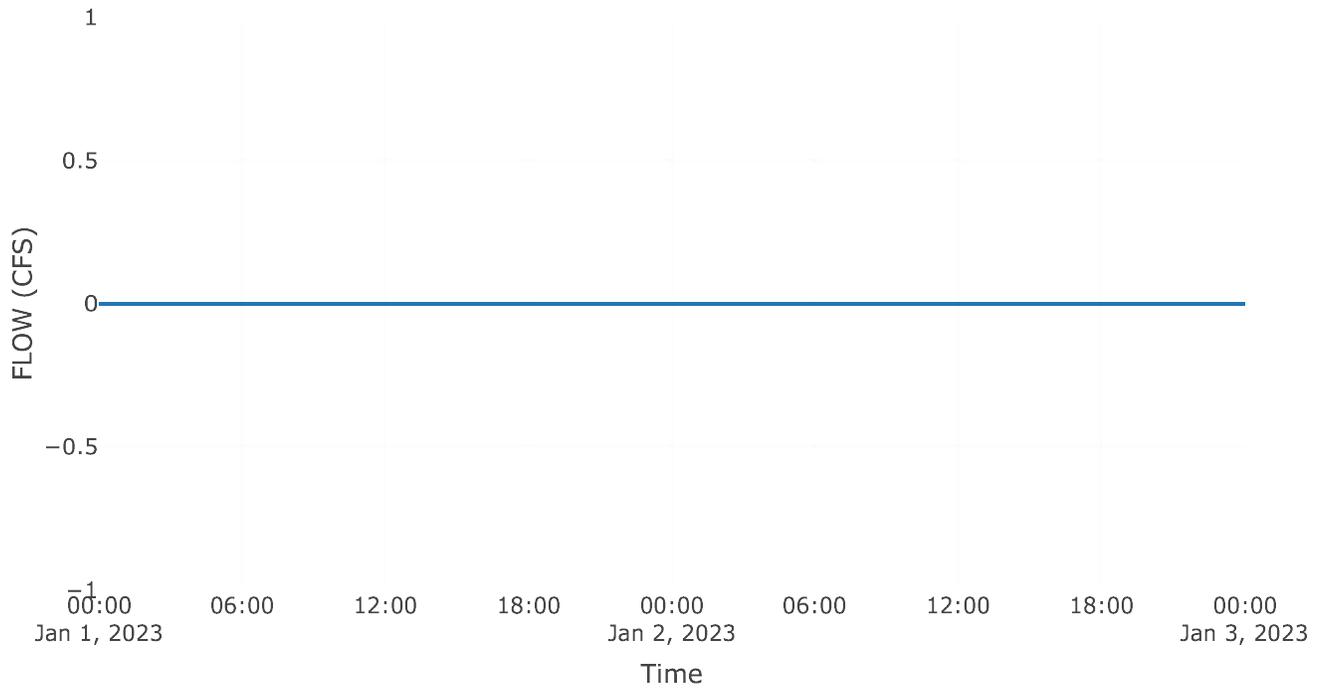
### Reservoir Area



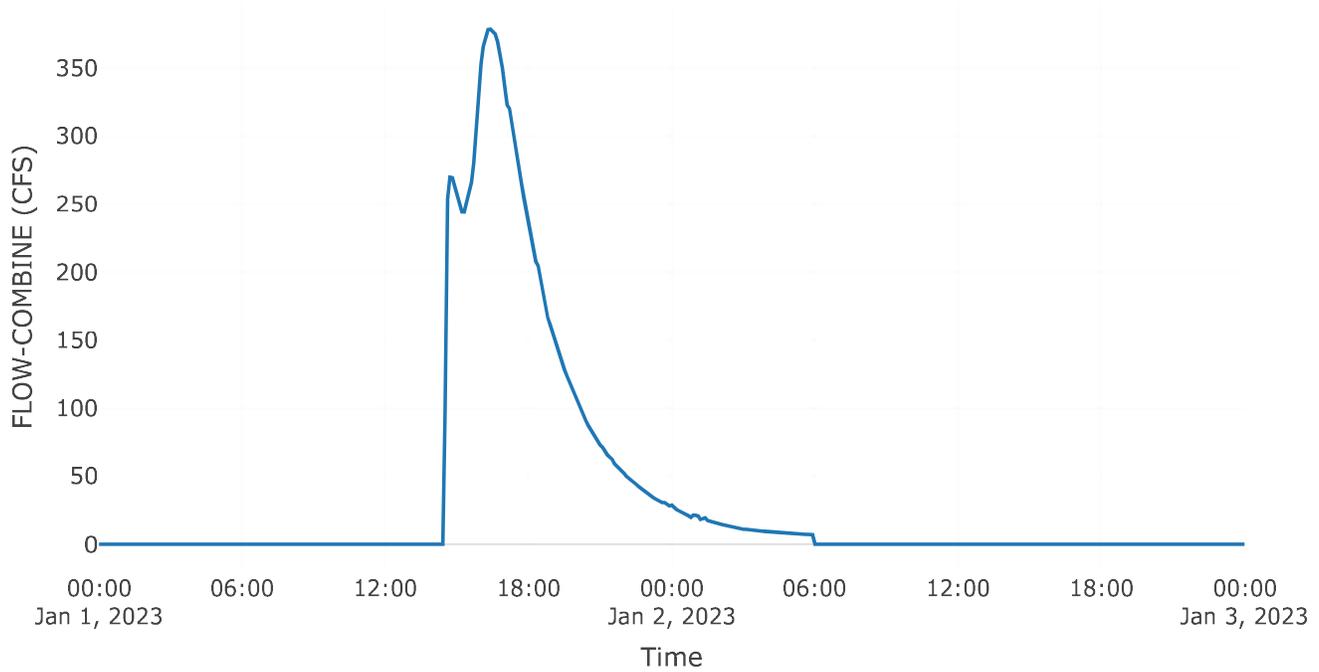
### Storage



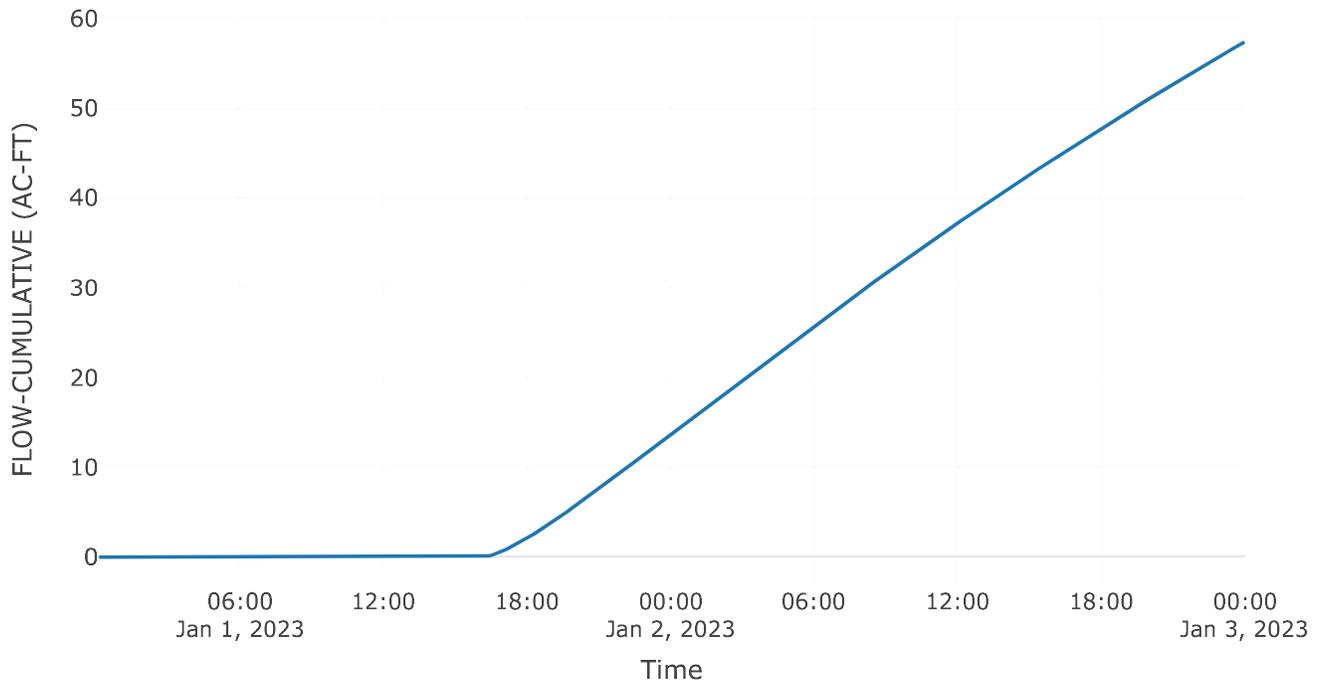
### Spillway 3



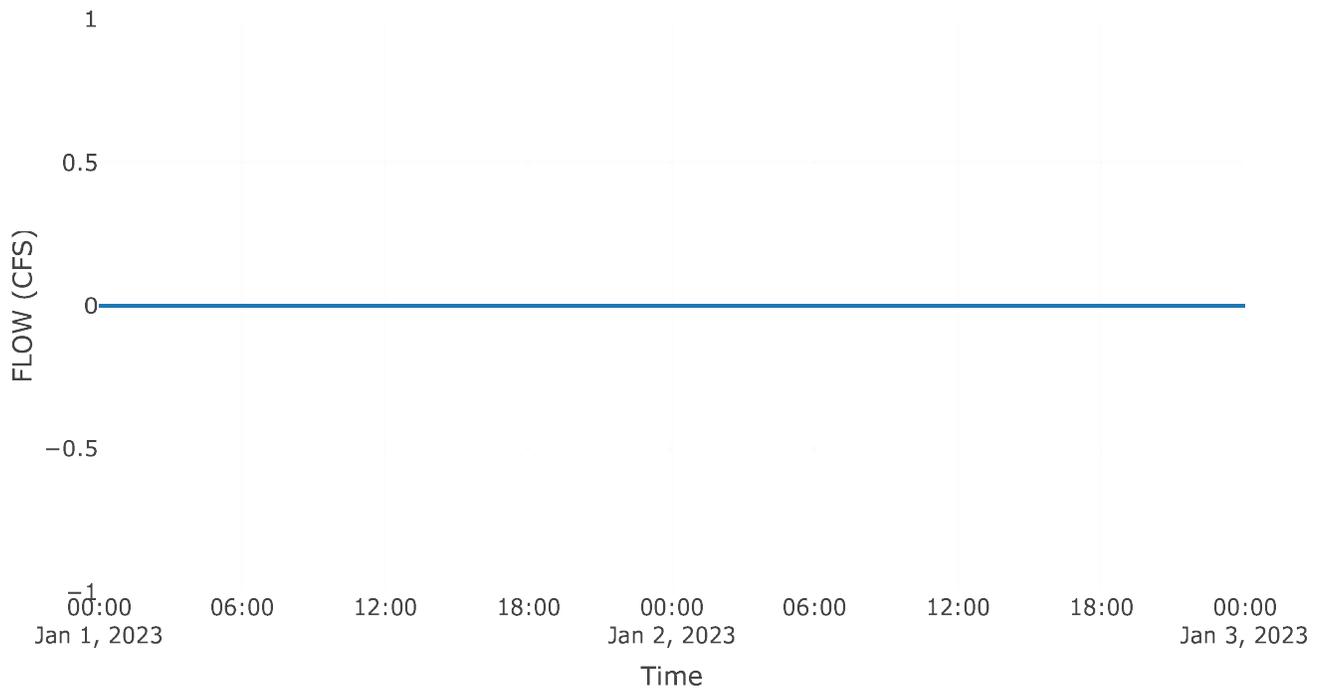
### Combined Inflow



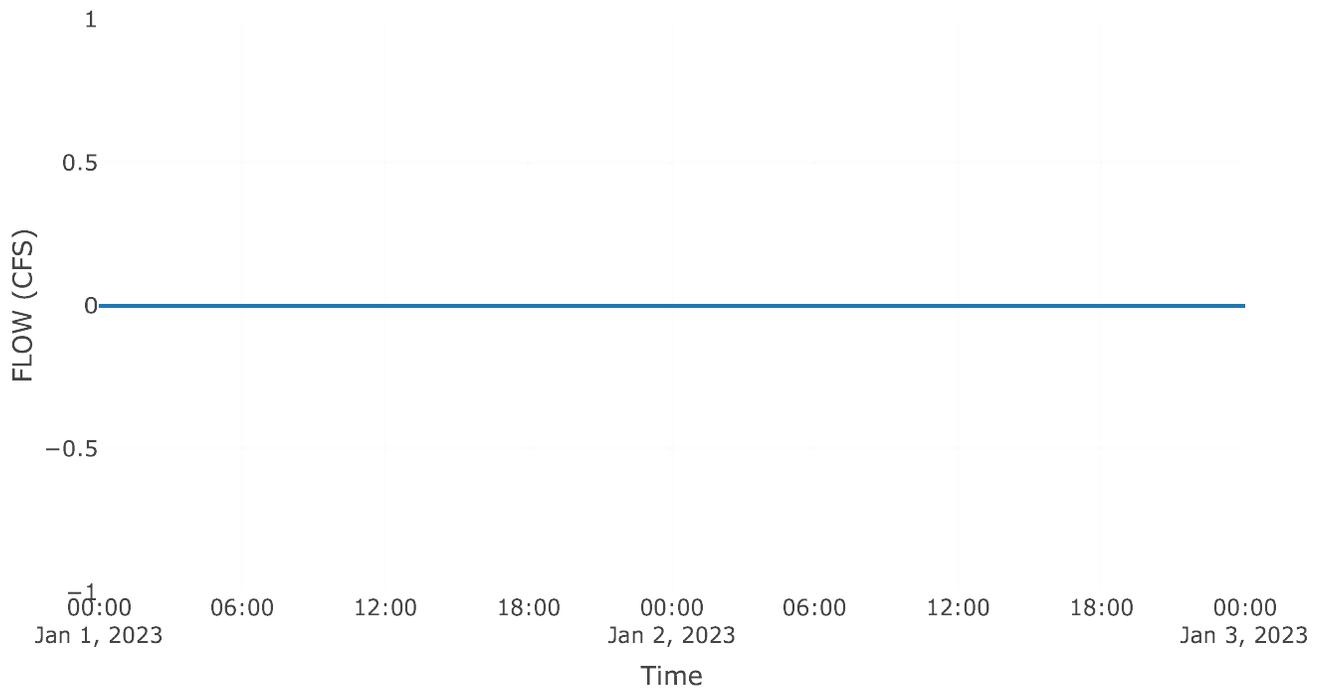
### Cumulative Outflow



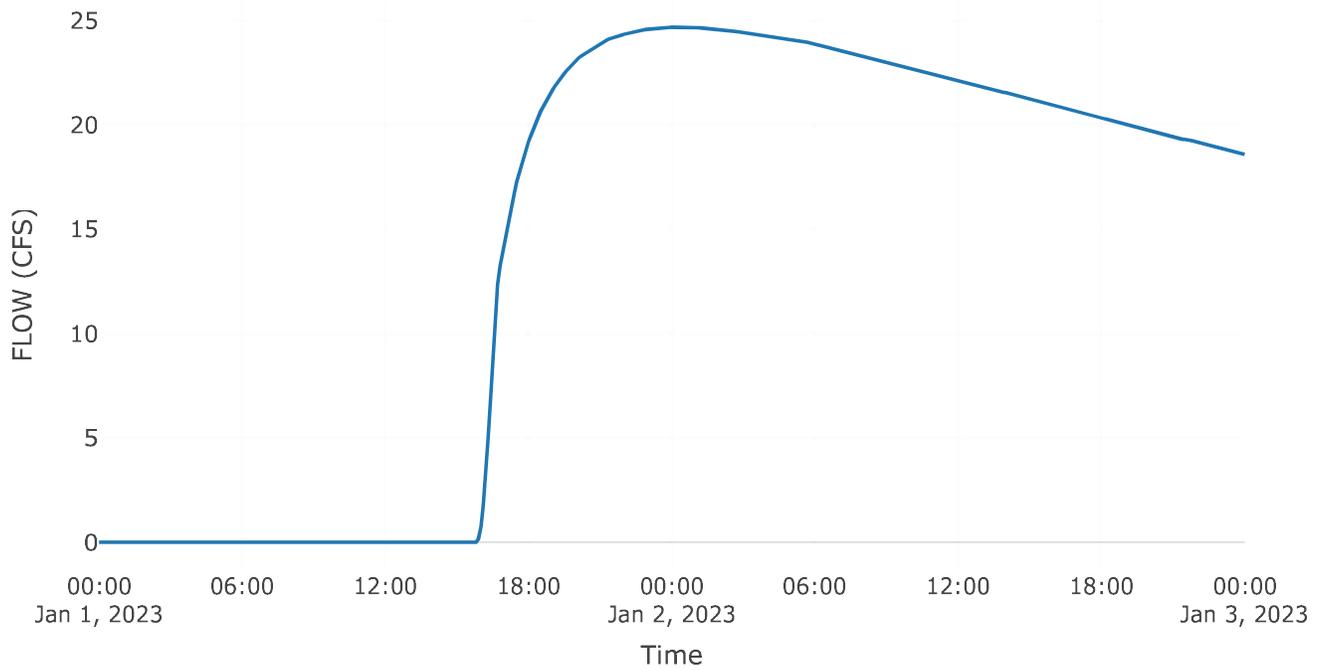
### Spillway 1



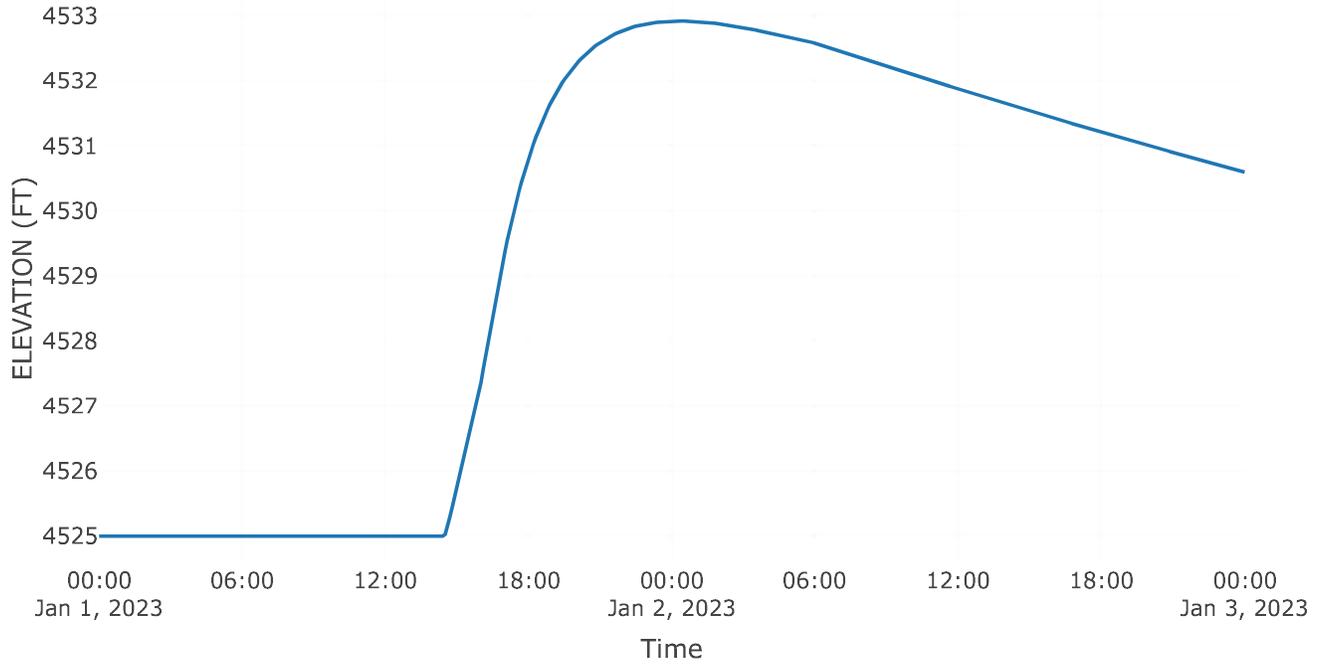
### Spillway 2



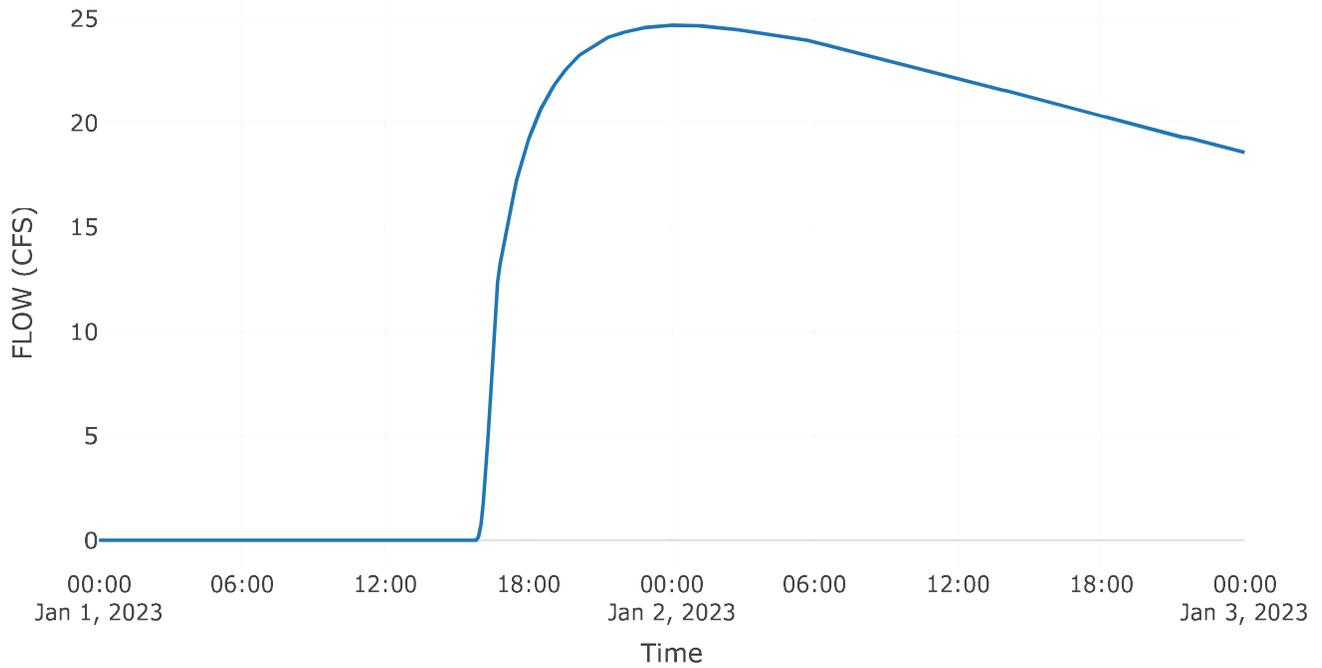
### Outlet 1



### Pool Elevation



### Outflow



## Source: XS ID 17 - EX\_25

**Downstream** : Shawnee Basin

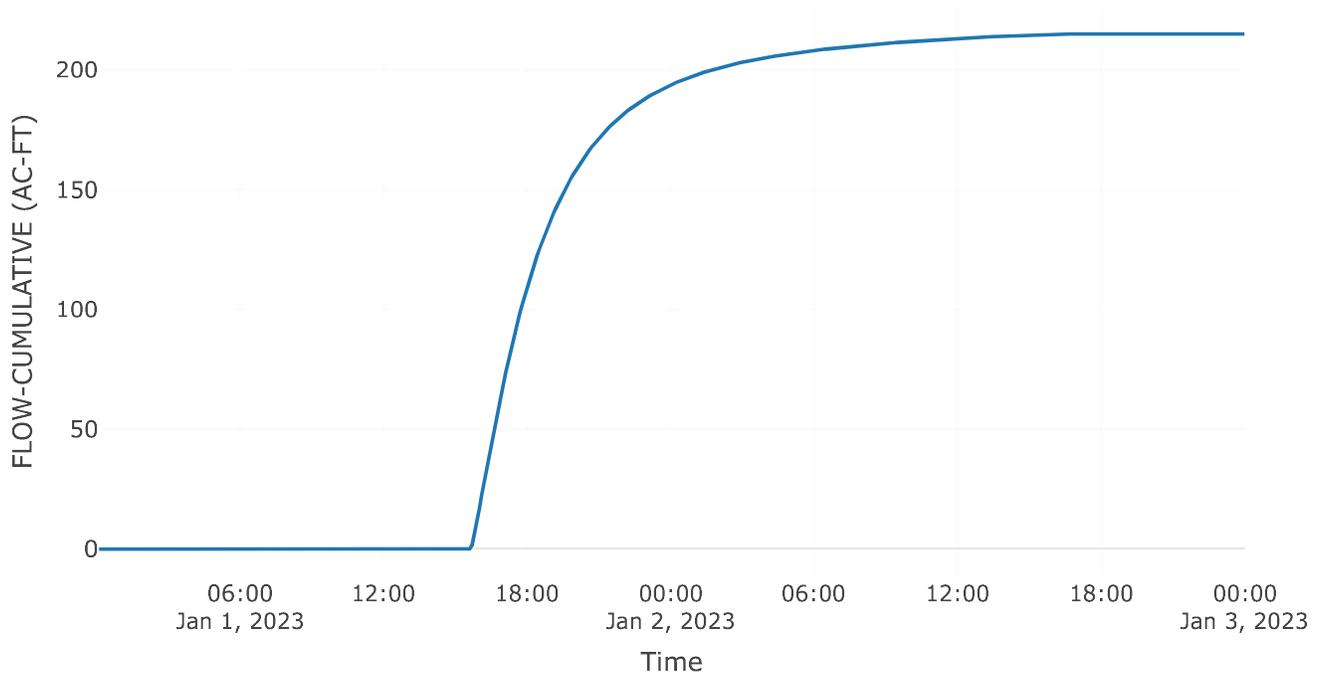
**Flow Method** : Gage Flow

**Flow Gage** : XS ID 17 - EX\_25

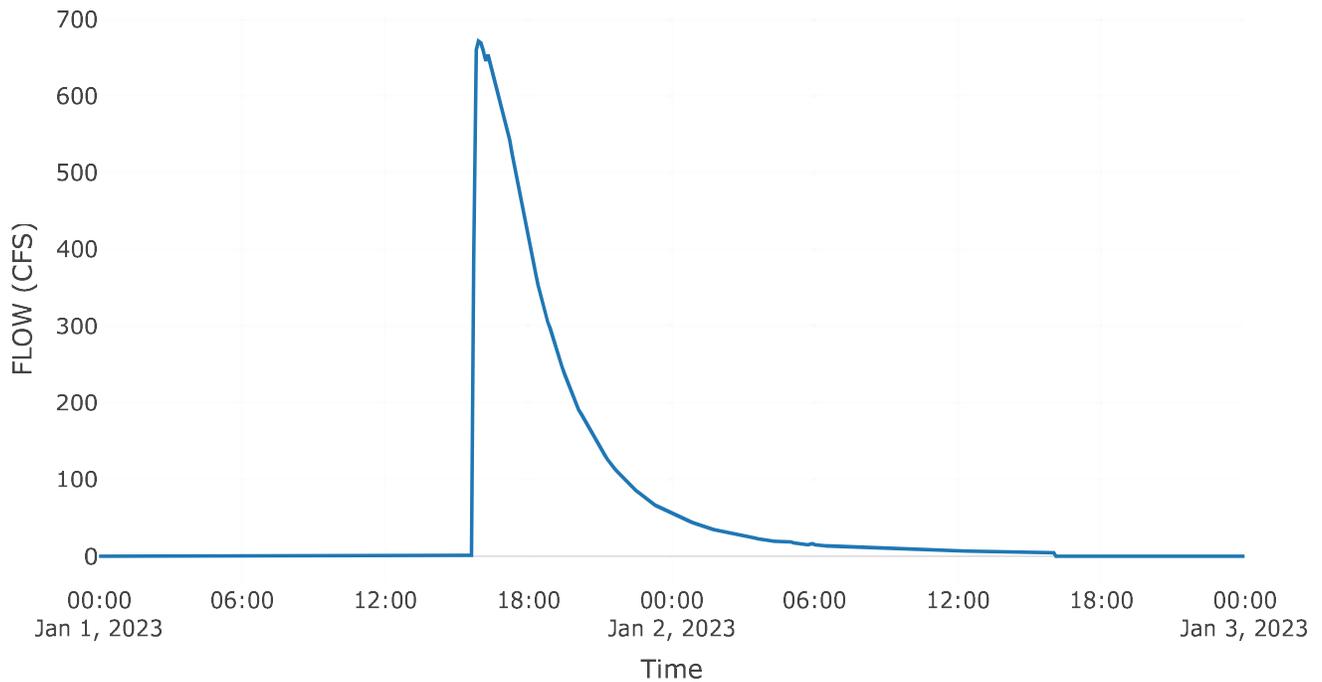
### Results: XS ID 17 - EX\_25

Peak Discharge (CFS)	671.78
Time of Peak Discharge	01Jan2023, 15:54

### Cumulative Outflow



### Outflow

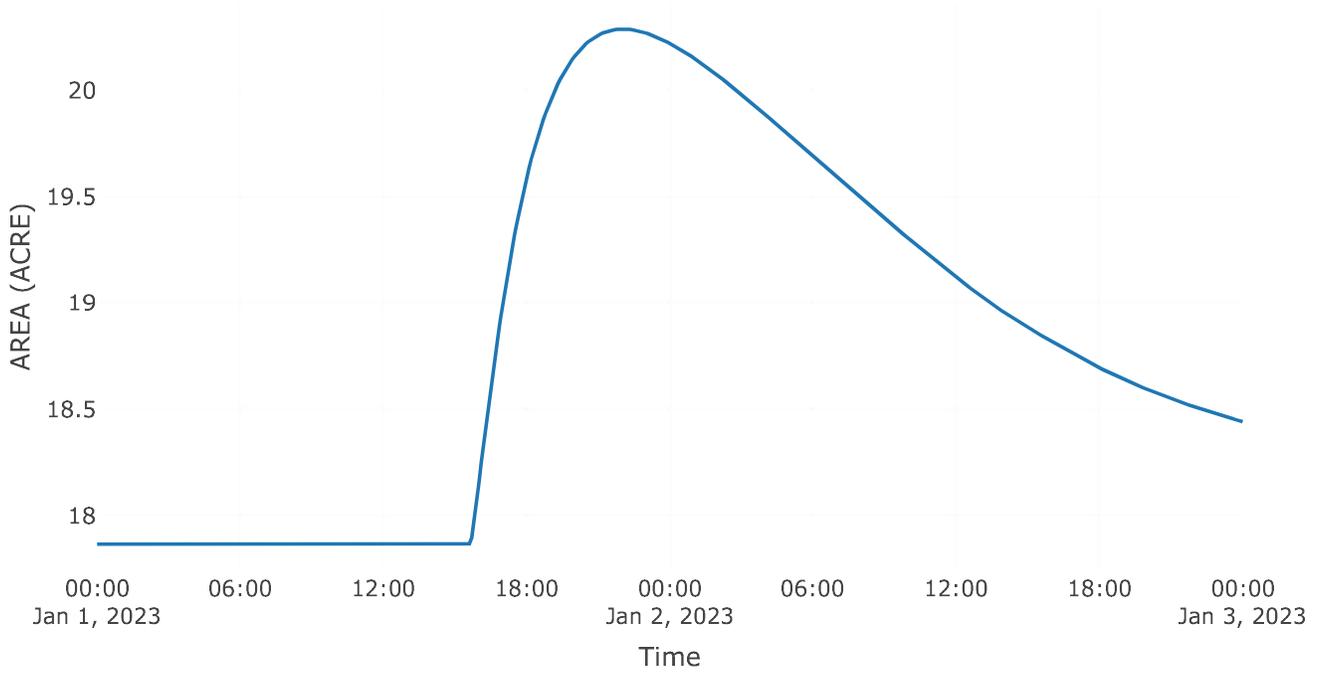


# Reservoir: Shawnee Basin

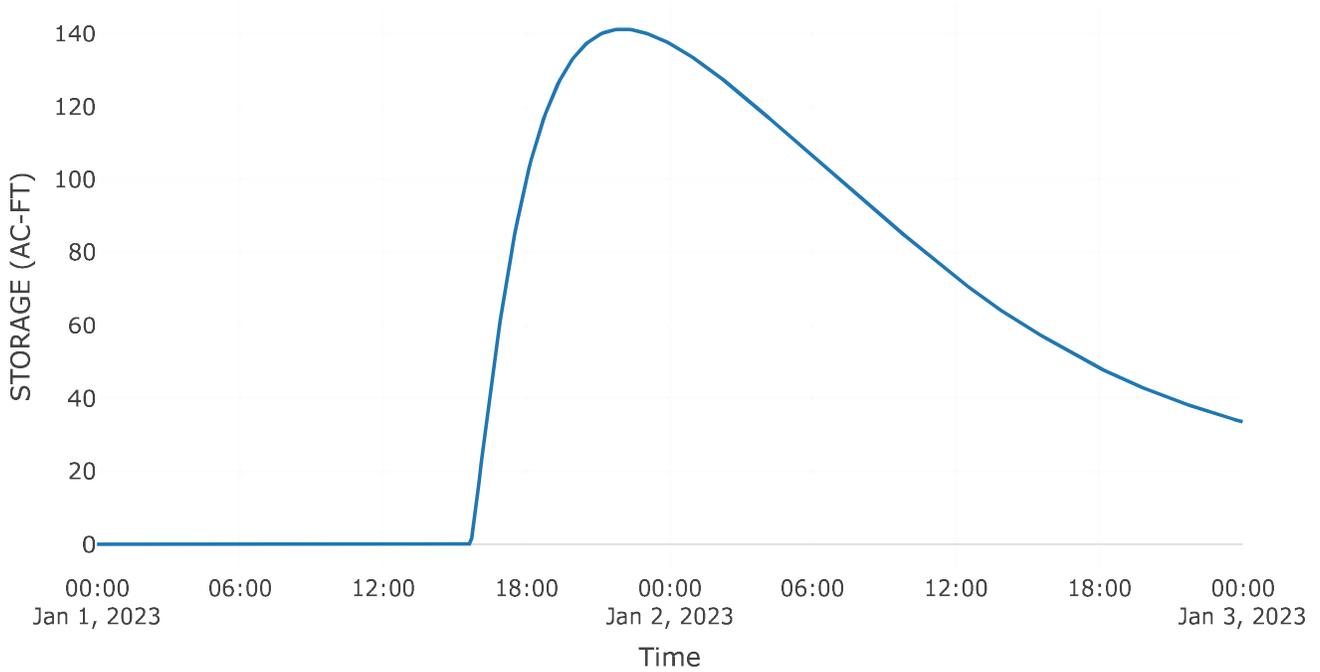
## Results: Shawnee Basin

Peak Discharge (CFS)	99.07
Time of Peak Discharge	01Jan2023, 22:00
Peak Inflow (CFS)	671.78
Time of Peak Inflow	01Jan2023, 15:54
Inflow Volume (AC - FT)	215.06
Maximum Storage (AC - FT)	141.29
Peak Elevation (FT)	4495.17
Discharge Volume (AC - FT)	181.46

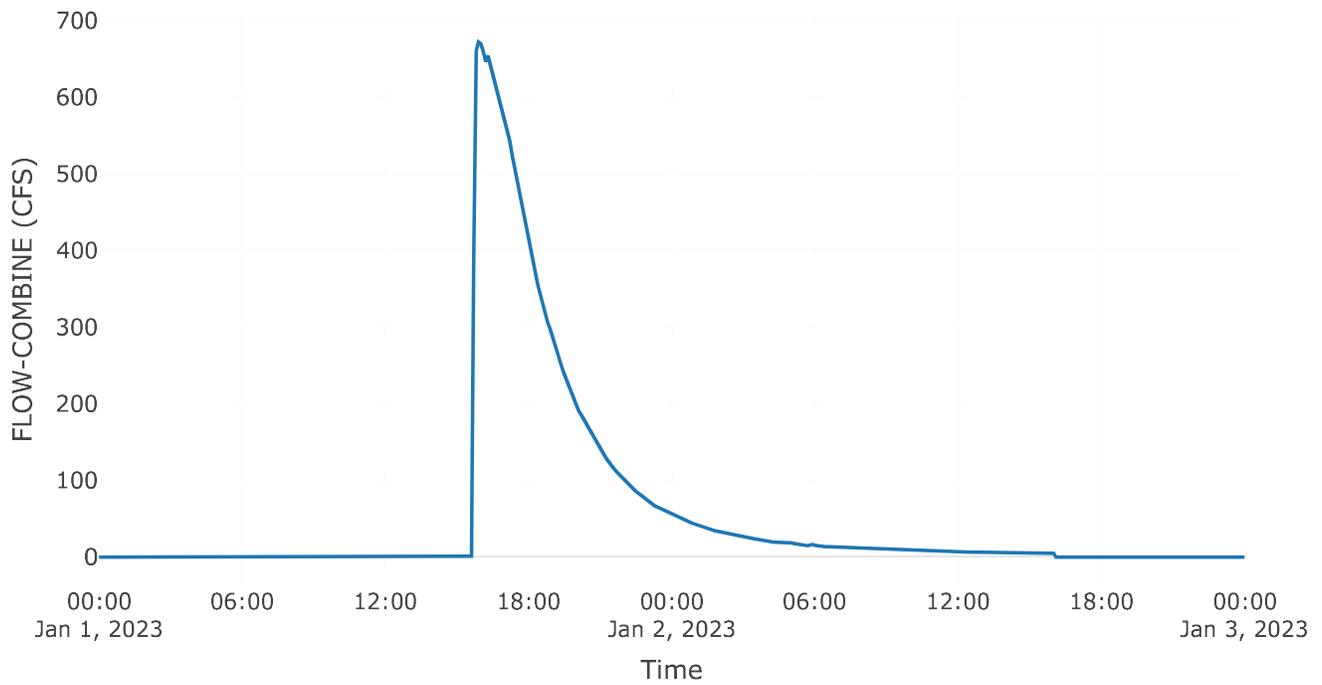
### Reservoir Area



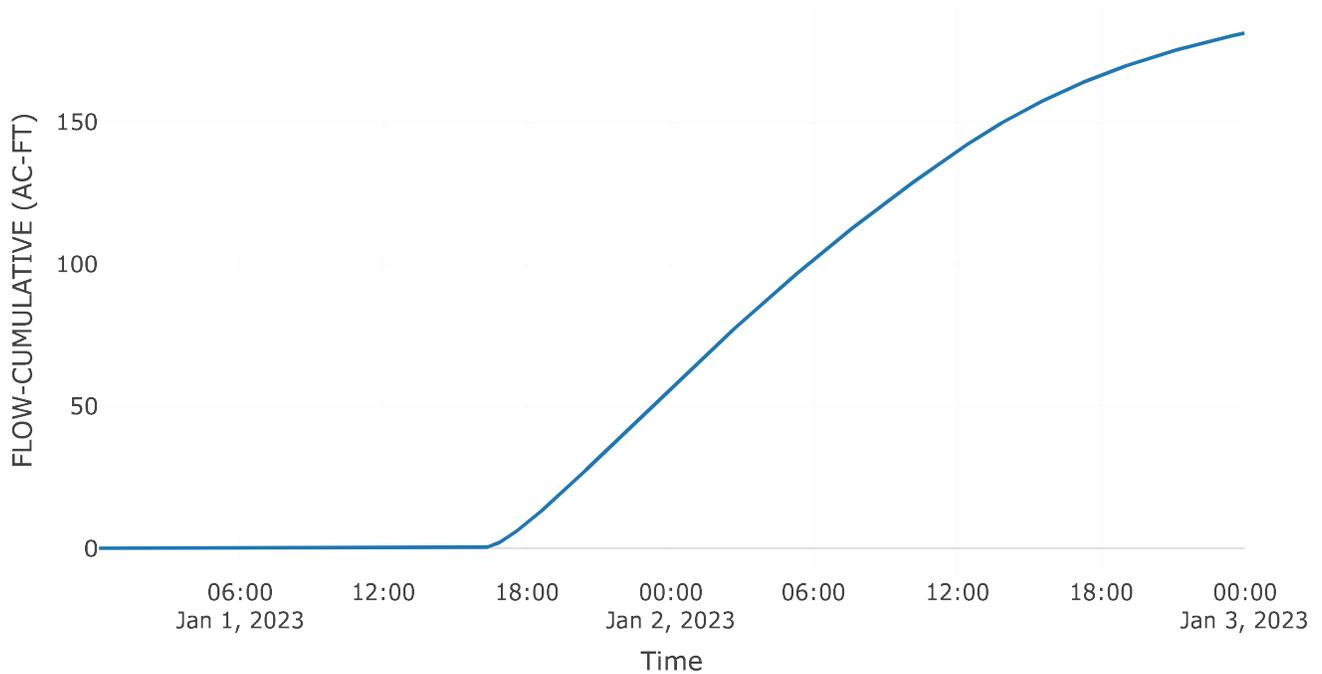
### Storage



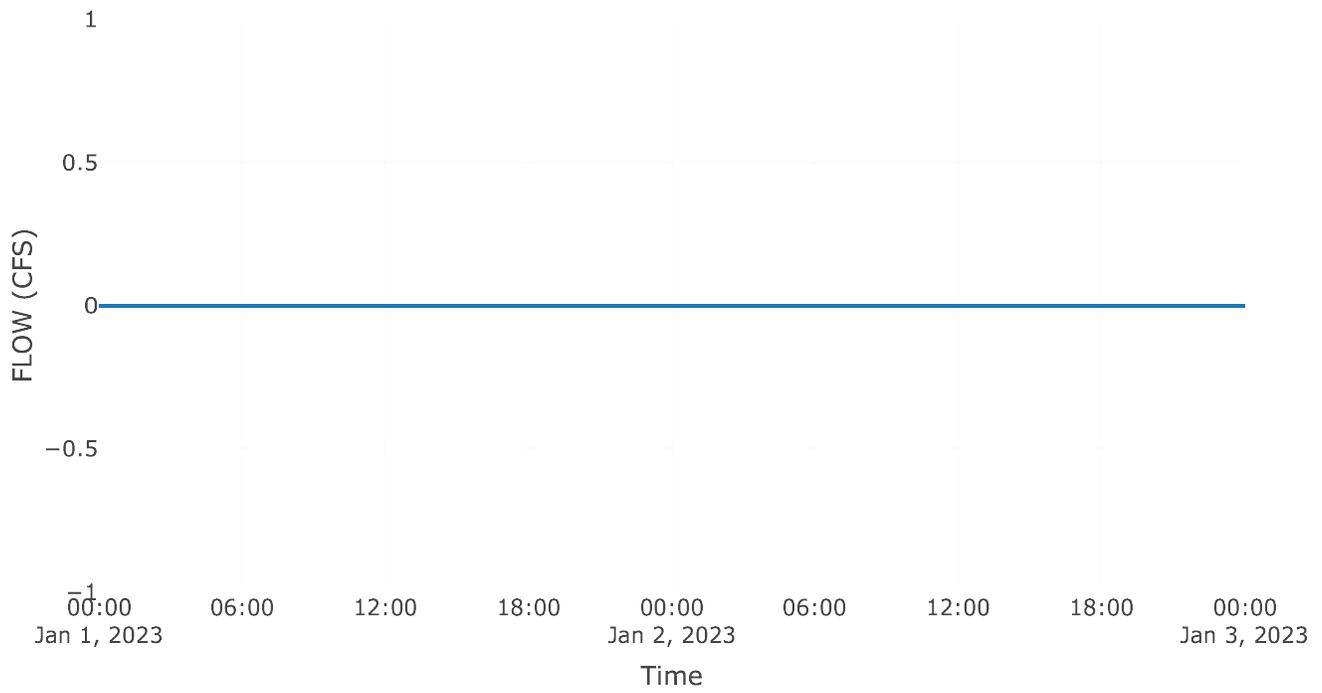
### Combined Inflow



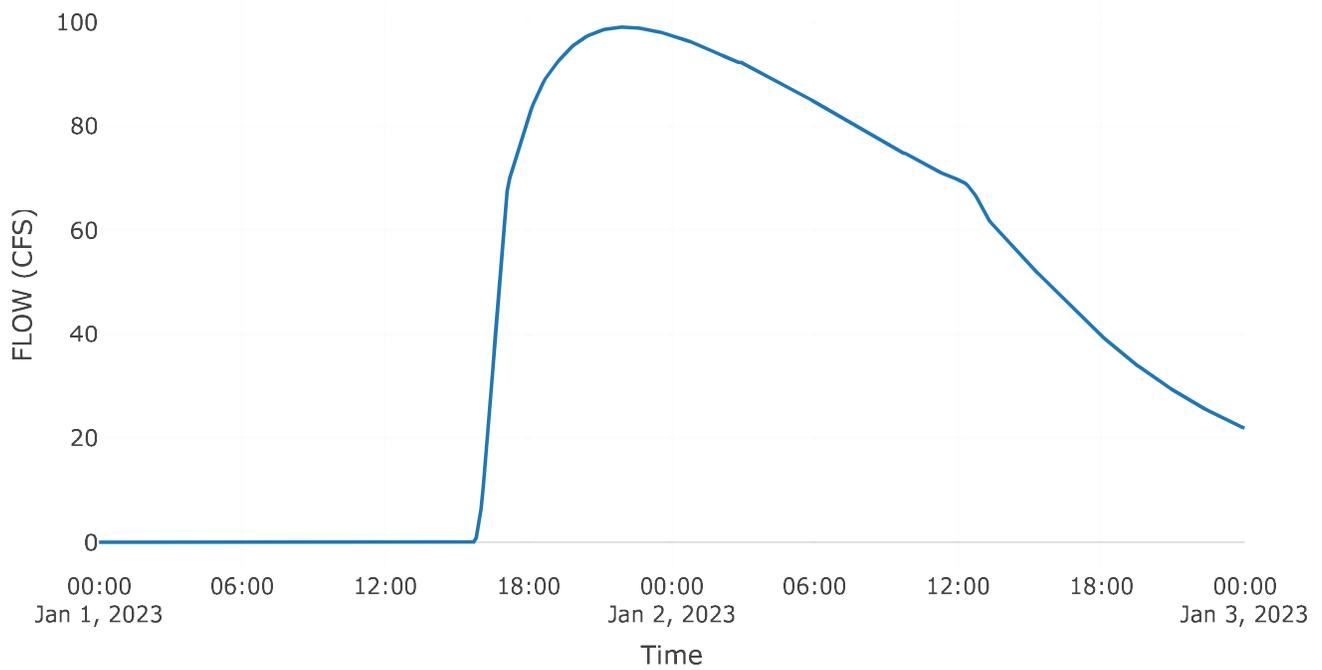
### Cumulative Outflow



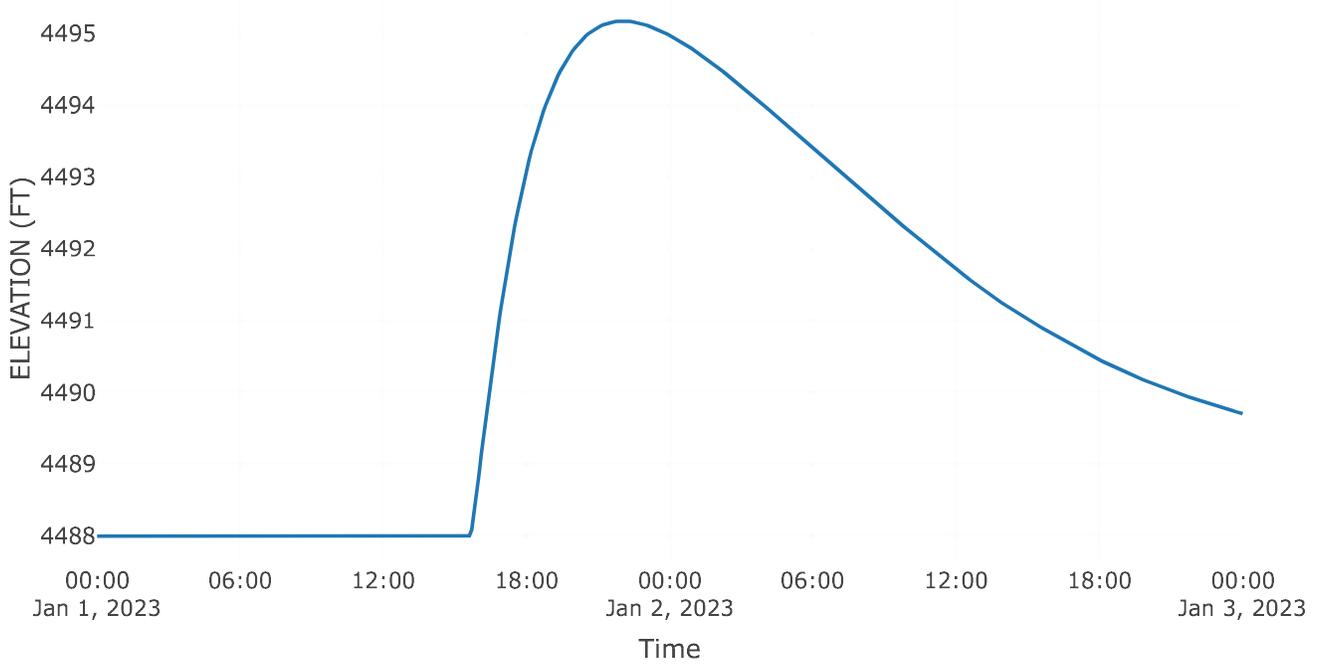
### Spillway 1



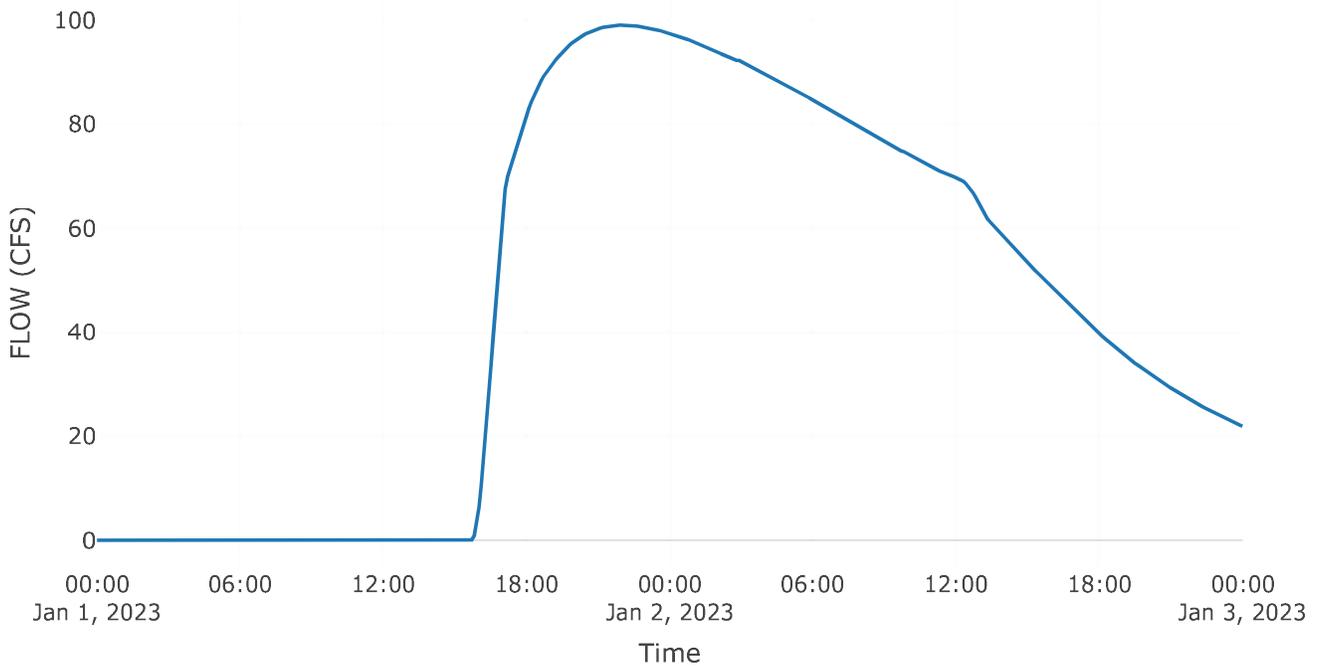
### Outlet 1



### Pool Elevation



### Outflow



## Source: XS ID 72 - EX\_25

**Downstream** : Iron Mountain Basin

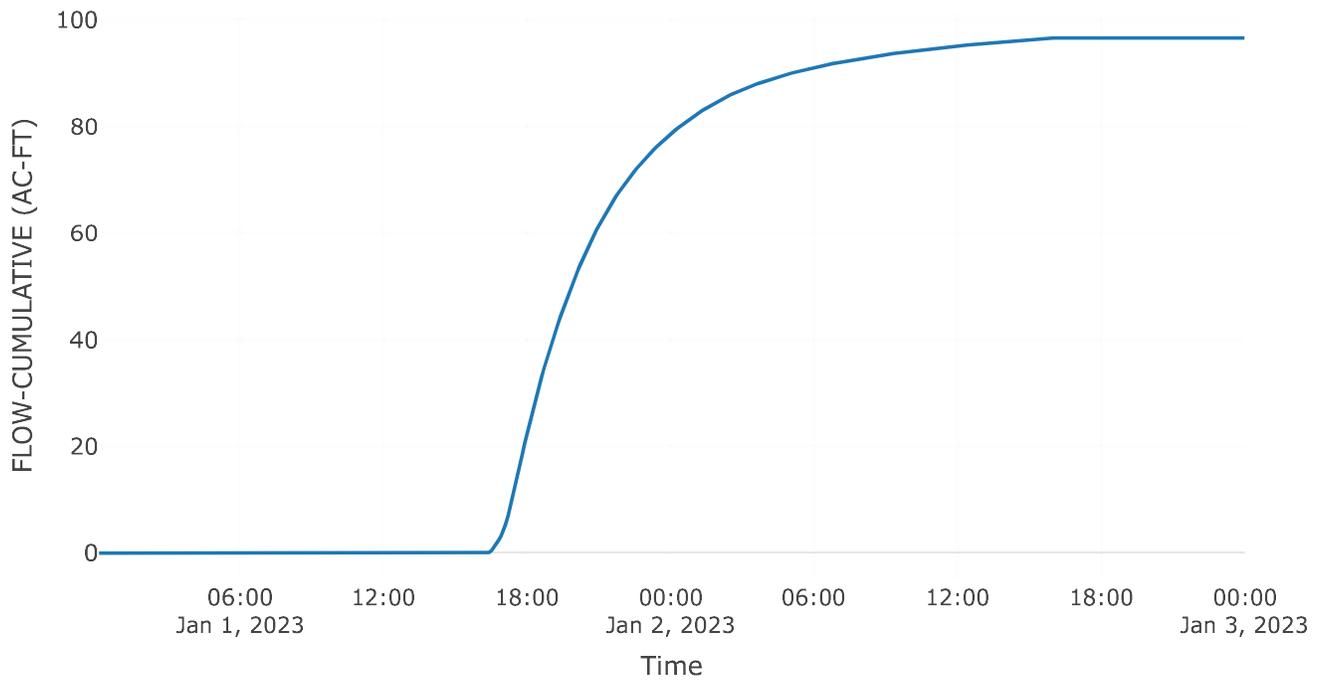
**Flow Method** : Gage Flow

**Flow Gage** : XS ID 72 - EX\_25

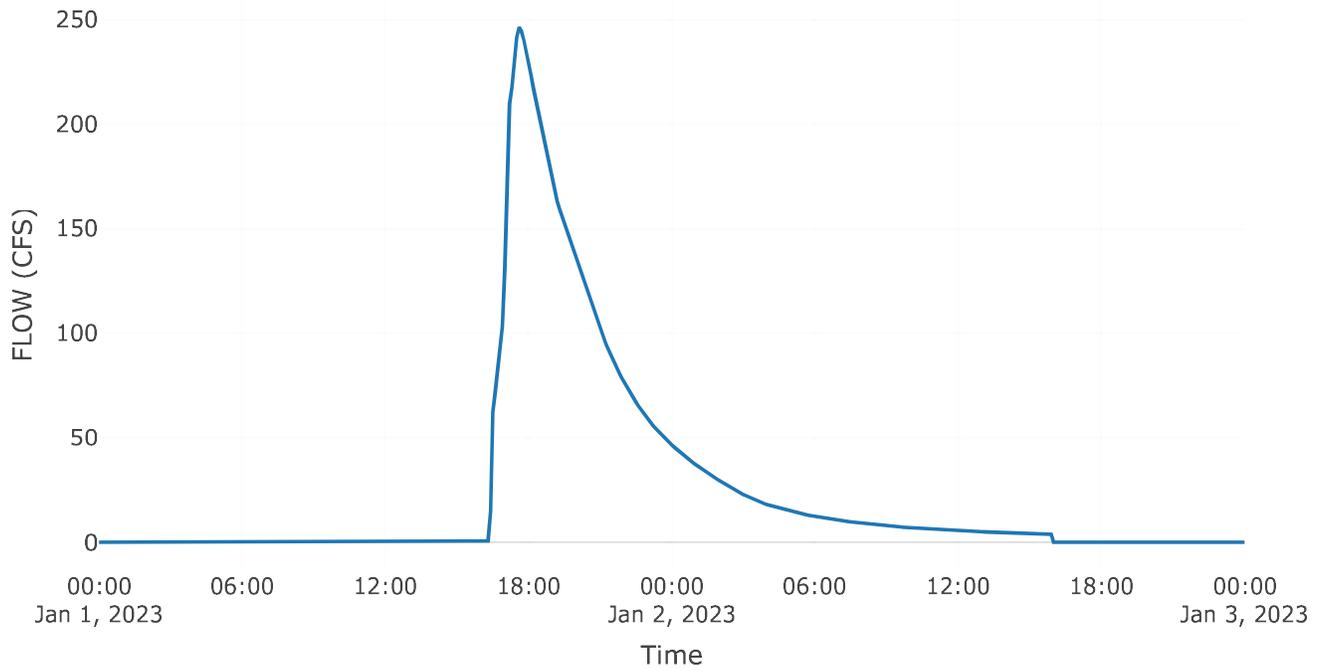
### Results: XS ID 72 - EX\_25

Peak Discharge (CFS)	246.35
Time of Peak Discharge	01Jan2023, 17:36

### Cumulative Outflow



### Outflow

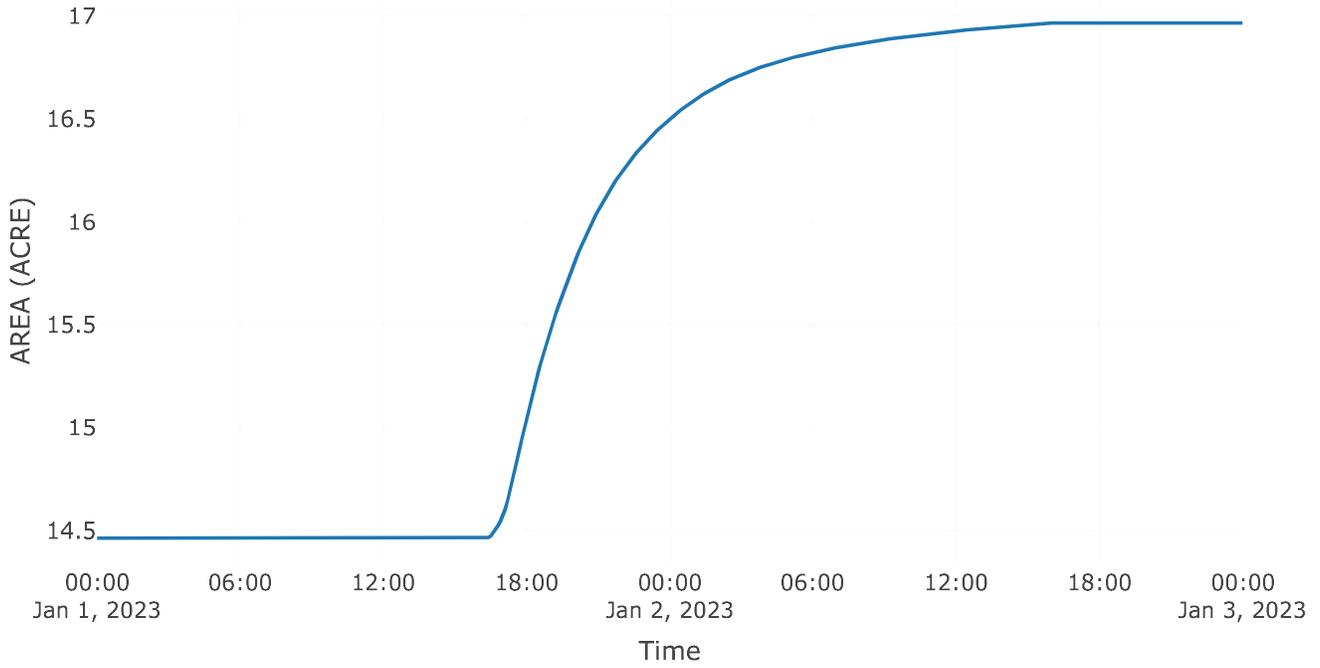


## Reservoir: Iron Mountain Basin

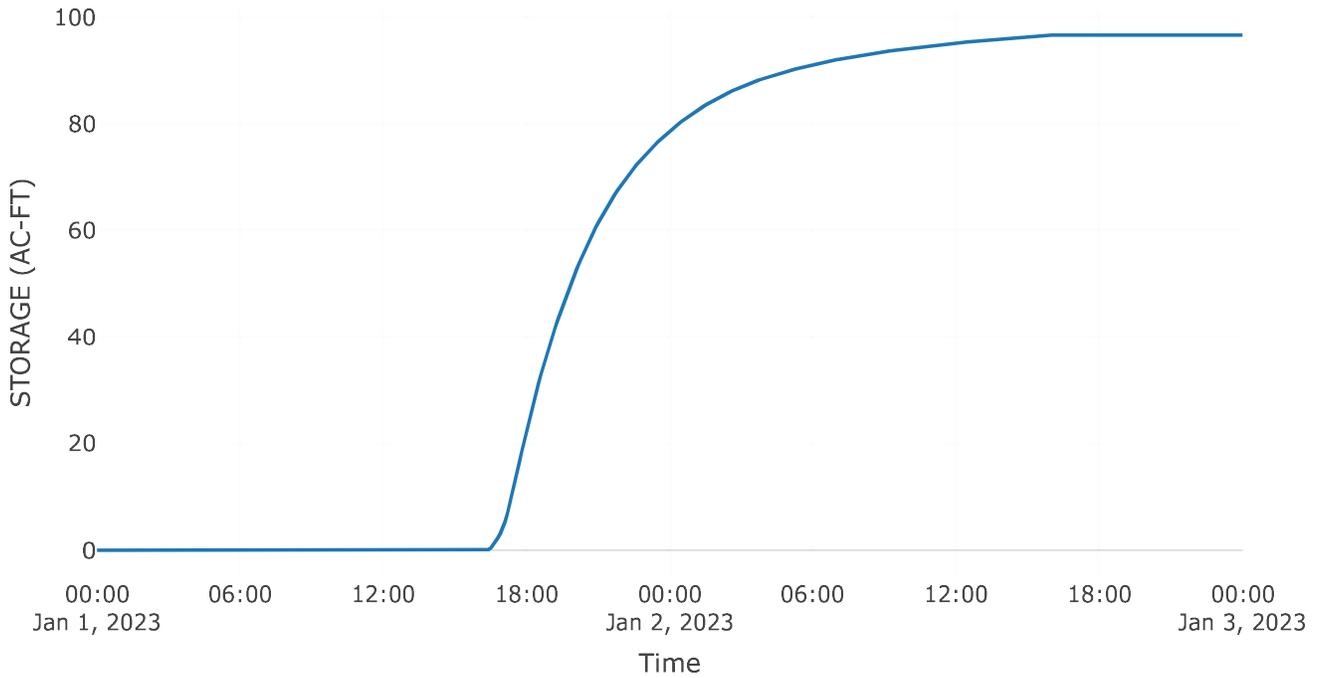
### Results: Iron Mountain Basin

Peak Discharge (CFS)	0
Time of Peak Discharge	31Dec2022, 24:00
Peak Inflow (CFS)	246.35
Time of Peak Inflow	01Jan2023, 17:36
Inflow Volume (AC - FT)	96.64
Maximum Storage (AC - FT)	96.64
Peak Elevation (FT)	4315.67
Discharge Volume (AC - FT)	0

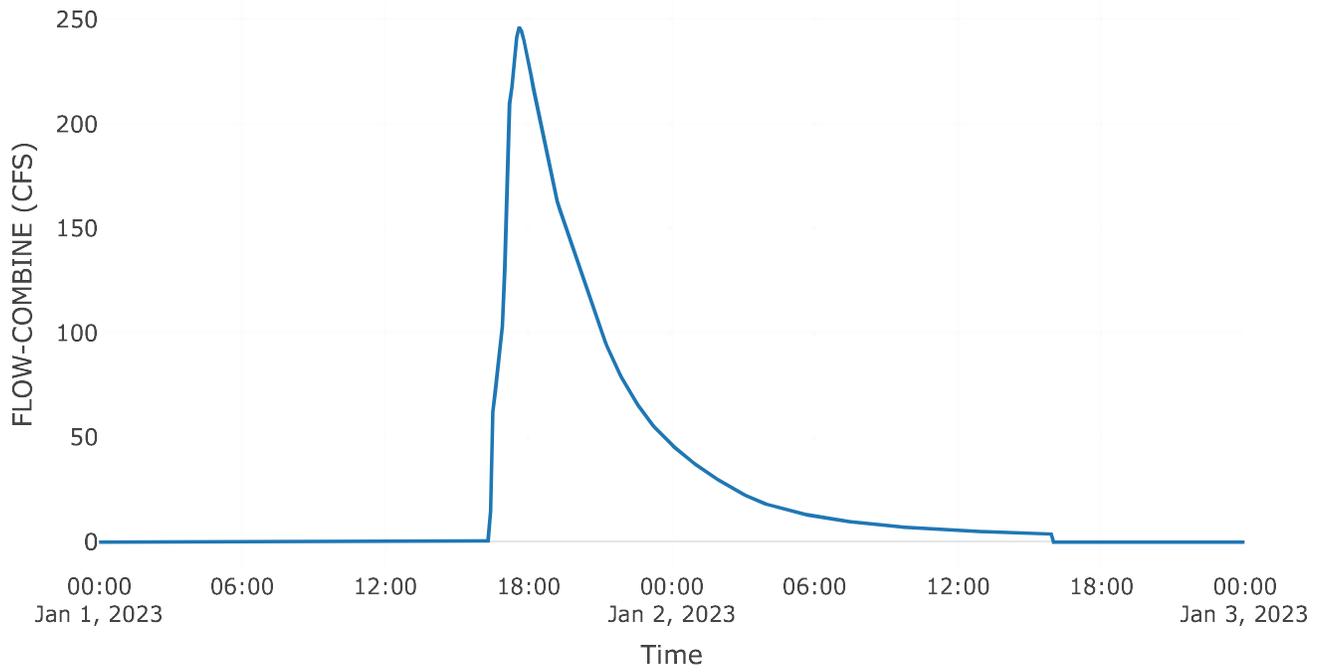
### Reservoir Area



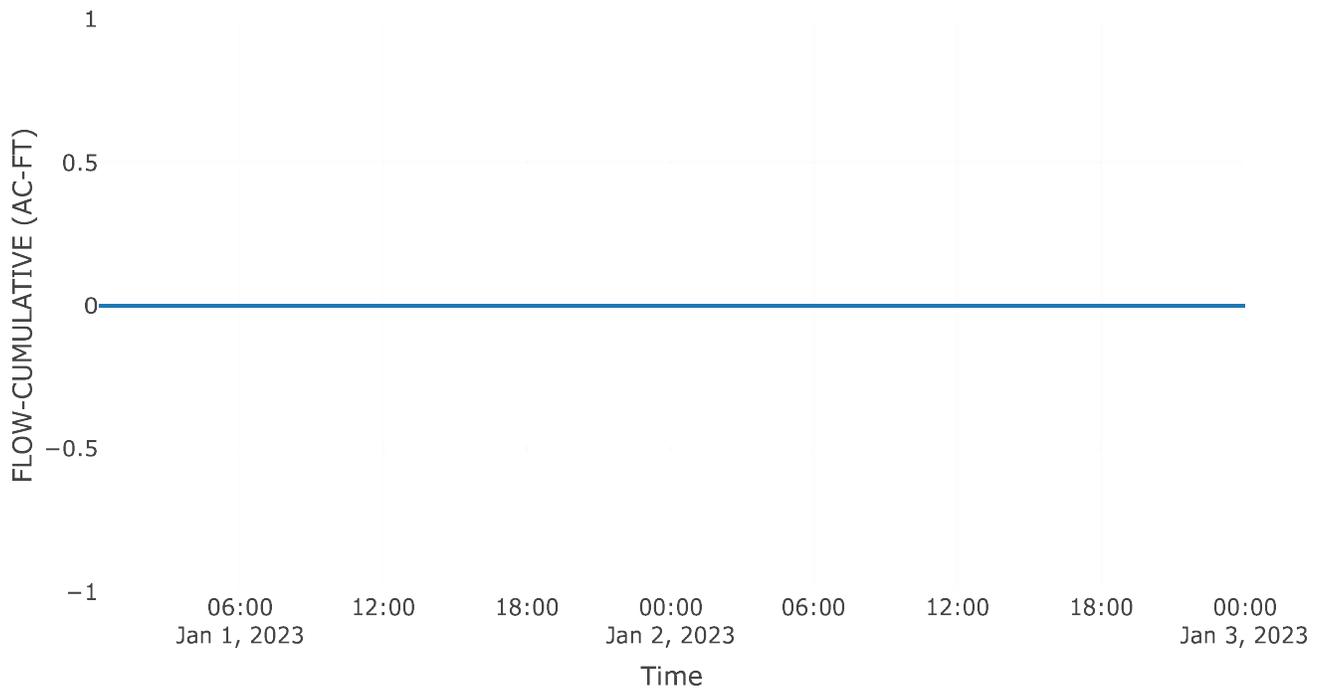
### Storage



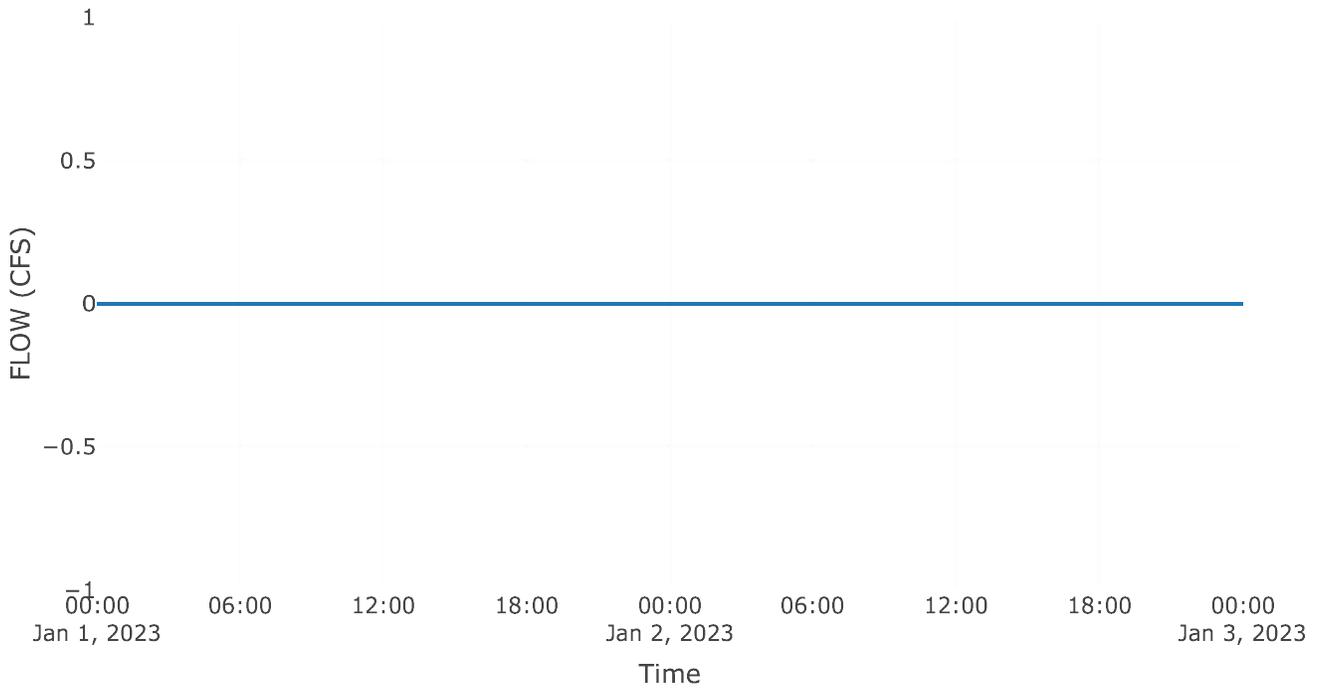
### Combined Inflow



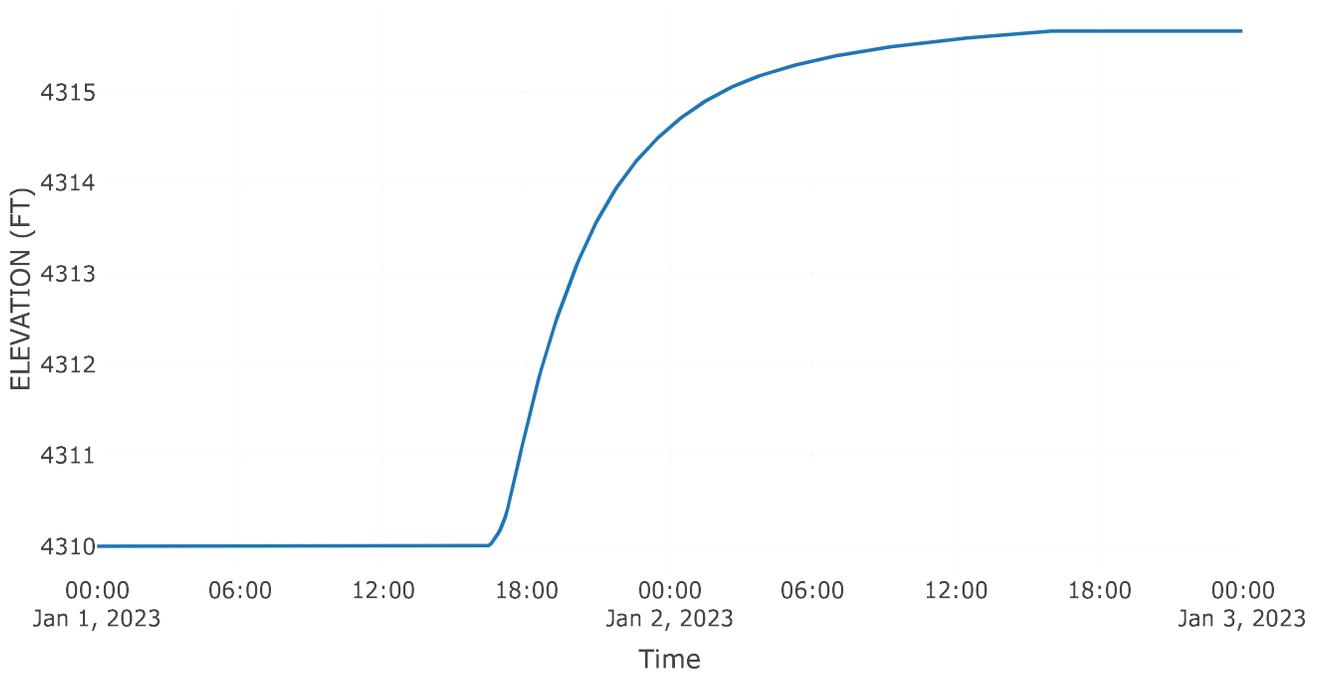
### Cumulative Outflow



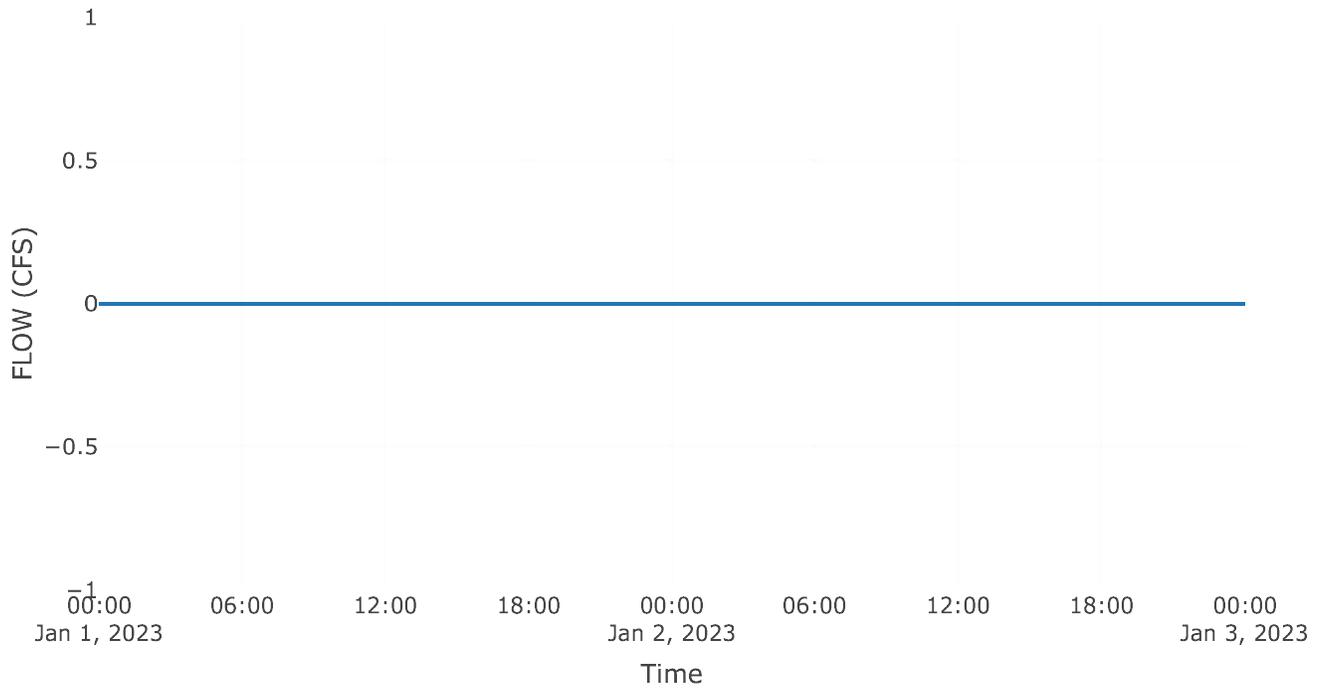
### Spillway 1



### Pool Elevation



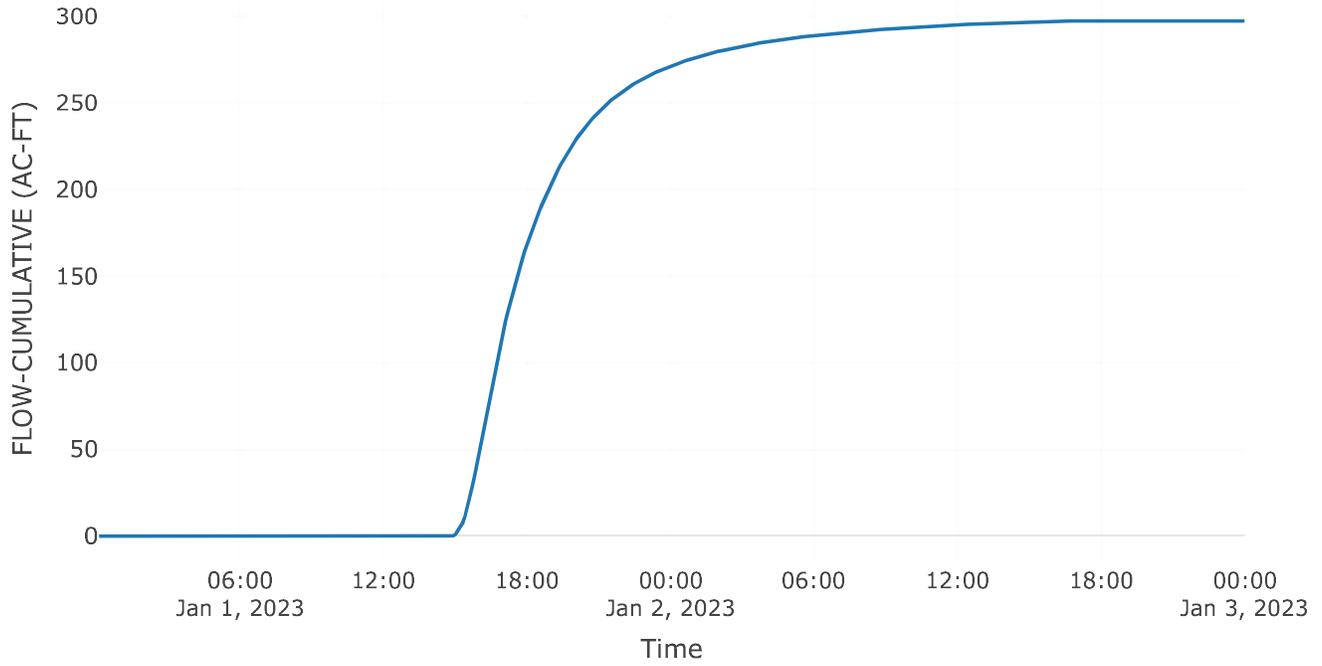
### Outflow



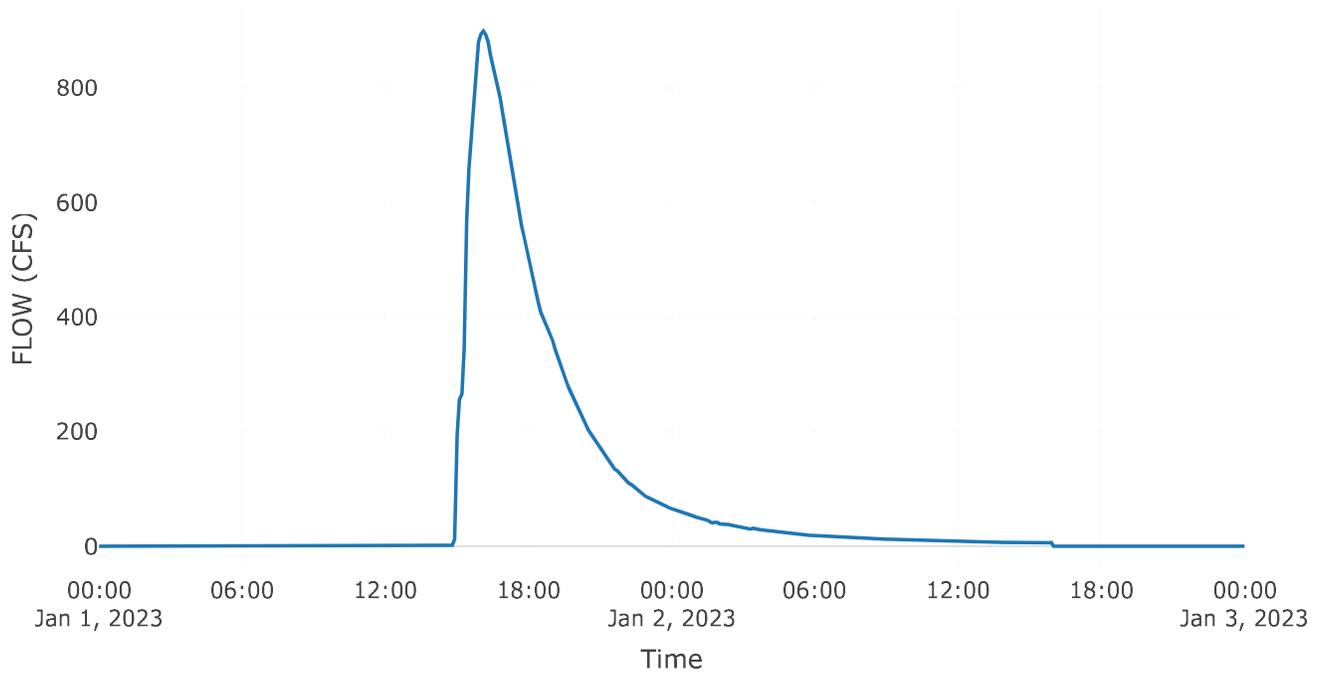
**Source: XS ID 74 - EX\_25****Downstream** : Geurts Basin\_Upper**Flow Method** : Gage Flow**Flow Gage** : XS ID 74 - EX\_25**Results: XS ID 74 - EX\_25**

Peak Discharge (CFS)	898.97
Time of Peak Discharge	01Jan2023, 16:06

### Cumulative Outflow



### Outflow

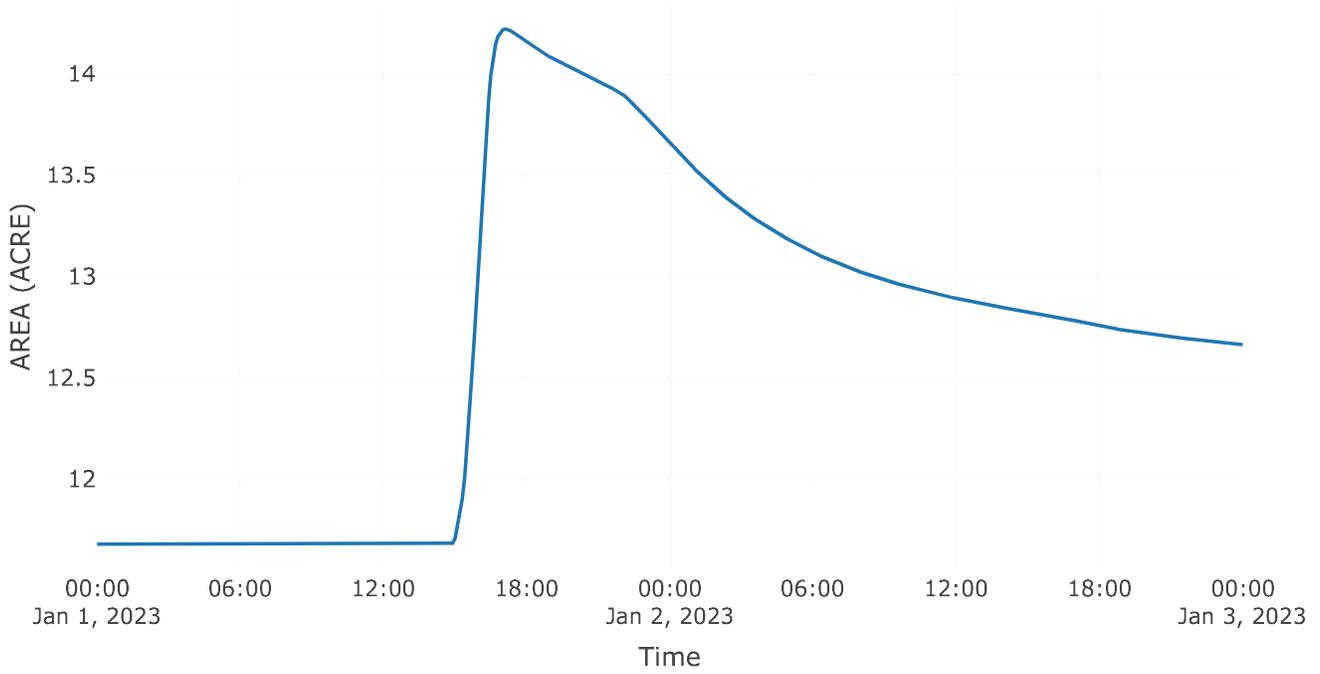


## Reservoir: Geurts Basin\_Upper

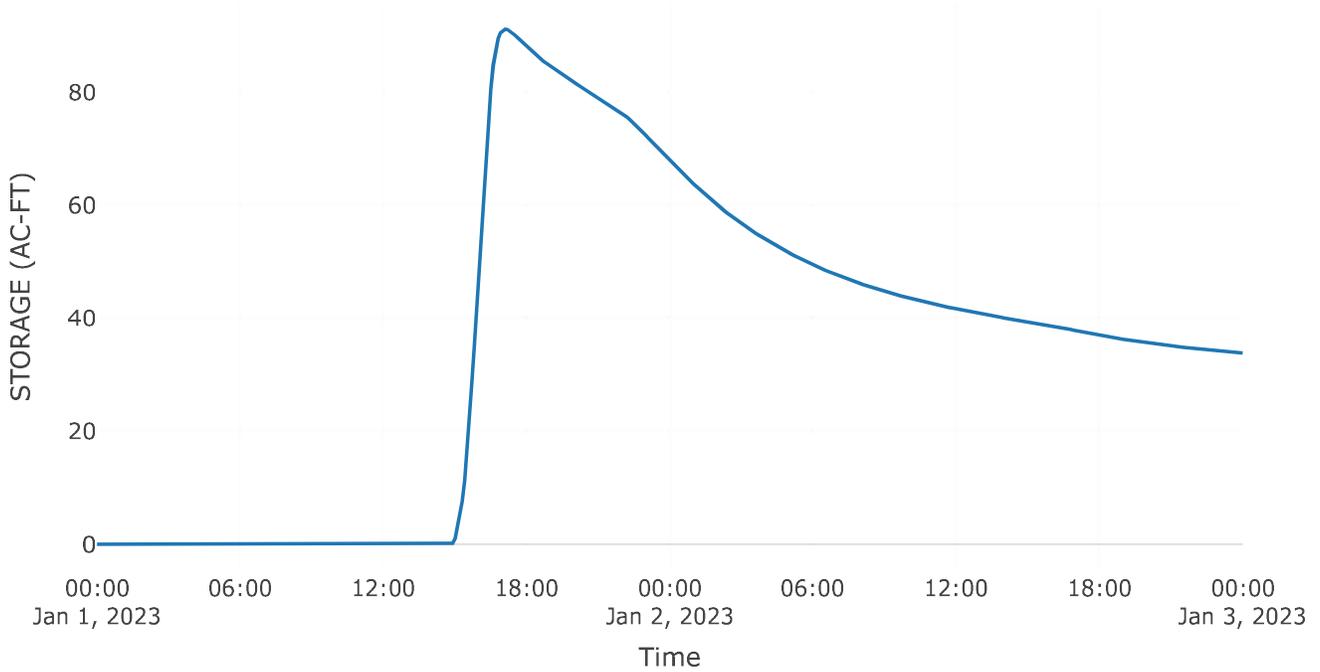
### Results: Geurts Basin\_Upper

Peak Discharge (CFS)	705.89
Time of Peak Discharge	01Jan2023, 17:06
Peak Inflow (CFS)	898.97
Time of Peak Inflow	01Jan2023, 16:06
Inflow Volume (AC - FT)	297.15
Maximum Storage (AC - FT)	91.2
Peak Elevation (FT)	4346.01
Discharge Volume (AC - FT)	263.39

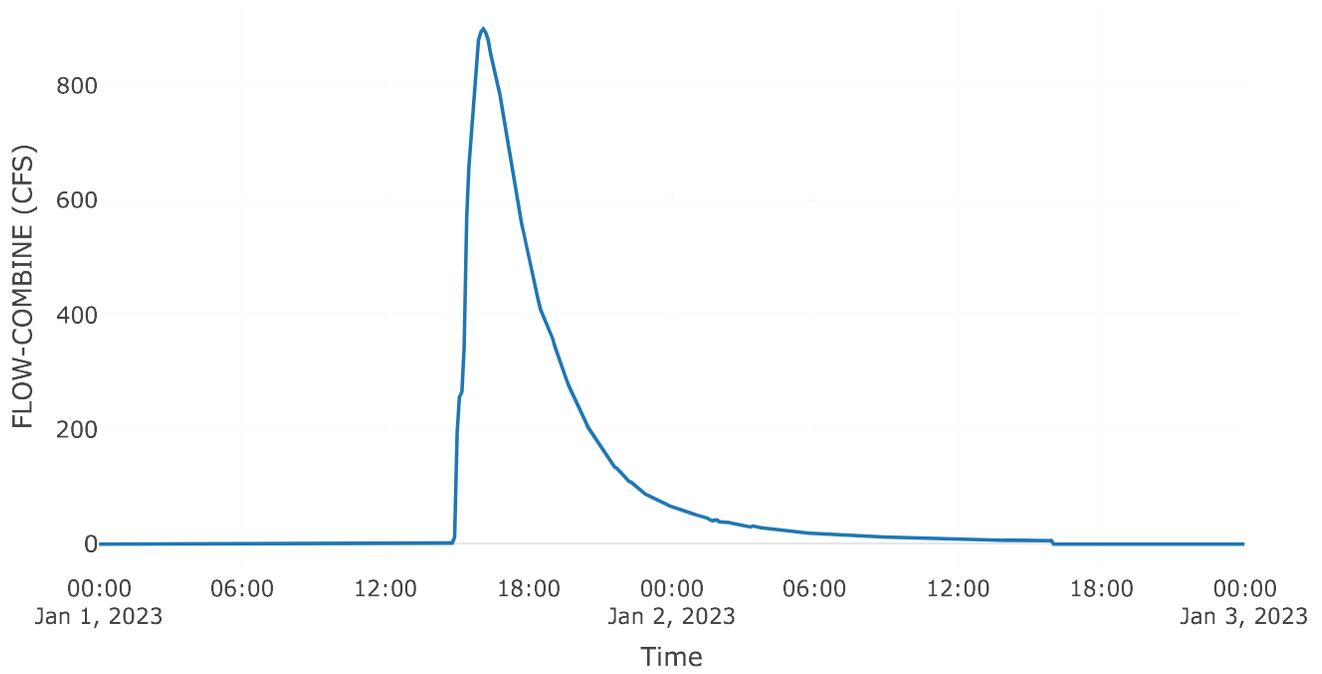
### Reservoir Area



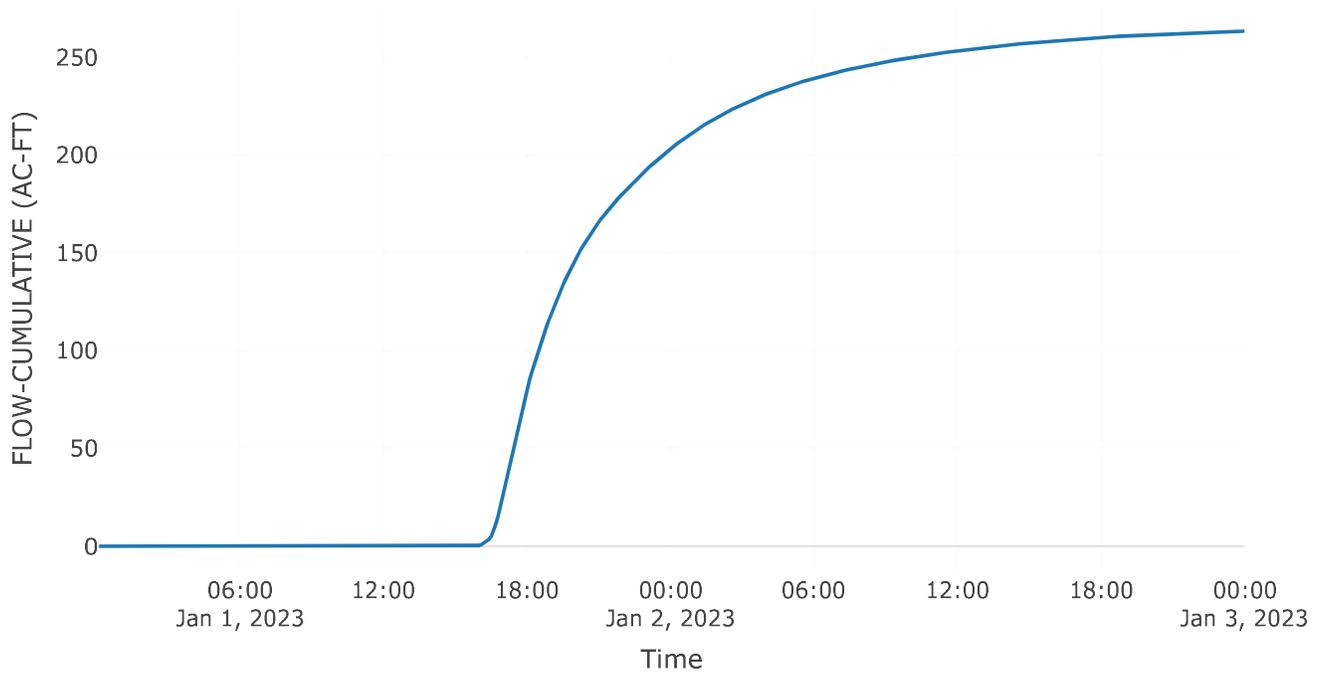
### Storage



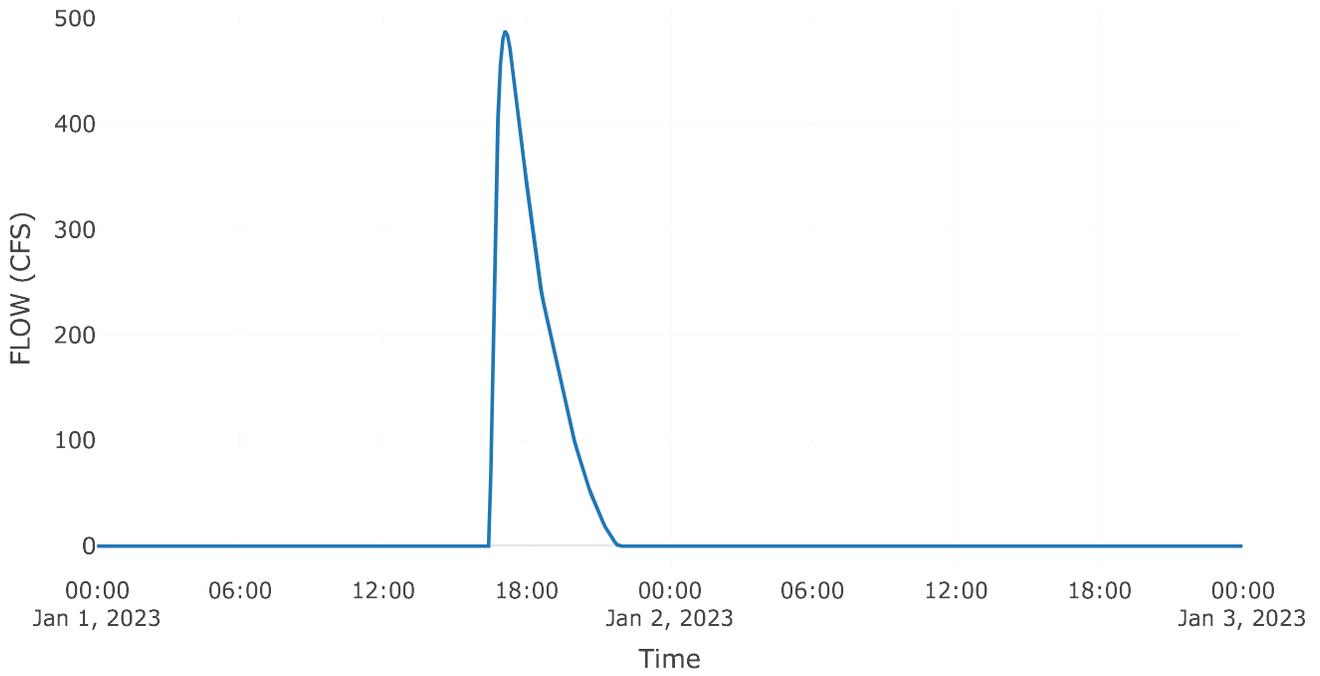
### Combined Inflow



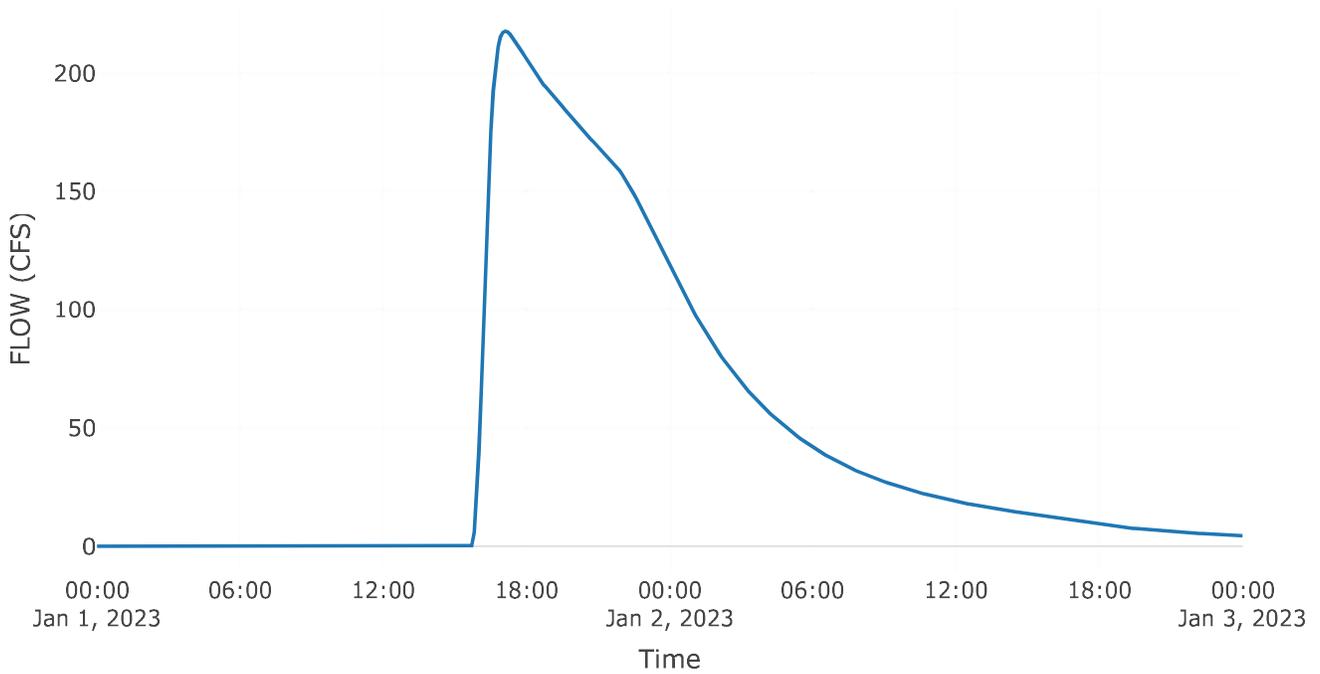
### Cumulative Outflow



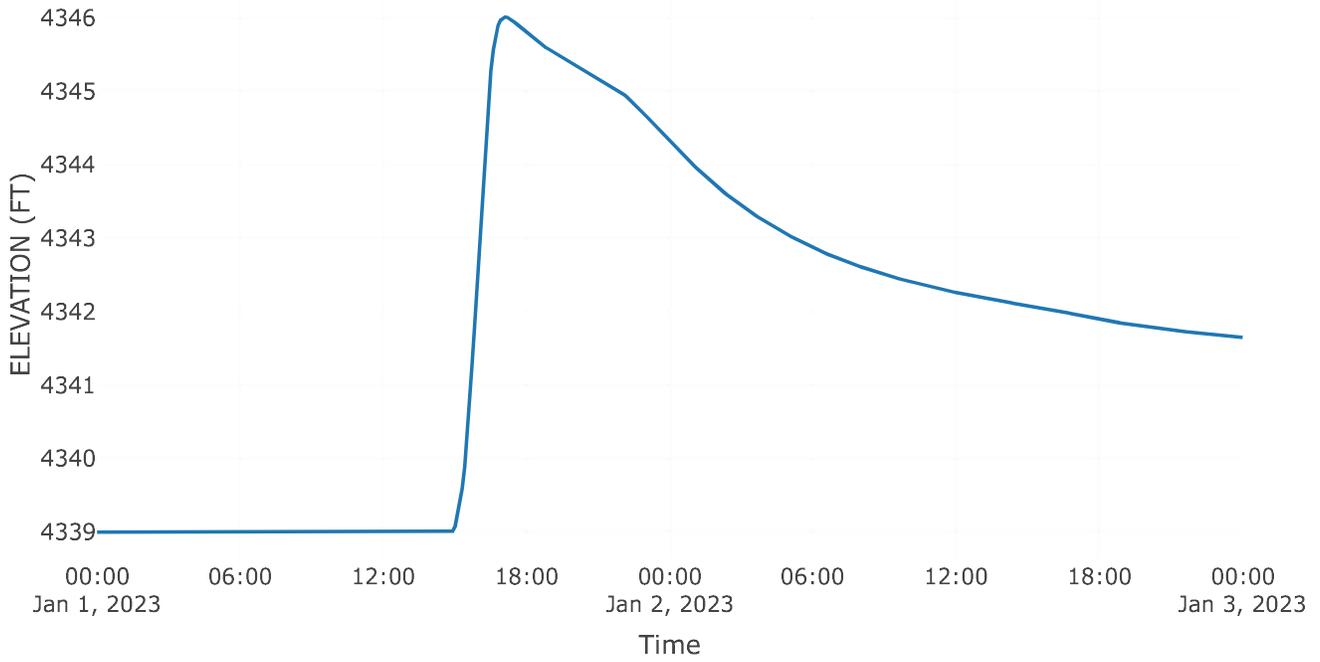
### Spillway 1



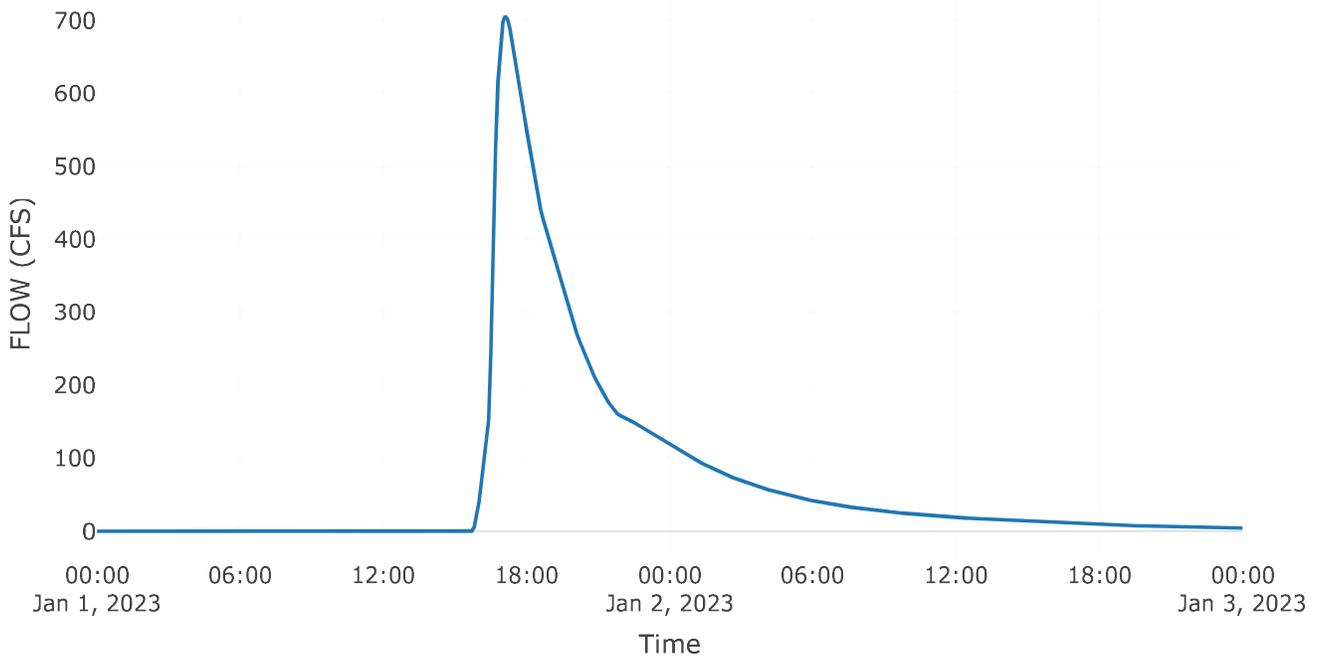
### Outlet 1



### Pool Elevation



### Outflow

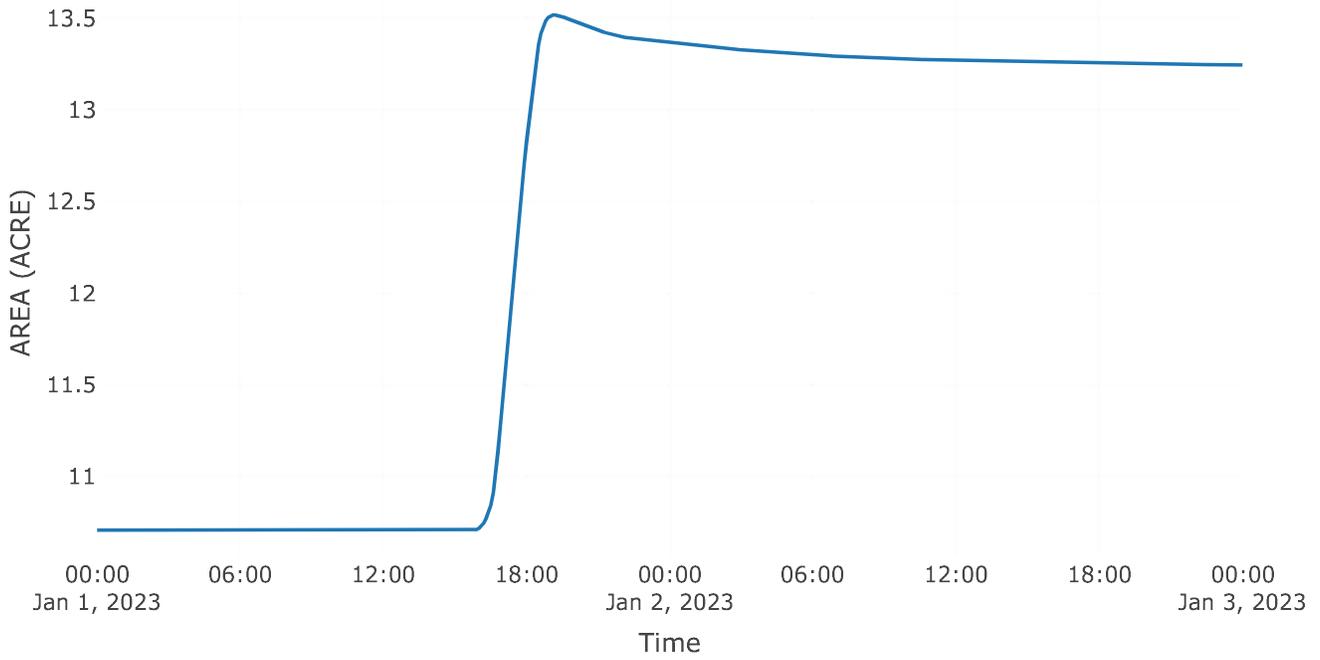


## Reservoir: Geurts Basin\_Lower

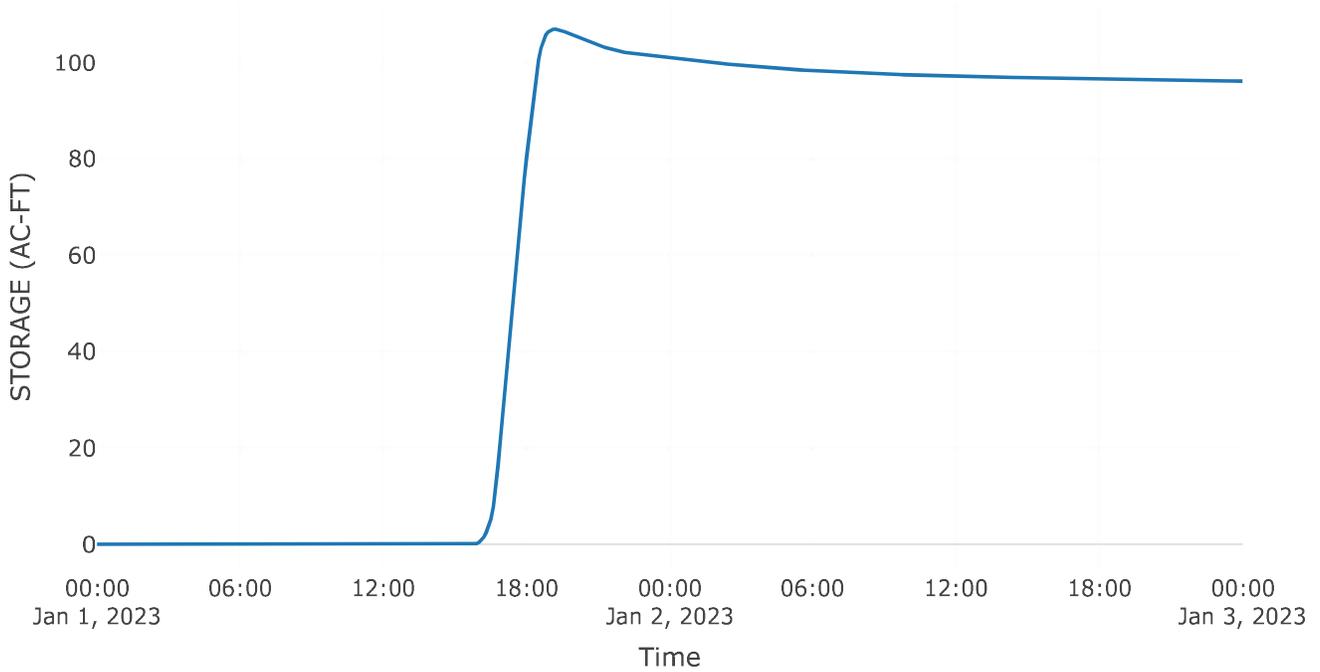
### Results: Geurts Basin\_Lower

Peak Discharge (CFS)	371.74
Time of Peak Discharge	01Jan2023, 19:12
Peak Inflow (CFS)	705.89
Time of Peak Inflow	01Jan2023, 17:06
Inflow Volume (AC - FT)	263.39
Maximum Storage (AC - FT)	107.03
Peak Elevation (FT)	4338.84
Discharge Volume (AC - FT)	167.21

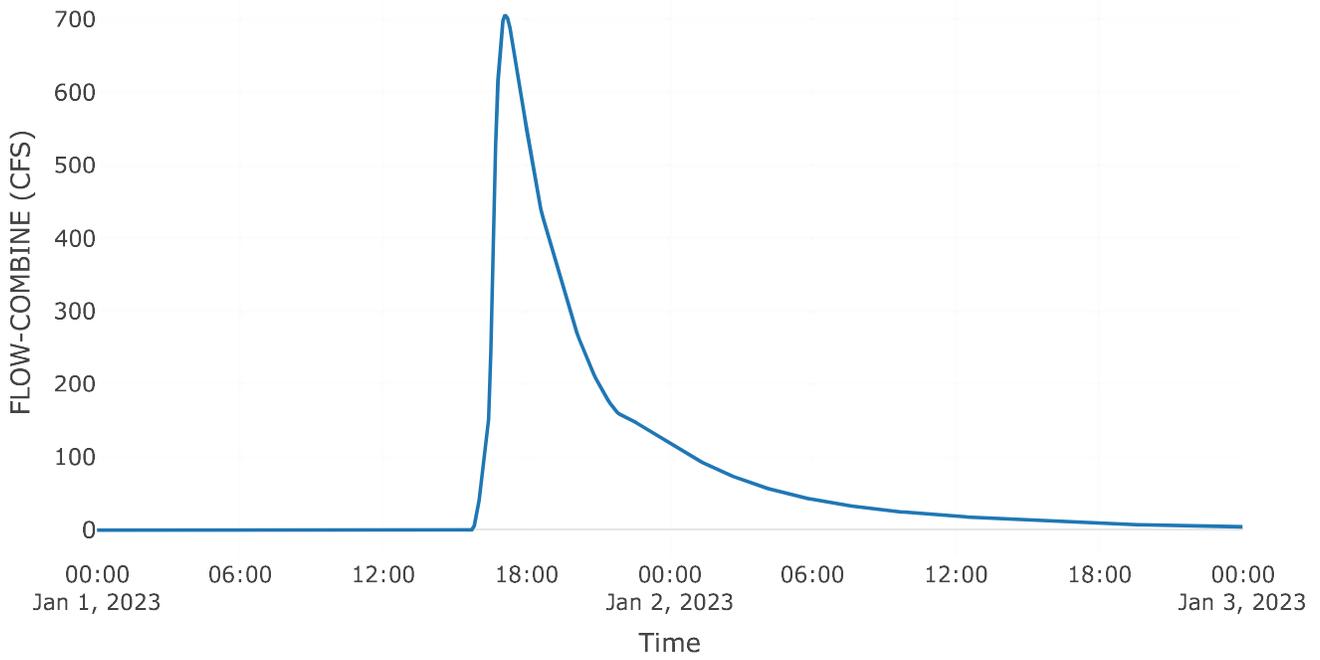
### Reservoir Area



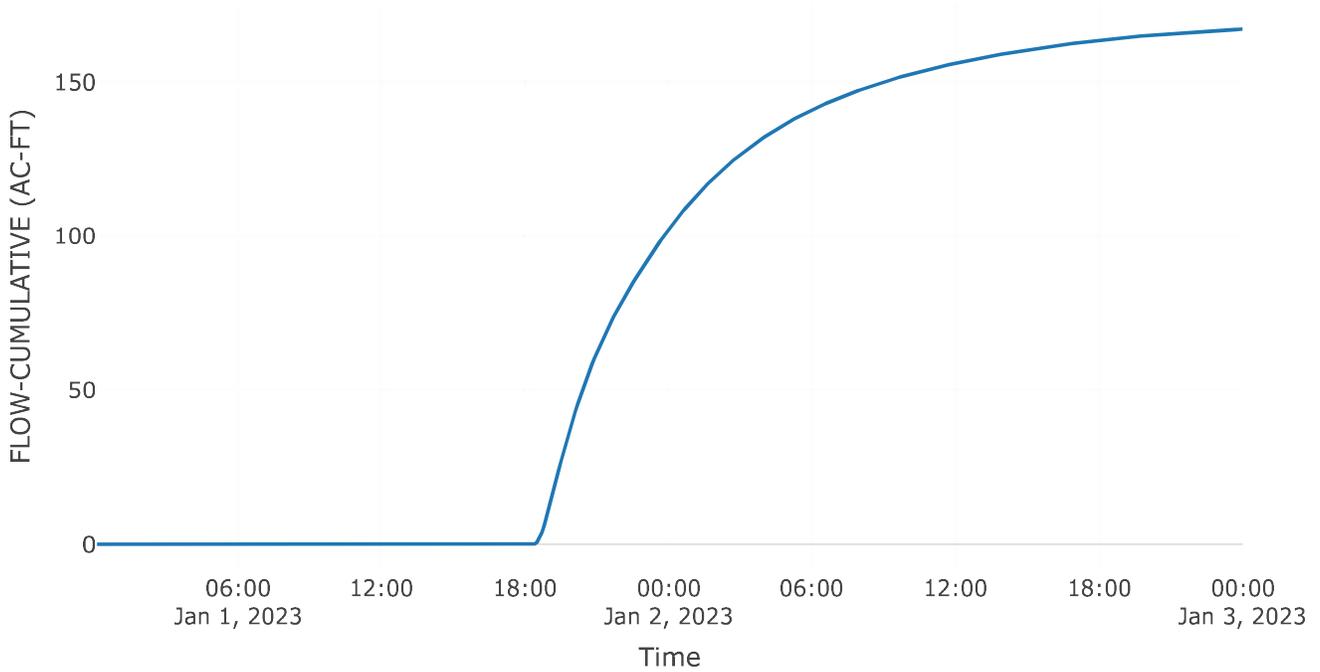
### Storage



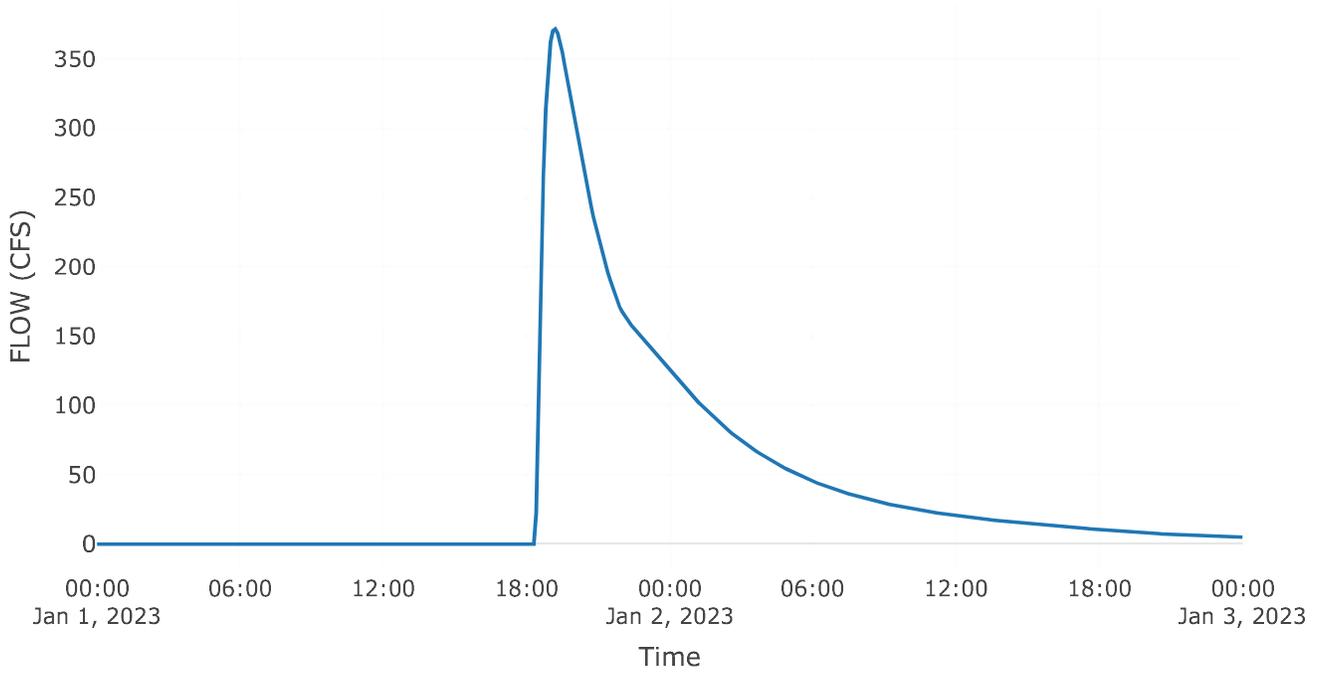
### Combined Inflow



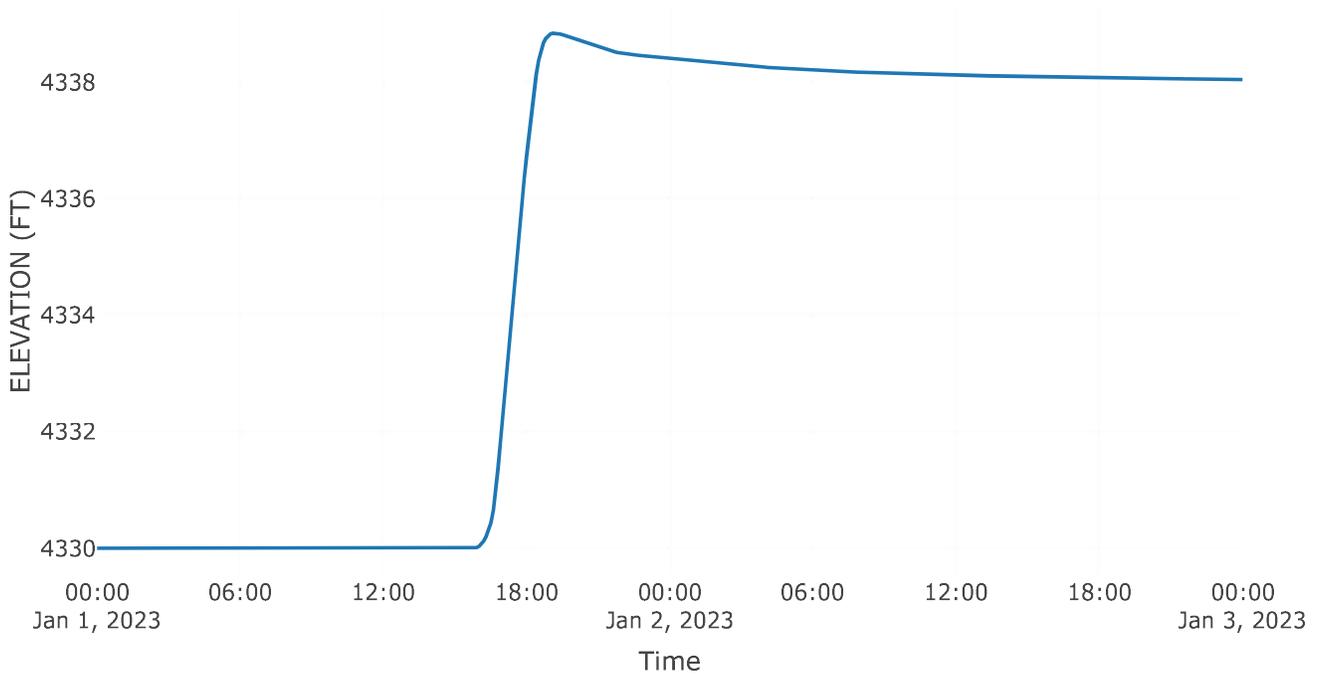
### Cumulative Outflow



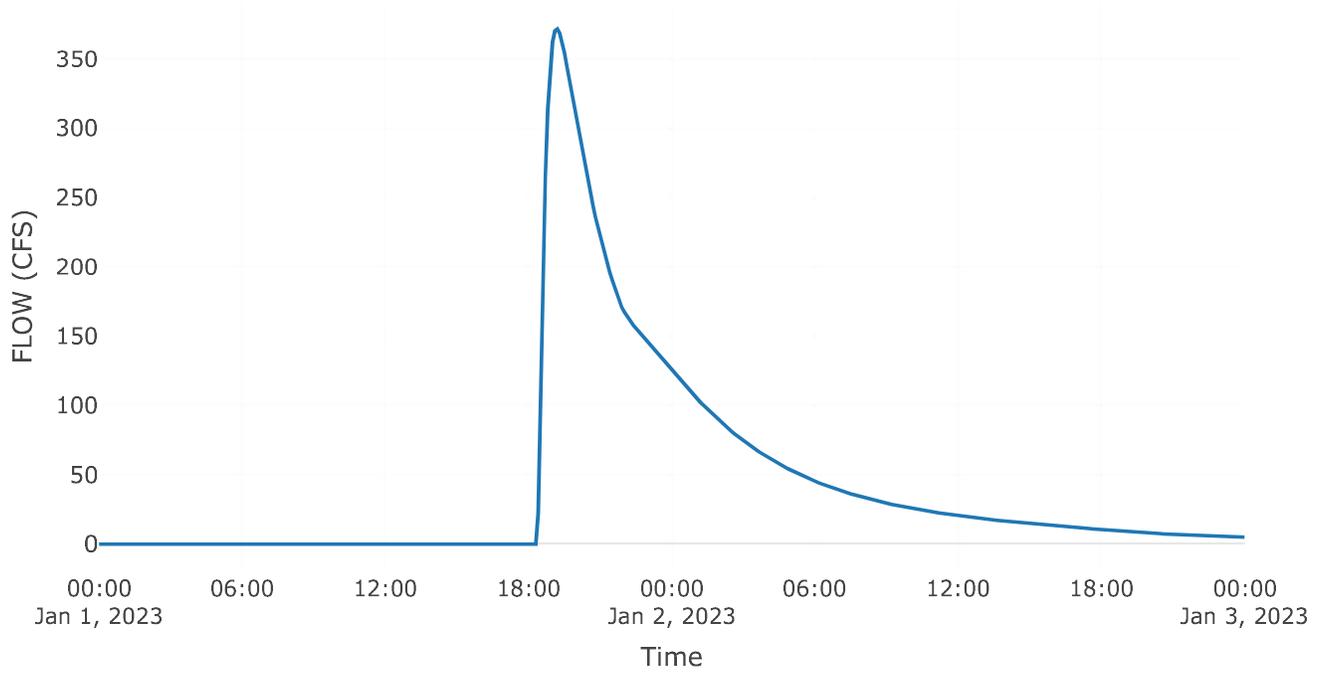
### Spillway 1



### Pool Elevation



### Outflow



**Project:** Detention Hydrographs

**Simulation Run:** 100 Year Run

**Simulation Start:** 31 December 2022, 24:00

**Simulation End:** 2 January 2023, 24:00

**HMS Version:** 4.12

**Executed:** 11 June 2024, 18:18

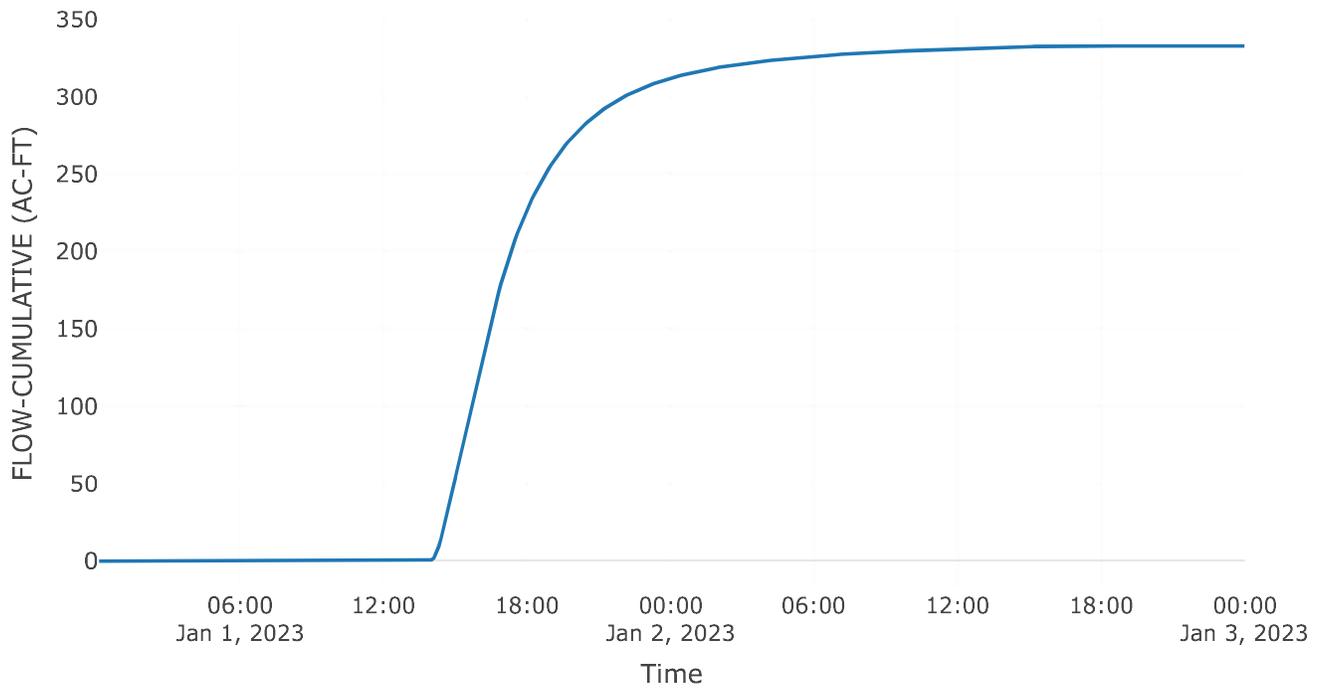
## Global Results Summary

Hydrologic Element	Drainage Area (MI <sup>2</sup> )	Peak Discharge (CFS)	Time of Peak	Volume ( )
XS ID 4 - EX_100	Not specified	853.88	01Jan2023, 15:42	Not specified
Quater Horse Basin	Not specified	65.37	01Jan2023, 23:24	Not specified
XS ID 79 - EX_100	Not specified	833.61	01Jan2023, 15:18	Not specified
Apache Basin	Not specified	308.57	01Jan2023, 17:48	Not specified
XS ID 17 - EX_100	Not specified	1584.14	01Jan2023, 14:30	Not specified
Shawnee Basin	Not specified	717.55	01Jan2023, 17:24	Not specified
XS ID 72 - EX_100	Not specified	701.48	01Jan2023, 15:24	Not specified
Iron Mountain Basin	Not specified	60.14	01Jan2023, 23:36	Not specified
XS ID 74 - EX_100	Not specified	1929.73	01Jan2023, 14:48	Not specified
Geurts Basin_Upper	Not specified	1868.56	01Jan2023, 15:06	Not specified
Geurts Basin_Lower	Not specified	1689.59	01Jan2023, 15:48	Not specified

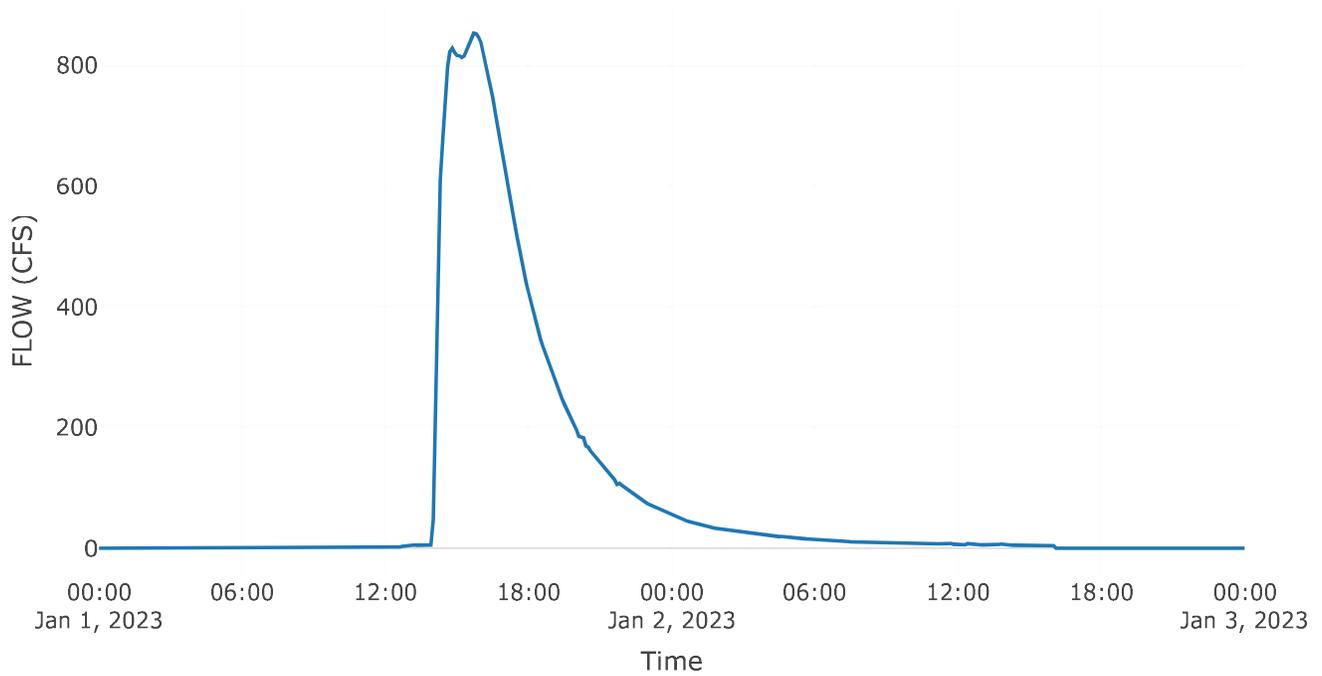
**Source: XS ID 4 - EX\_100****Downstream** : Quater Horse Basin**Flow Method** : Gage Flow**Flow Gage** : XS ID 4 - EX\_100**Results: XS ID 4 - EX\_100**

Peak Discharge (CFS)	853.88
Time of Peak Discharge	01Jan2023, 15:42

### Cumulative Outflow



### Outflow

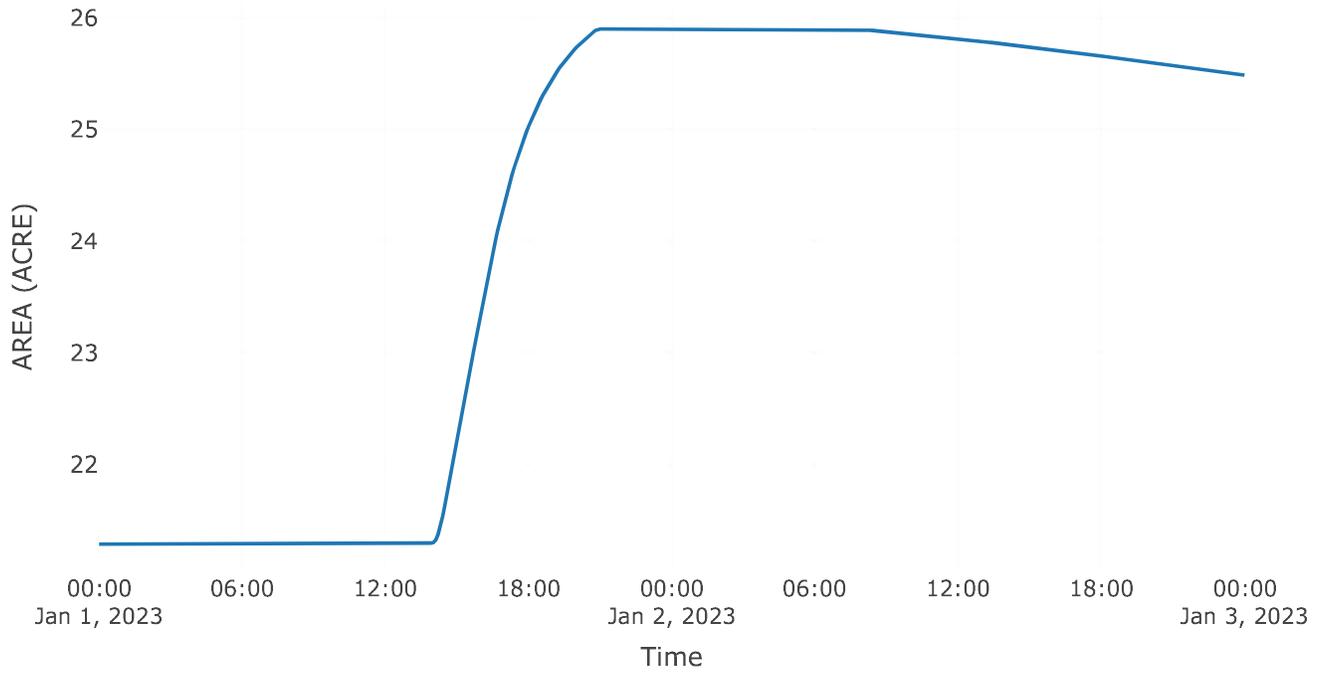


## Reservoir: Quater Horse Basin

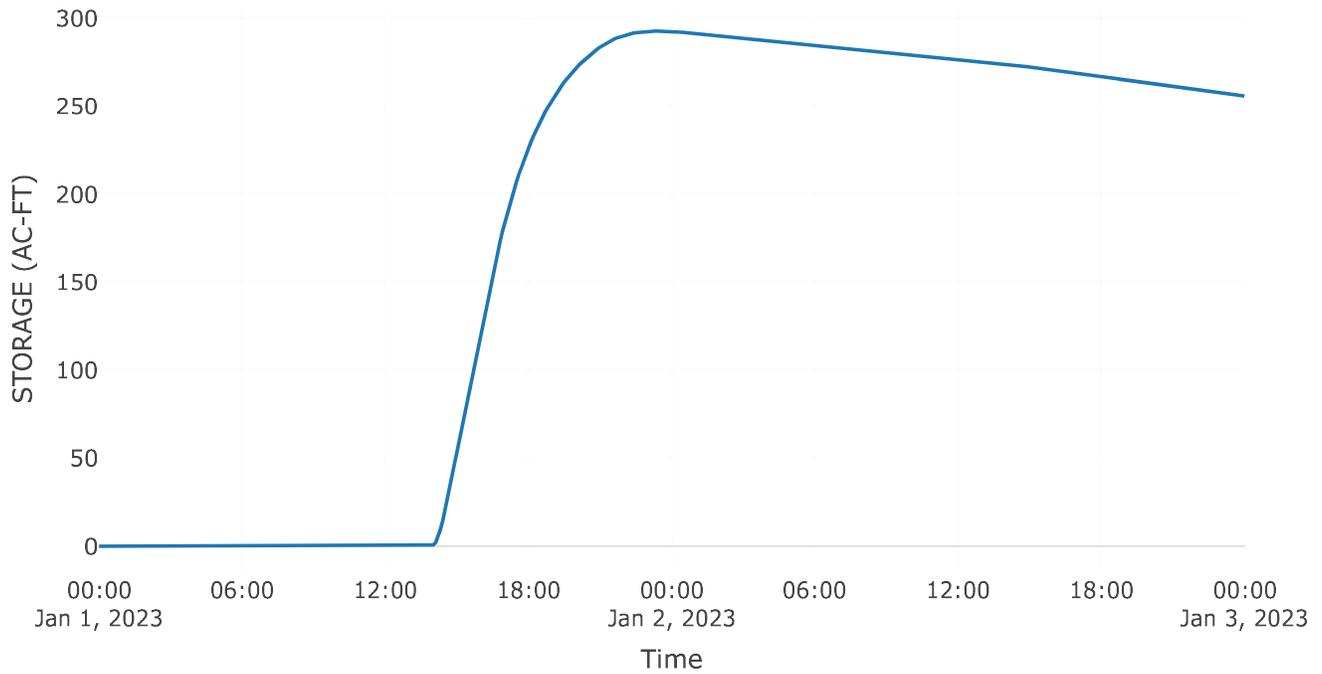
### Results: Quater Horse Basin

Peak Discharge (CFS)	65.37
Time of Peak Discharge	01Jan2023, 23:24
Peak Inflow (CFS)	853.88
Time of Peak Inflow	01Jan2023, 15:42
Inflow Volume (AC - FT)	332.83
Maximum Storage (AC - FT)	292.89
Peak Elevation (FT)	4397.39
Discharge Volume (AC - FT)	76.84

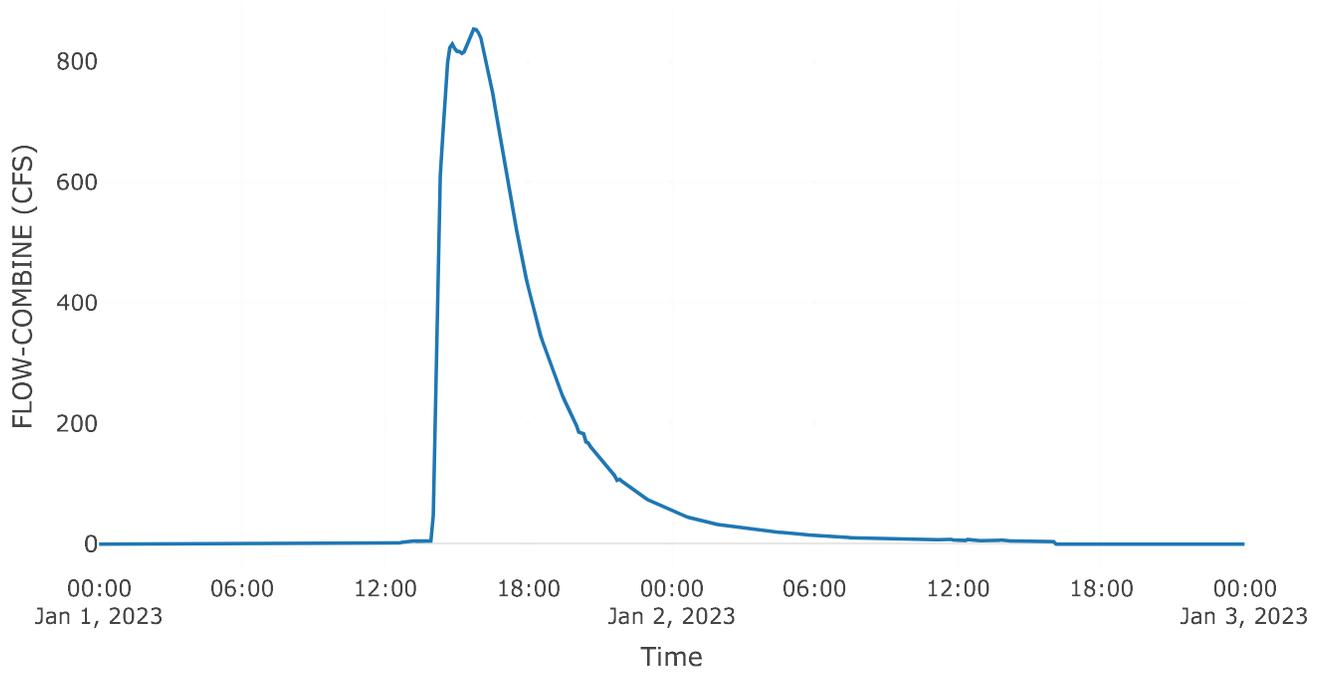
### Reservoir Area



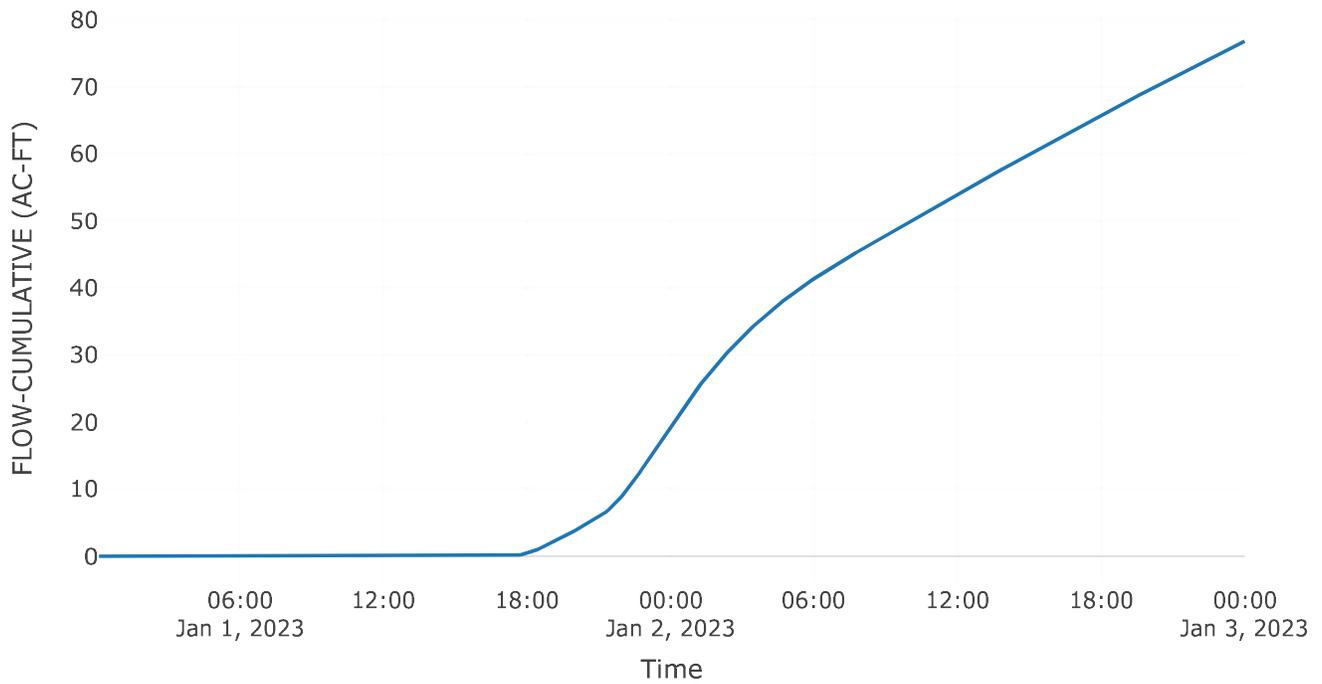
### Storage



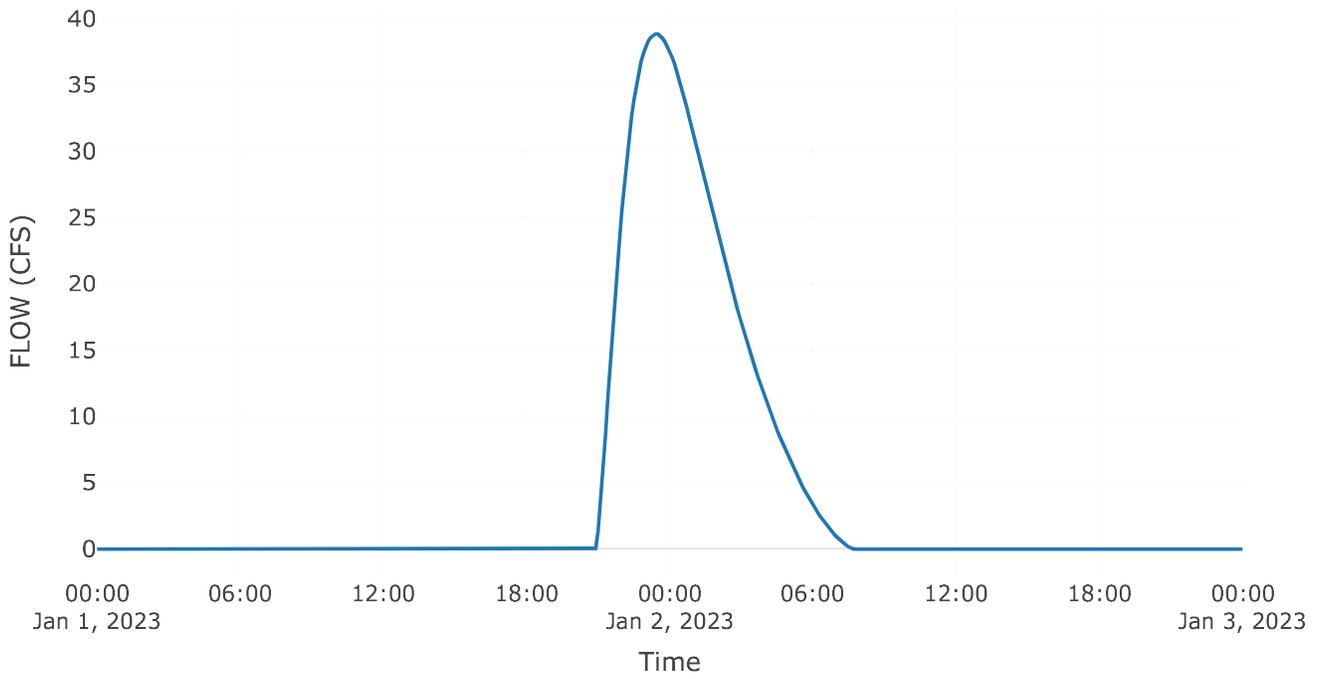
### Combined Inflow



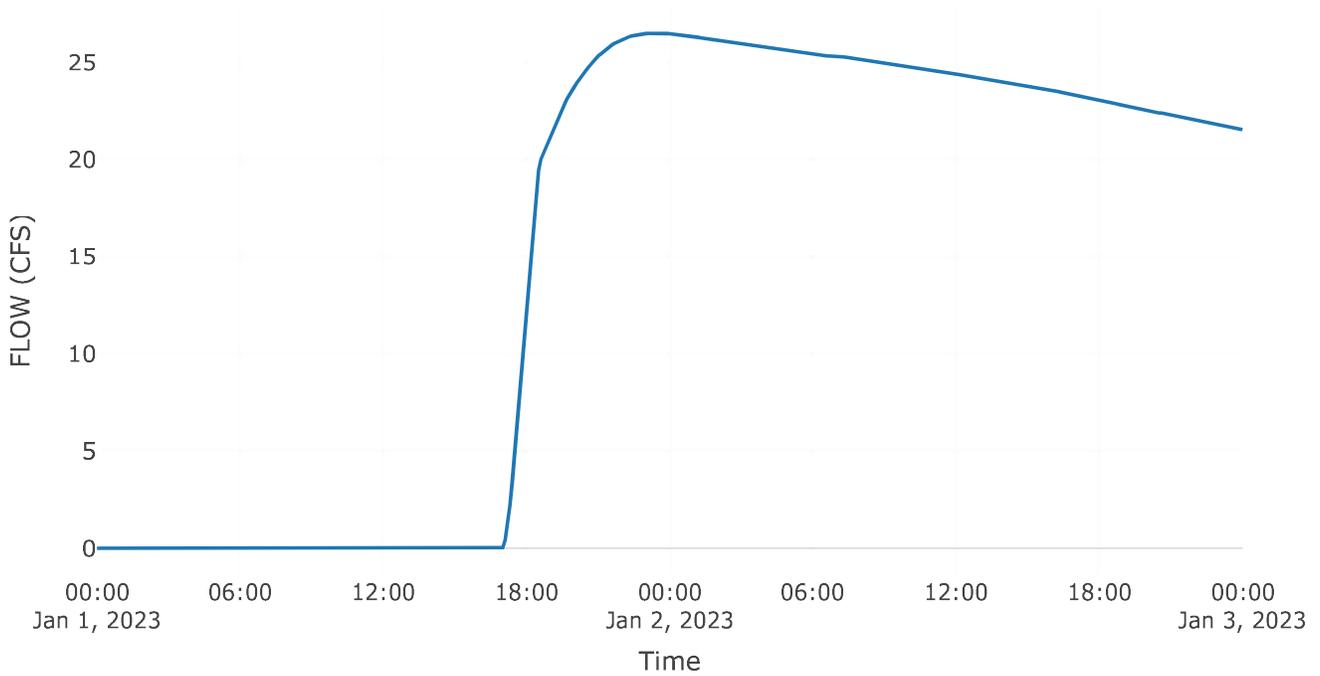
### Cumulative Outflow



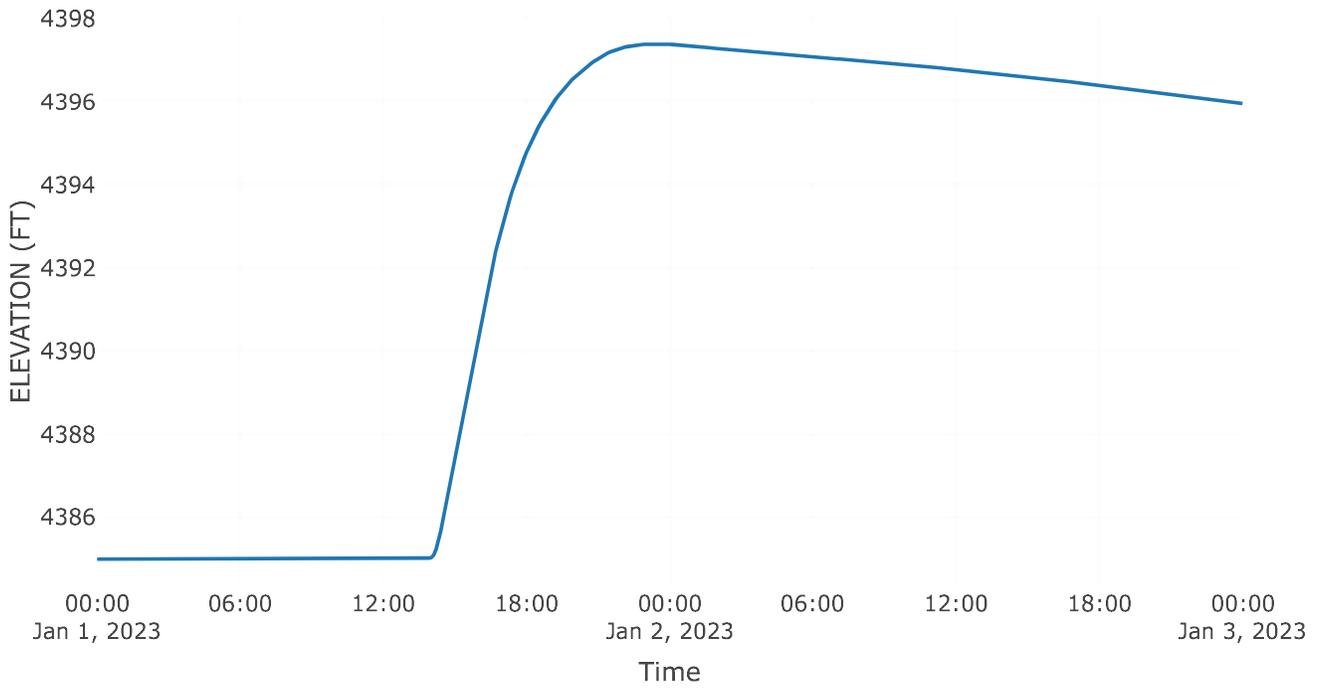
### Spillway 1



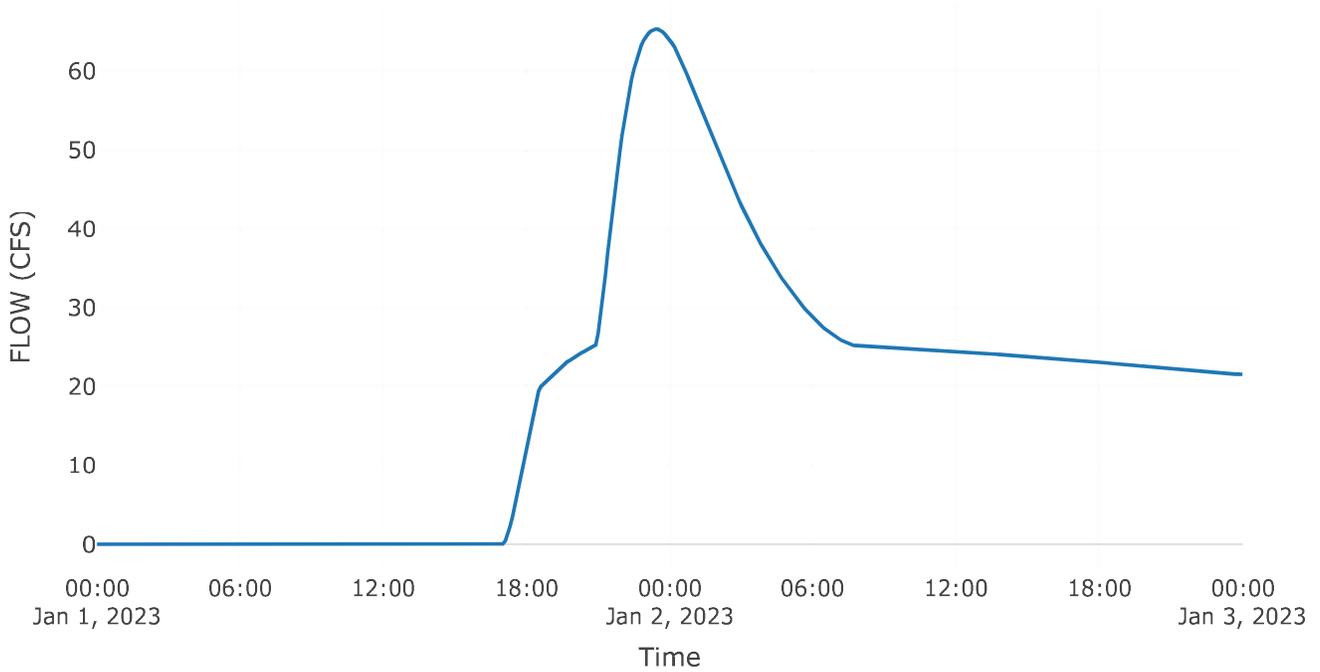
### Outlet 1



### Pool Elevation



### Outflow



**Source: XS ID 79 - EX\_100****Downstream** : Apache Basin**Flow Method** : Gage Flow**Flow Gage** : XS ID 79 - EX\_100**Results: XS ID 79 - EX\_100**

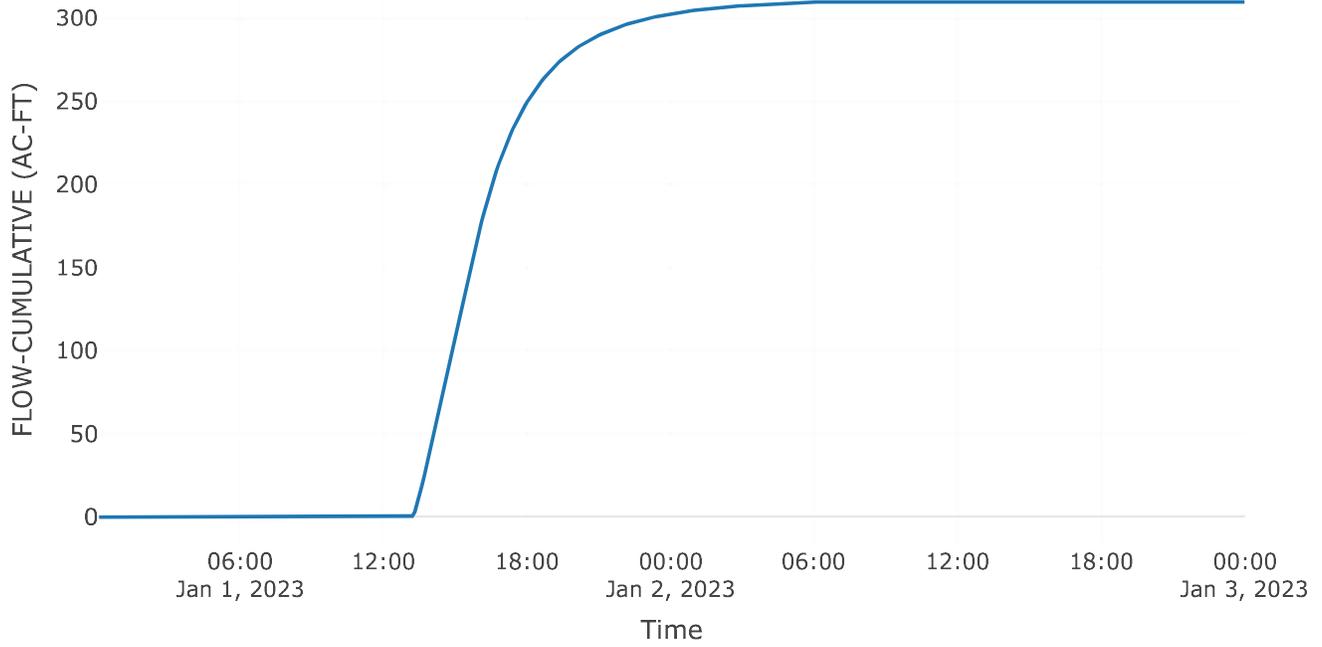
Peak Discharge (CFS)

833.61

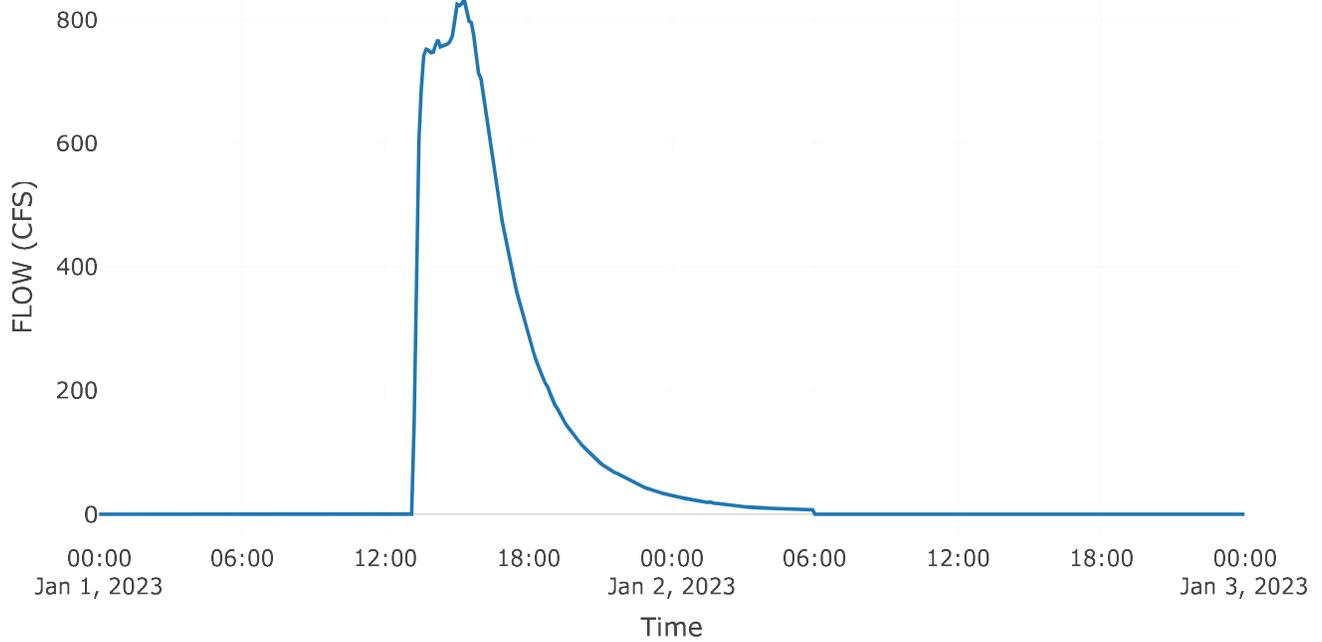
Time of Peak Discharge

01Jan2023, 15:18

### Cumulative Outflow



### Outflow

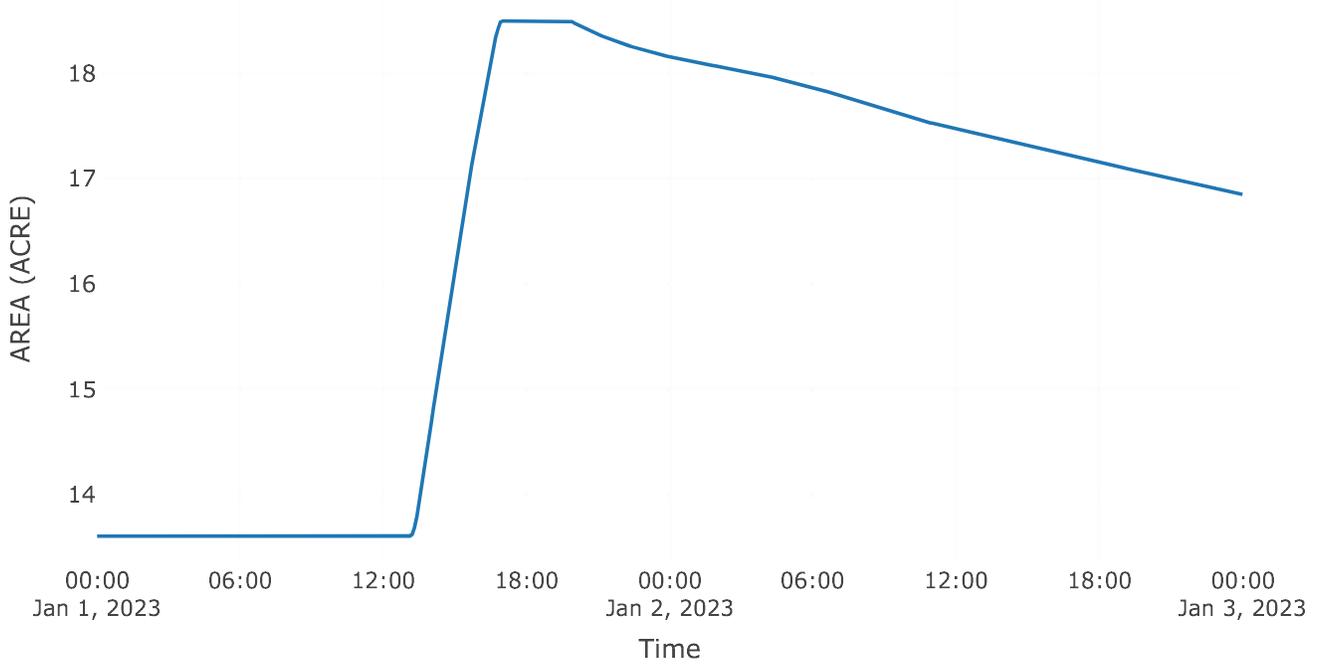


## Reservoir: Apache Basin

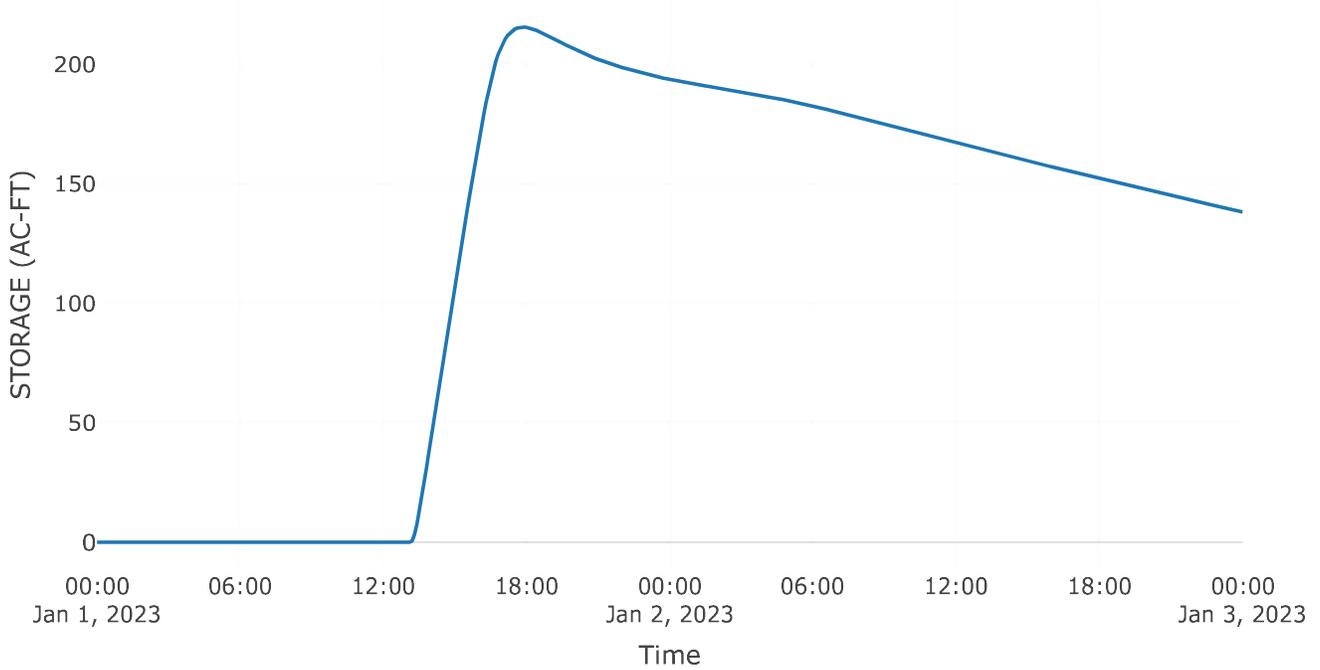
### Results: Apache Basin

Peak Discharge (CFS)	308.57
Time of Peak Discharge	01Jan2023, 17:48
Peak Inflow (CFS)	833.61
Time of Peak Inflow	01Jan2023, 15:18
Inflow Volume (AC - FT)	309.63
Maximum Storage (AC - FT)	215.7
Peak Elevation (FT)	4538.46
Discharge Volume (AC - FT)	171.29

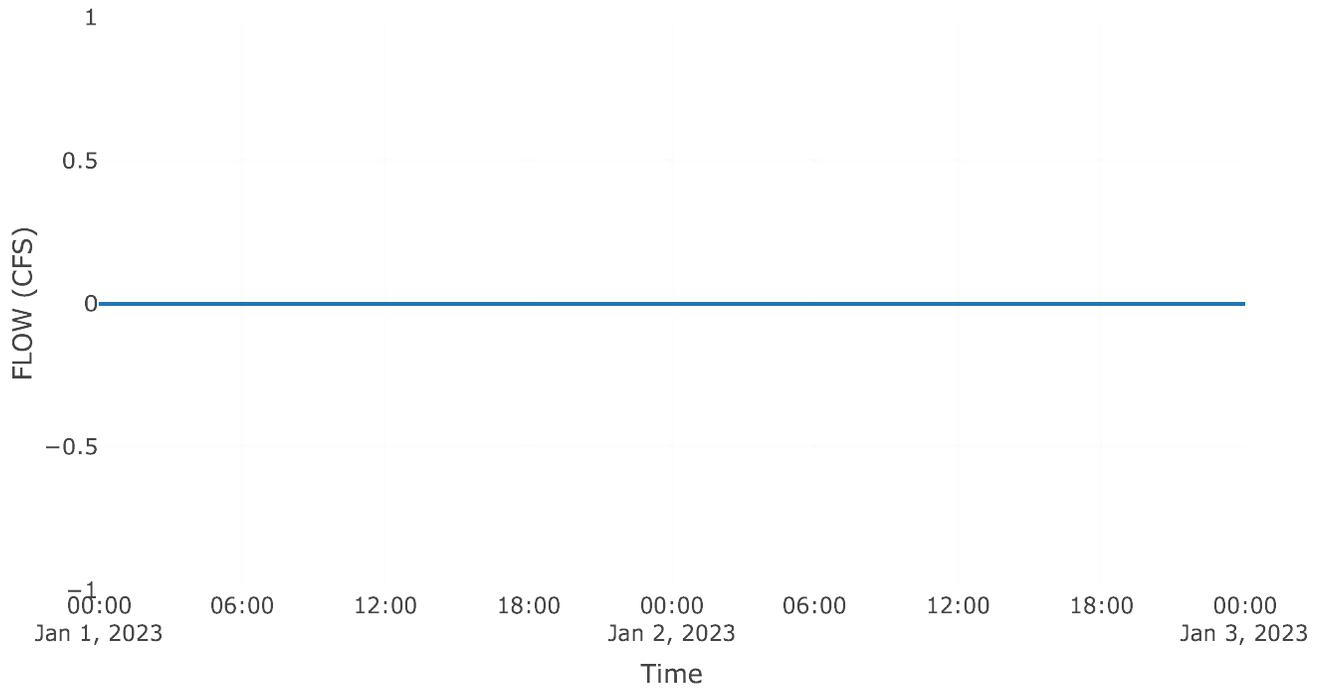
### Reservoir Area



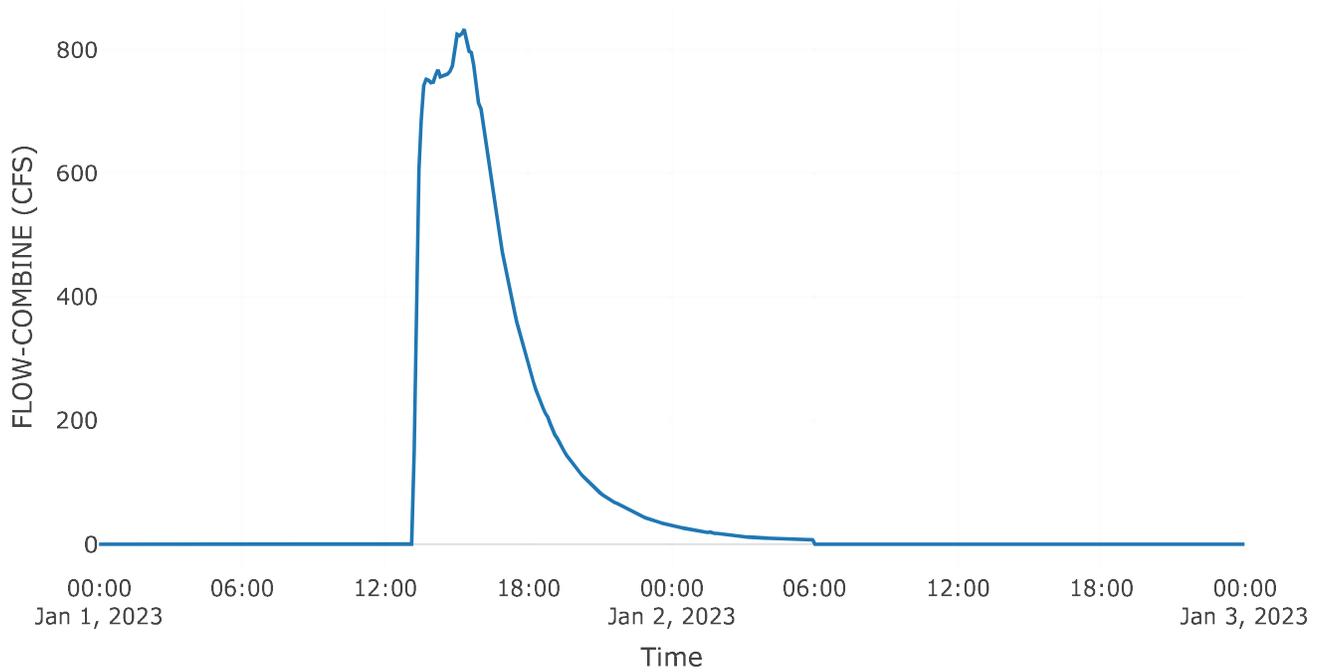
### Storage



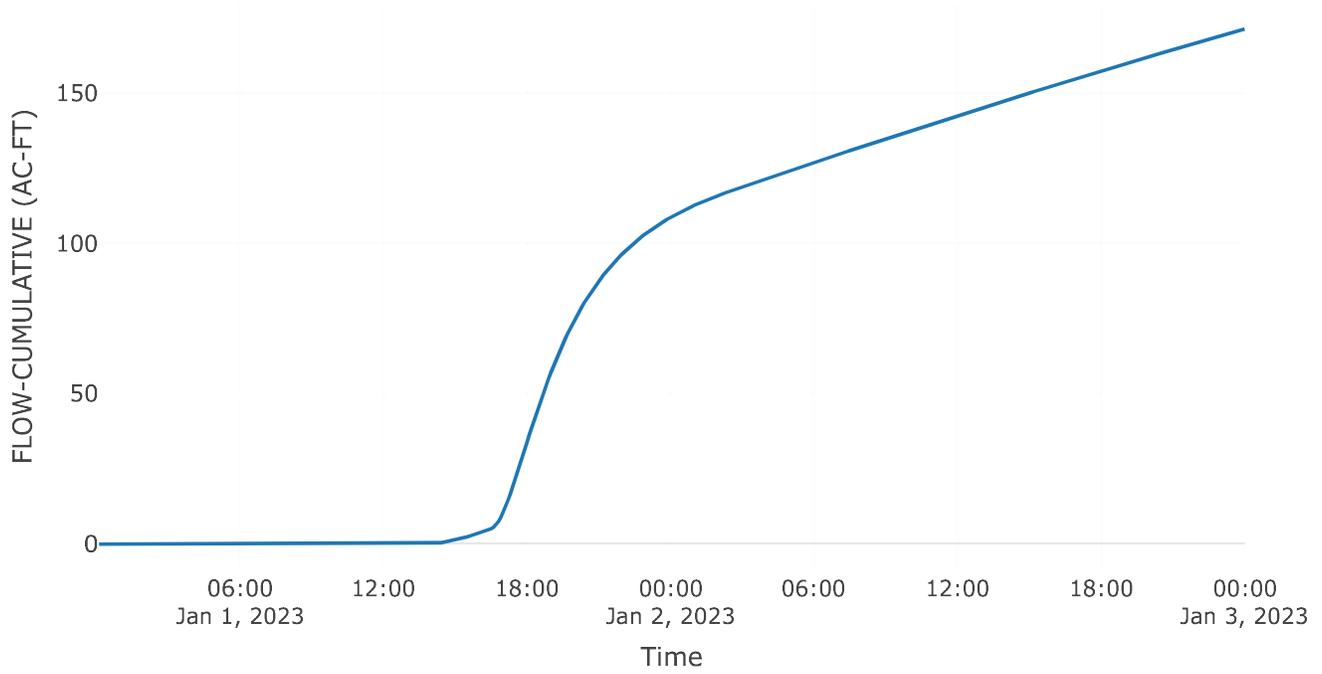
### Spillway 3



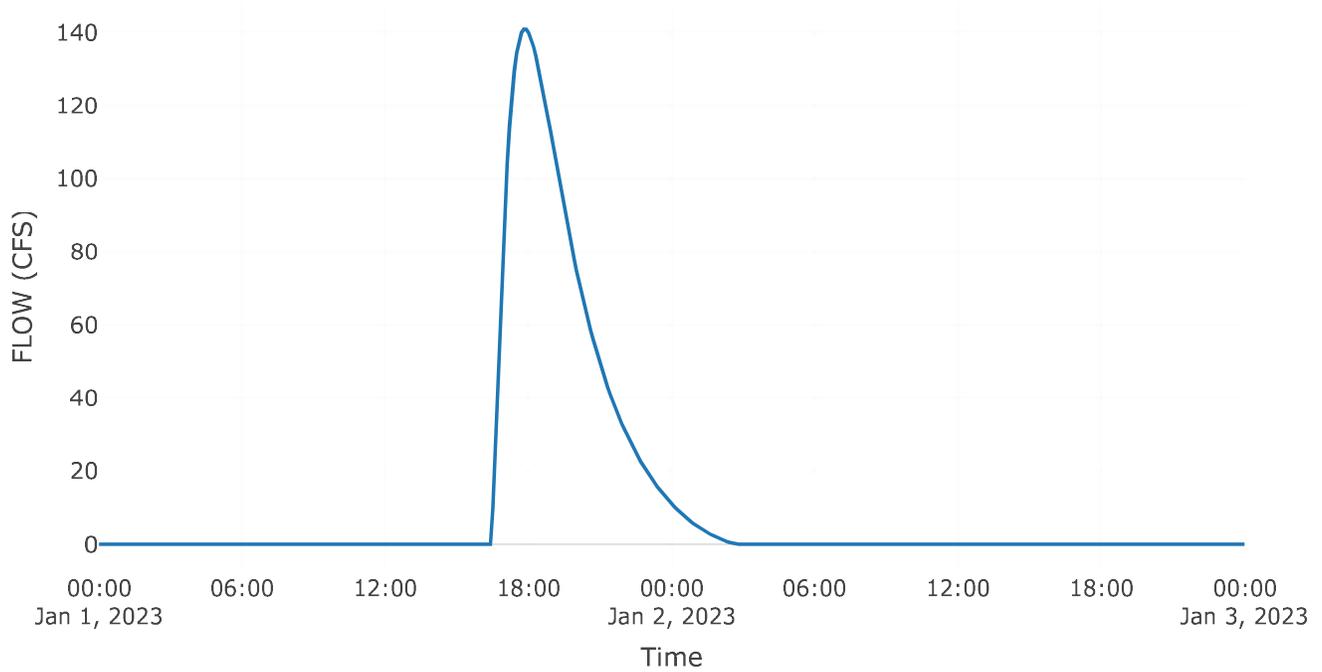
### Combined Inflow



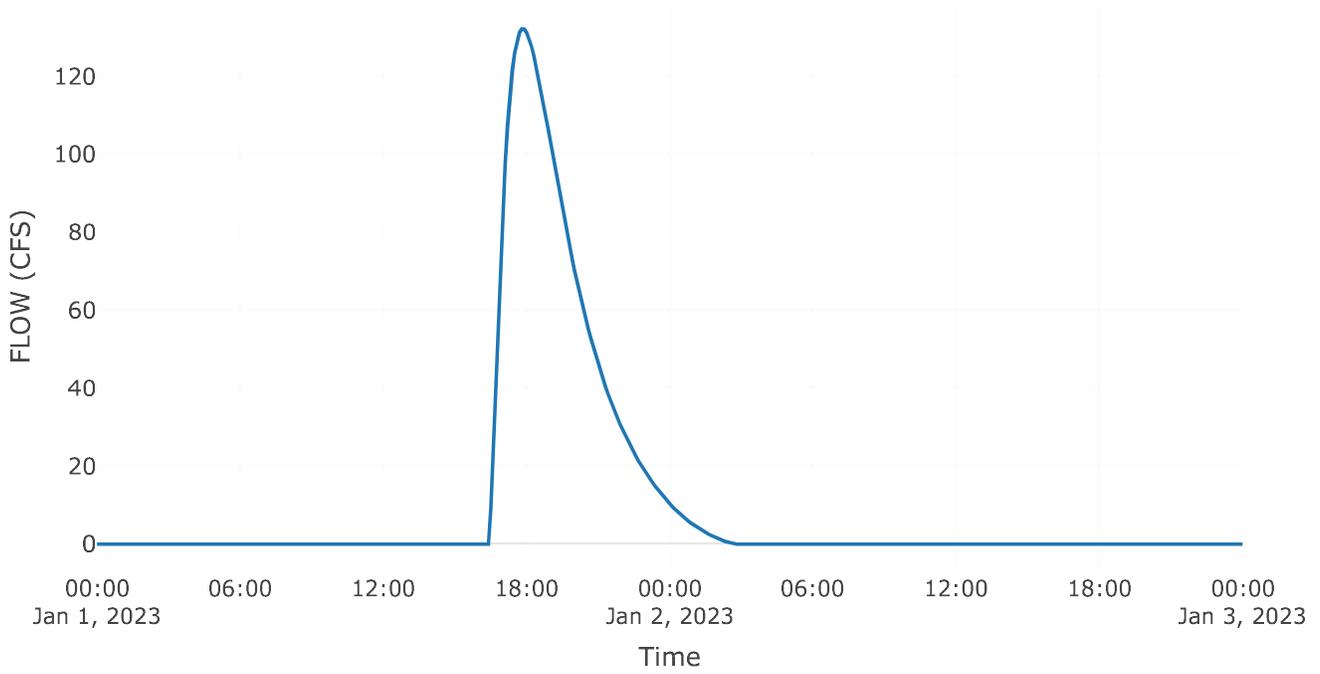
### Cumulative Outflow



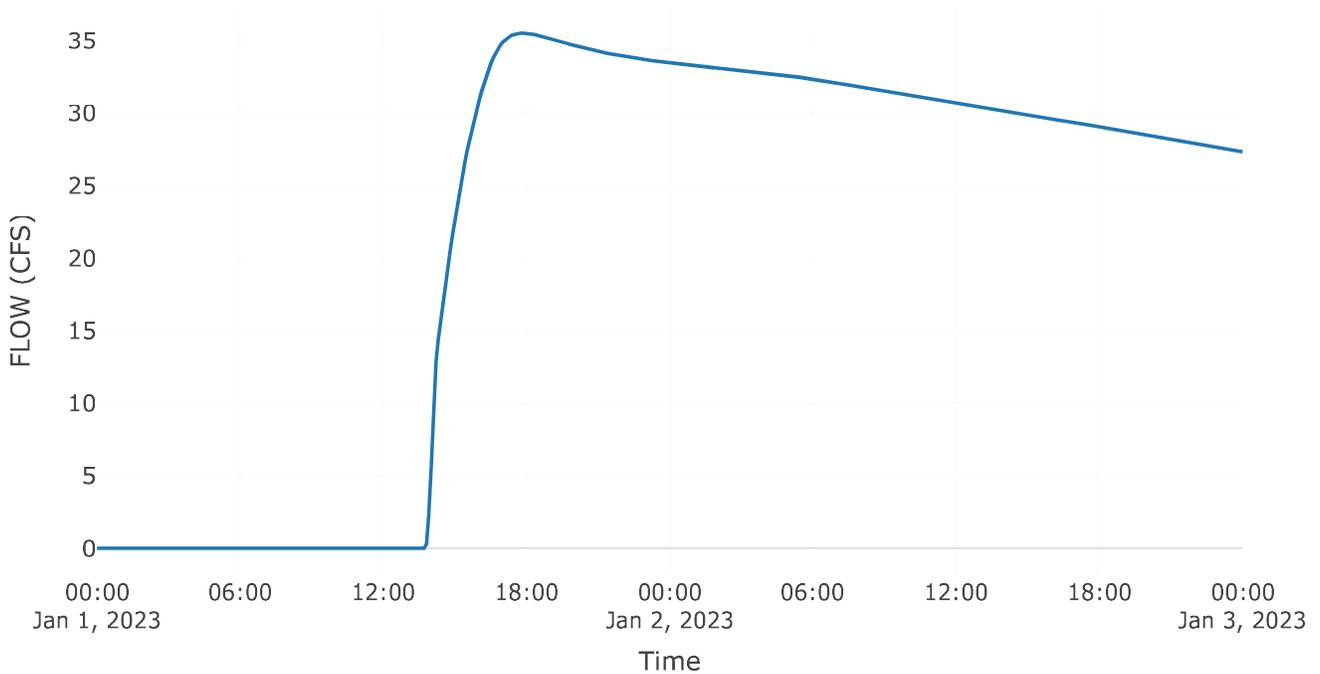
### Spillway 1



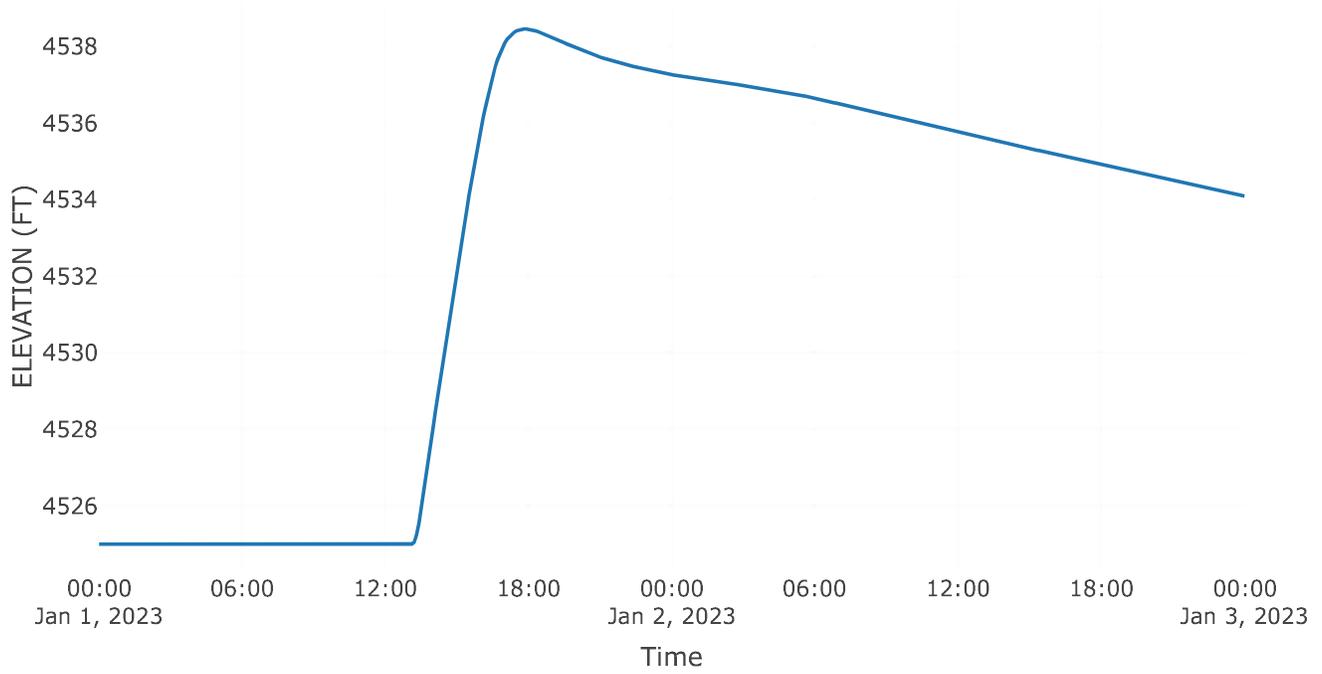
### Spillway 2



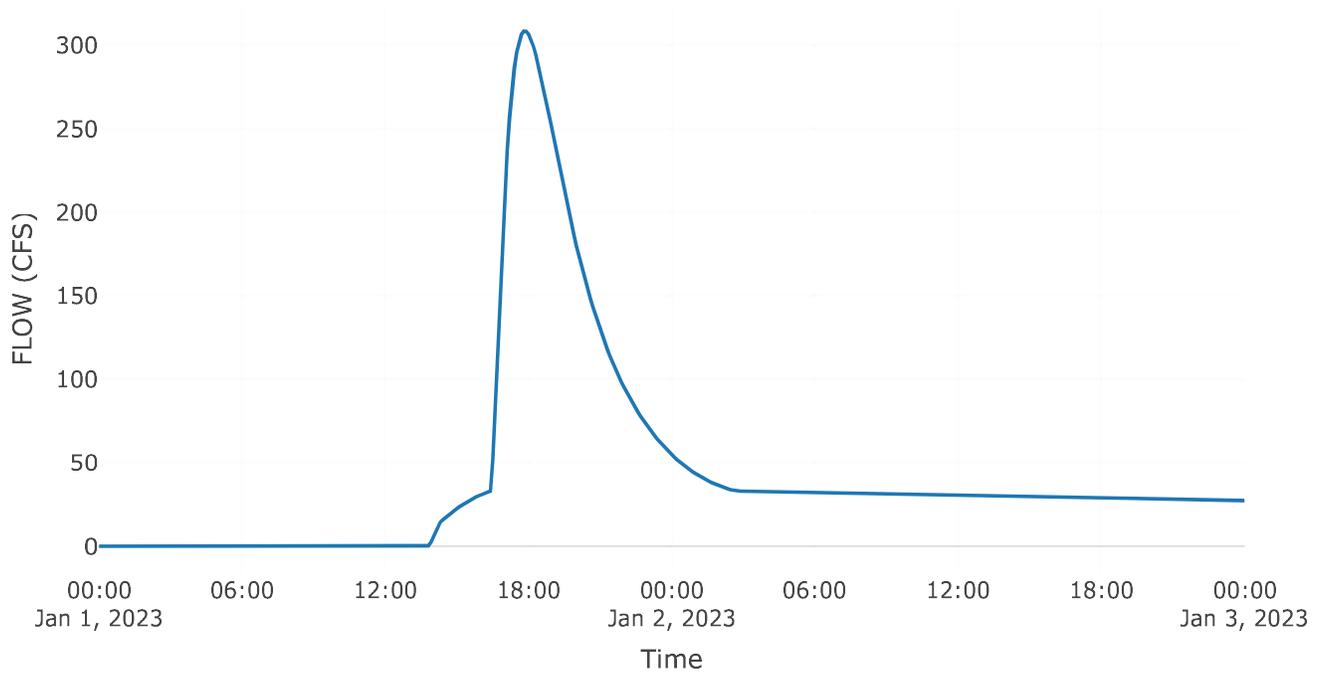
### Outlet 1



### Pool Elevation



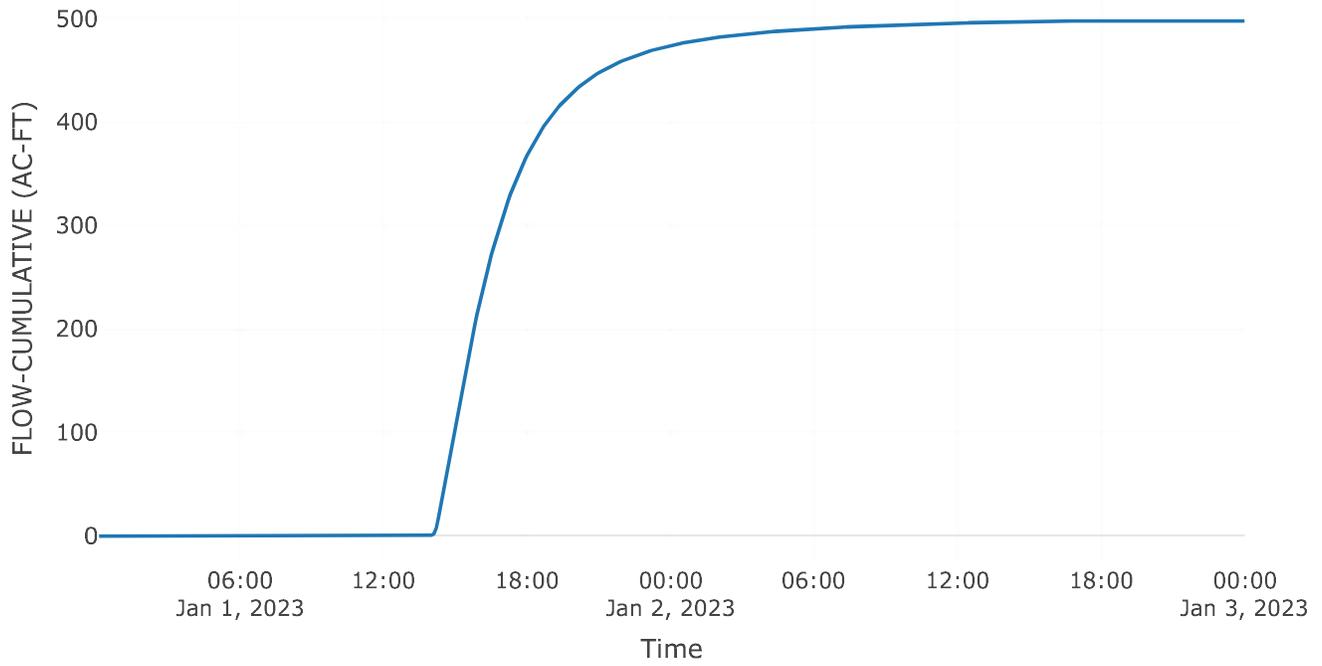
### Outflow



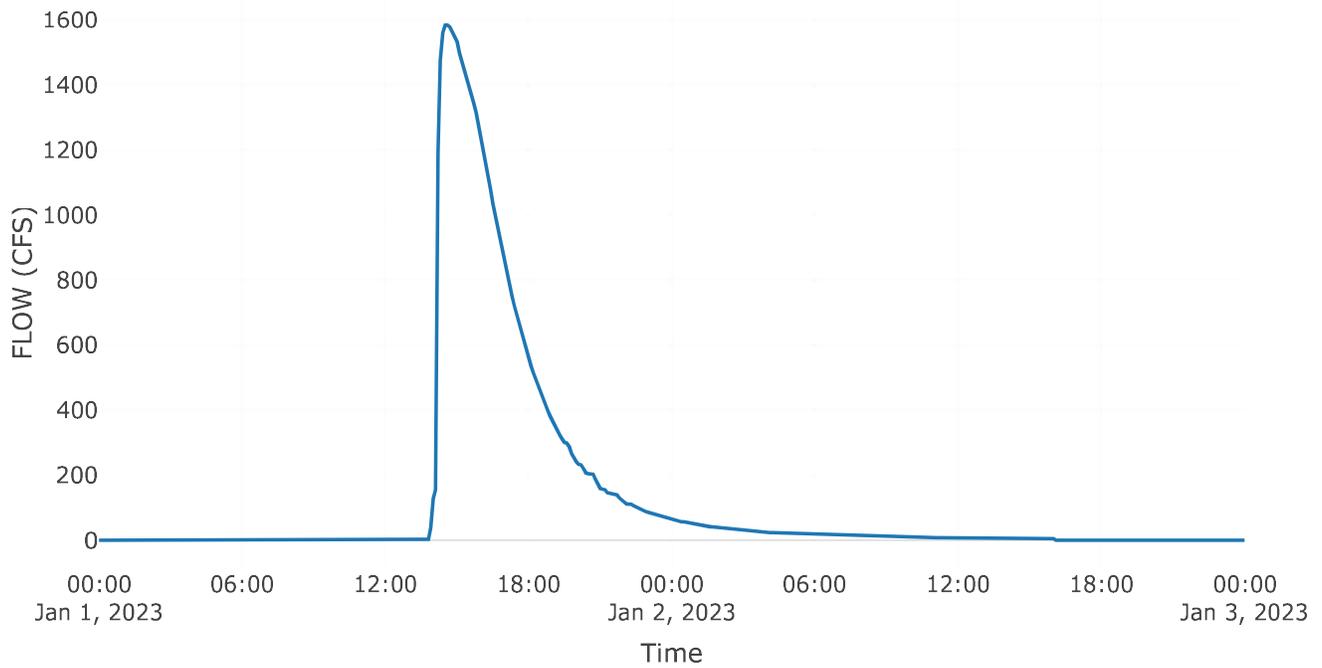
**Source: XS ID 17 - EX\_100****Downstream** : Shawnee Basin**Flow Method** : Gage Flow**Flow Gage** : XS ID 17 - EX\_100**Results: XS ID 17 - EX\_100**

Peak Discharge (CFS)	1584.14
Time of Peak Discharge	01Jan2023, 14:30

### Cumulative Outflow



### Outflow

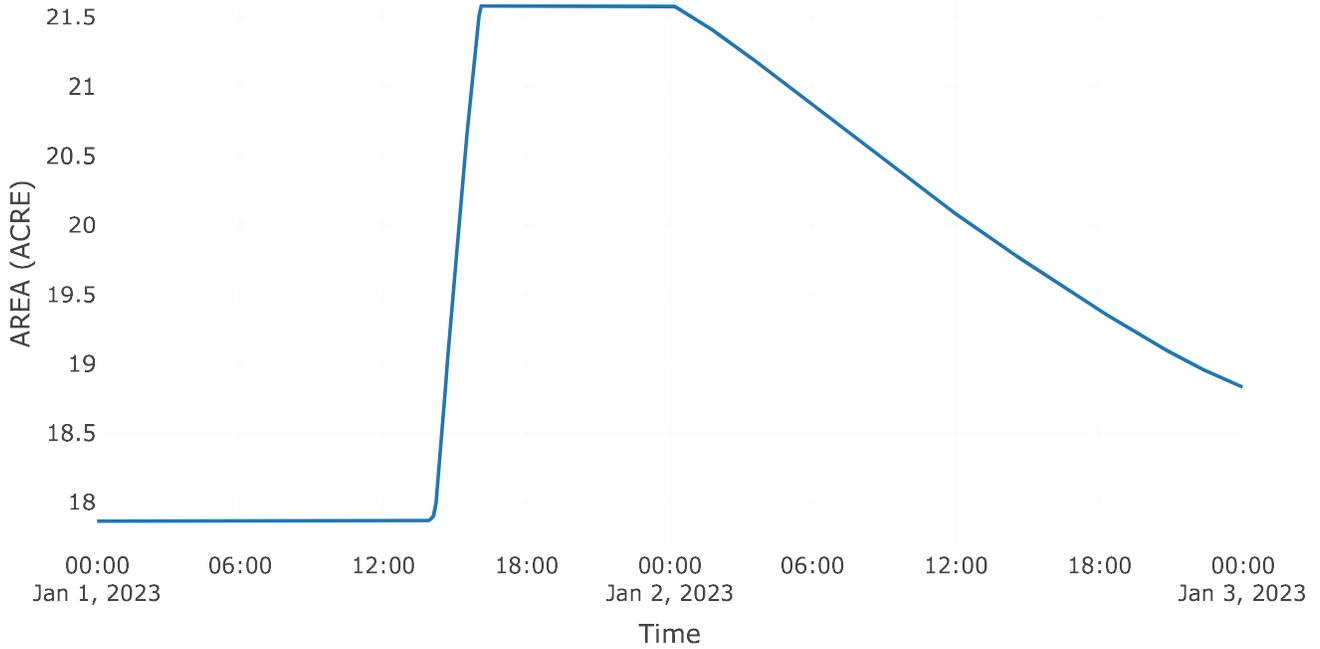


# Reservoir: Shawnee Basin

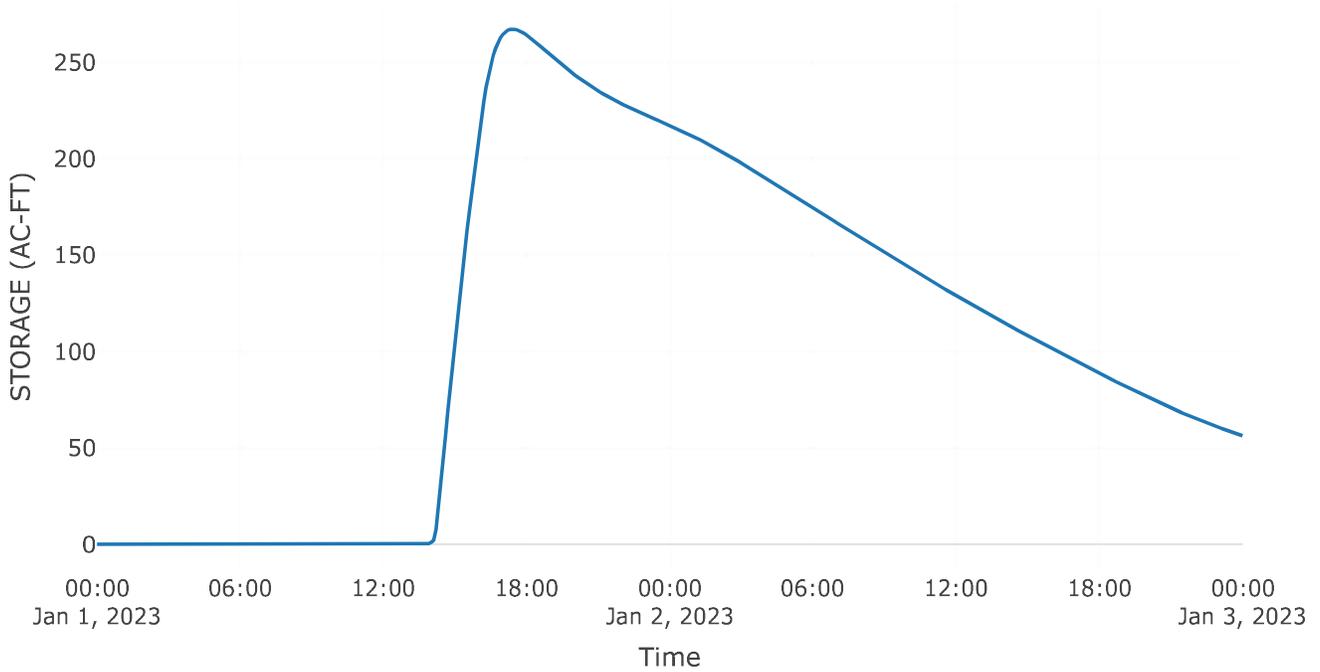
## Results: Shawnee Basin

Peak Discharge (CFS)	717.55
Time of Peak Discharge	01Jan2023, 17:24
Peak Inflow (CFS)	1584.14
Time of Peak Inflow	01Jan2023, 14:30
Inflow Volume (AC - FT)	497.78
Maximum Storage (AC - FT)	267.45
Peak Elevation (FT)	4501.35
Discharge Volume (AC - FT)	441.47

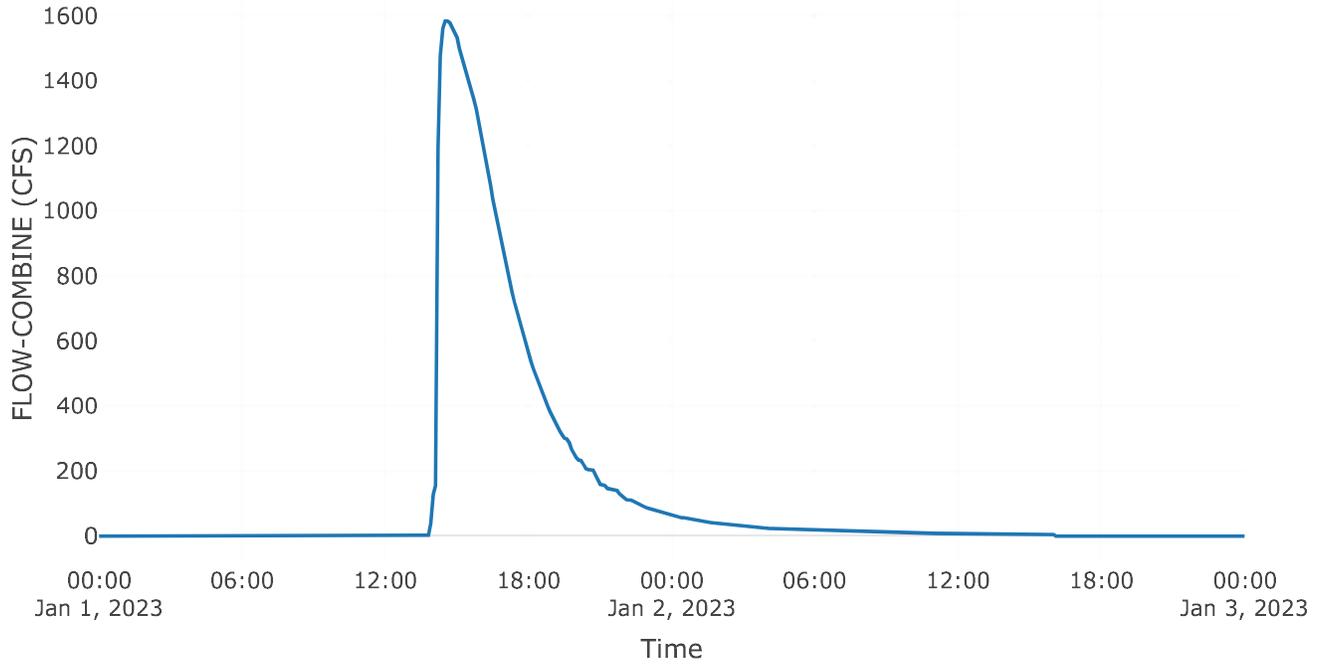
### Reservoir Area



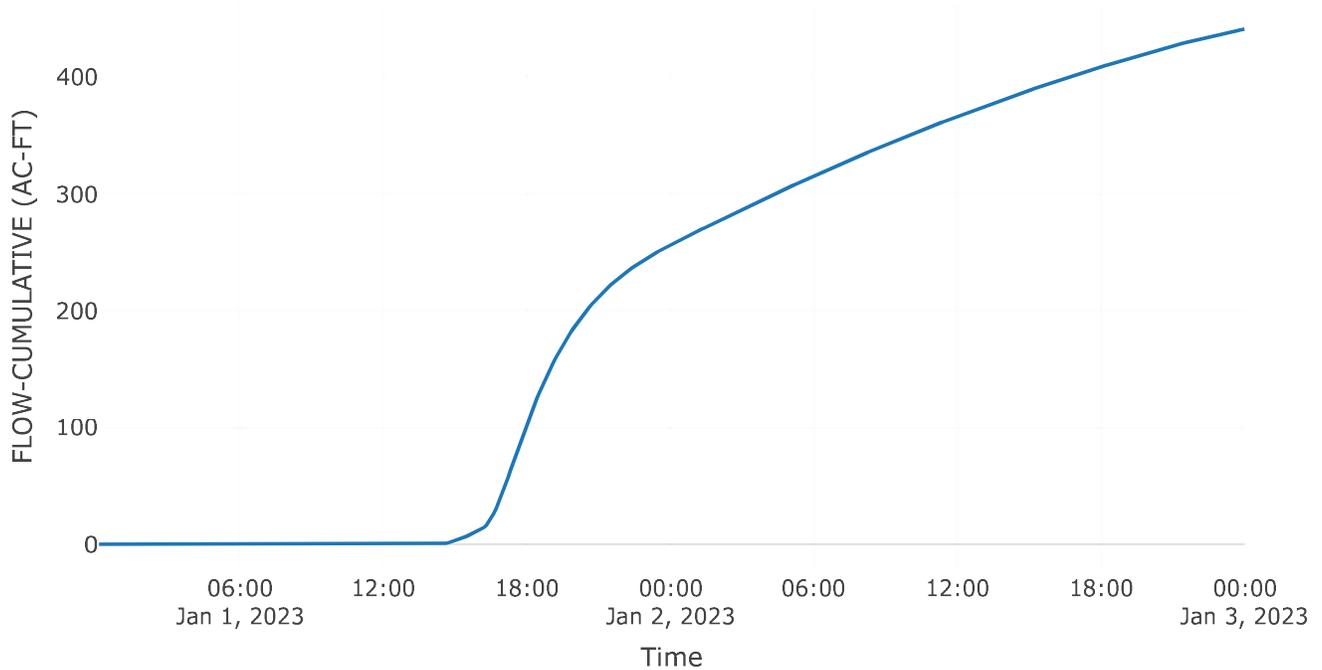
### Storage



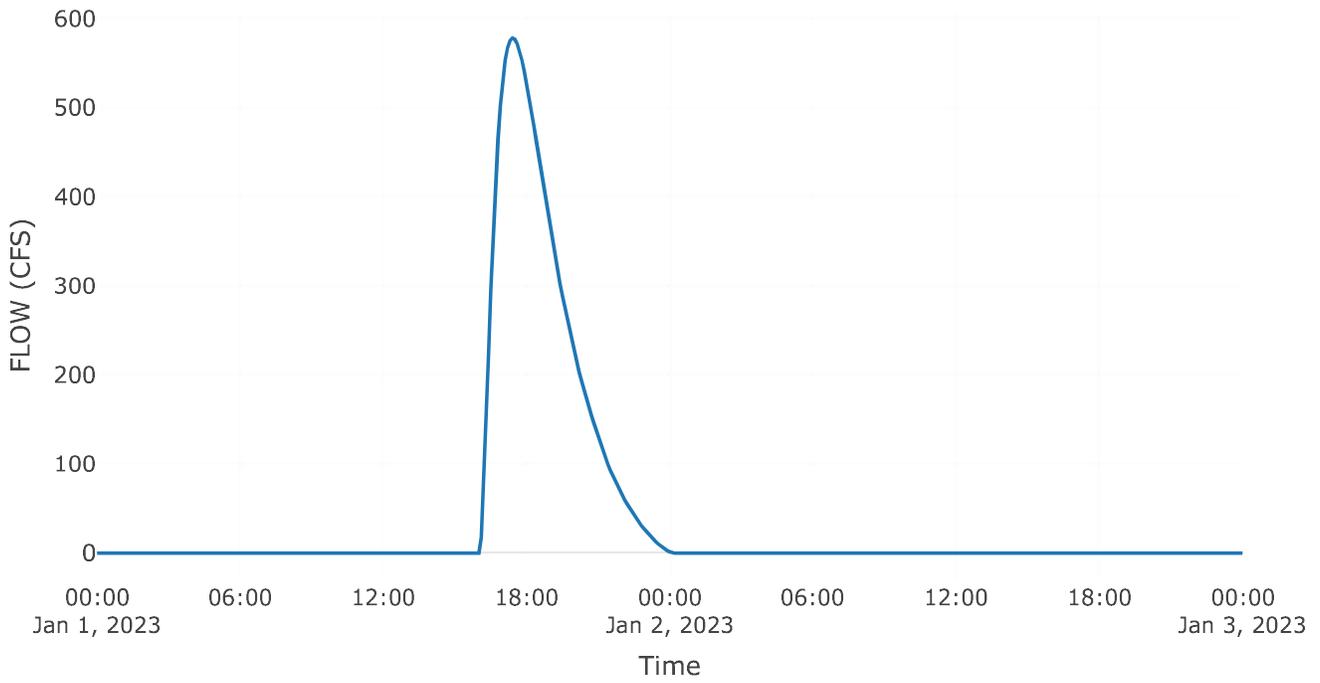
### Combined Inflow



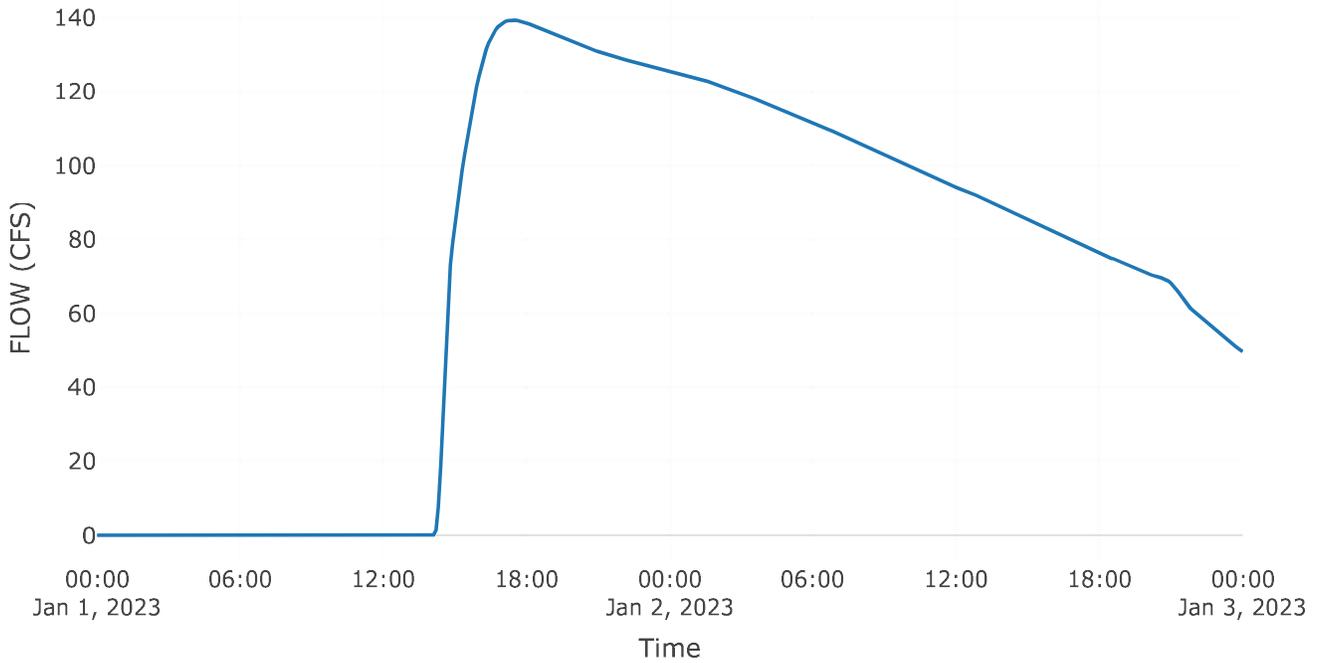
### Cumulative Outflow



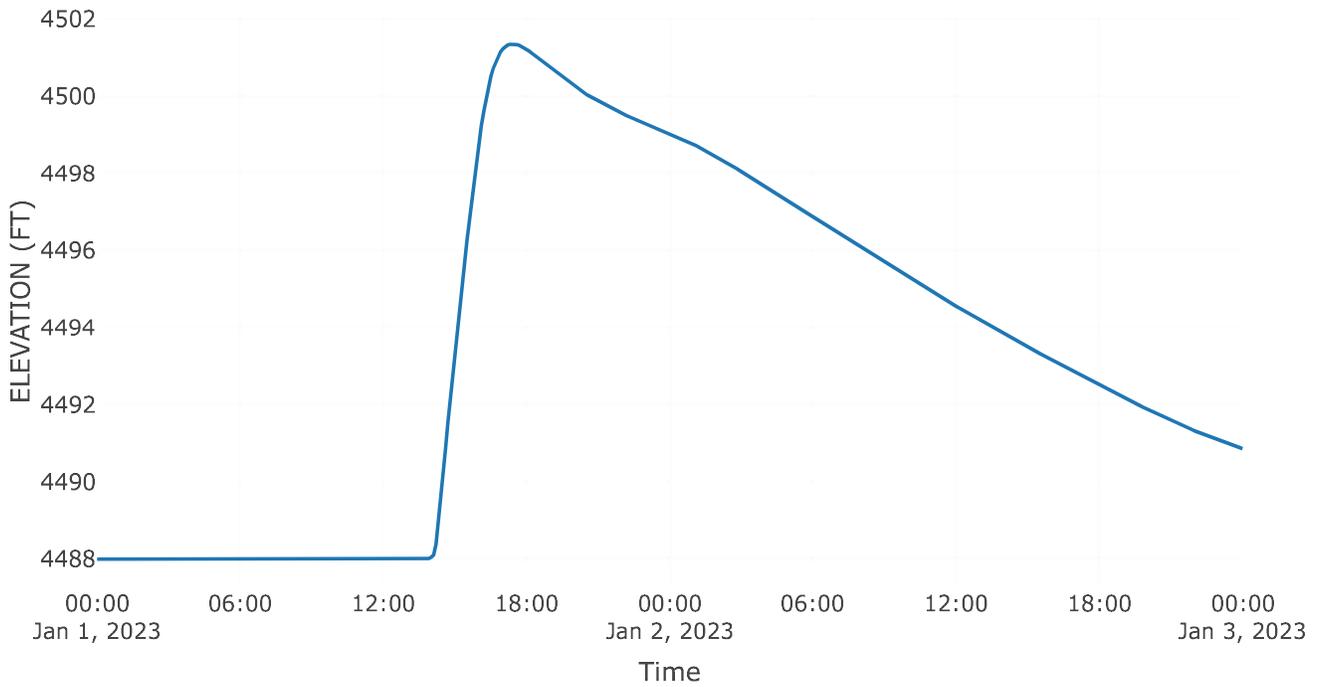
### Spillway 1



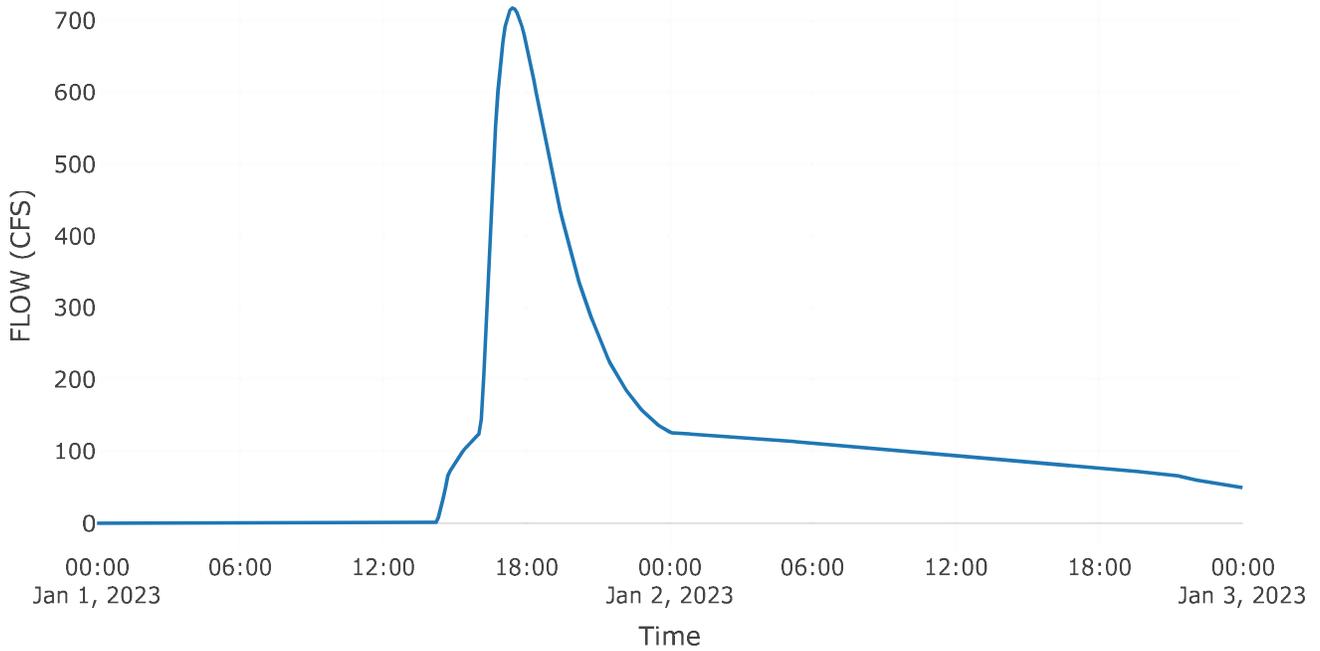
### Outlet 1



### Pool Elevation



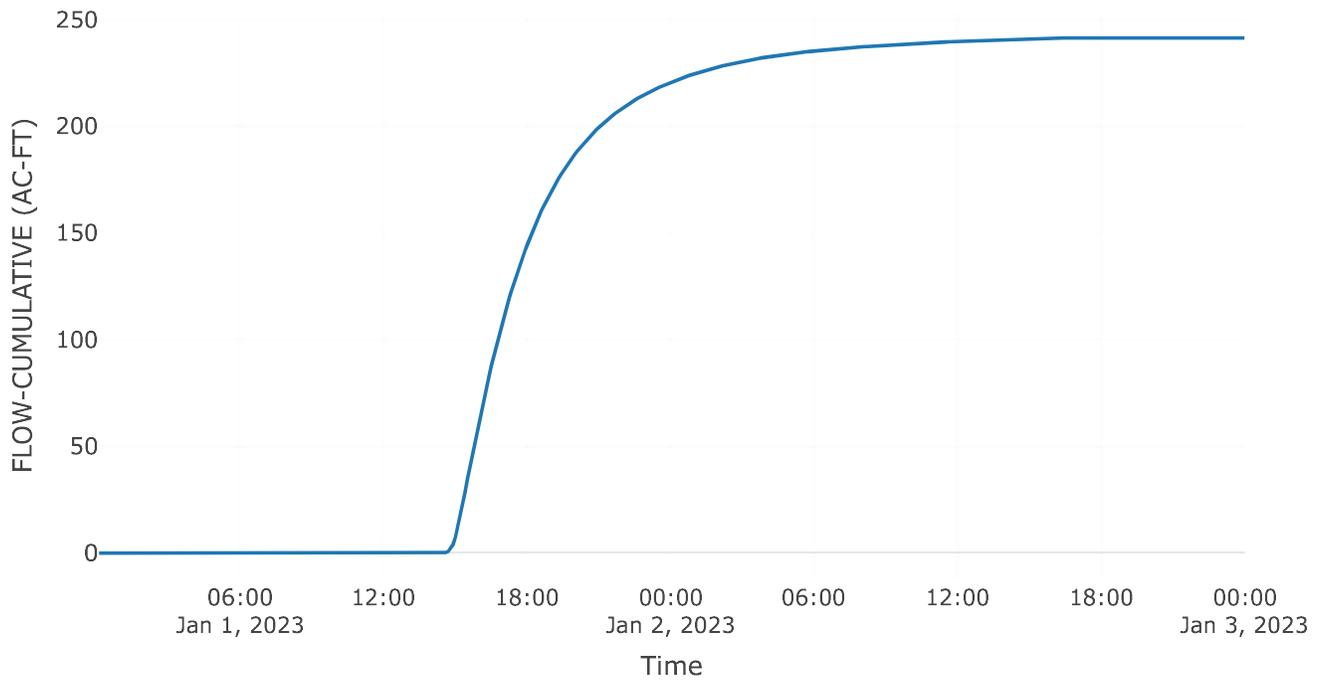
### Outflow



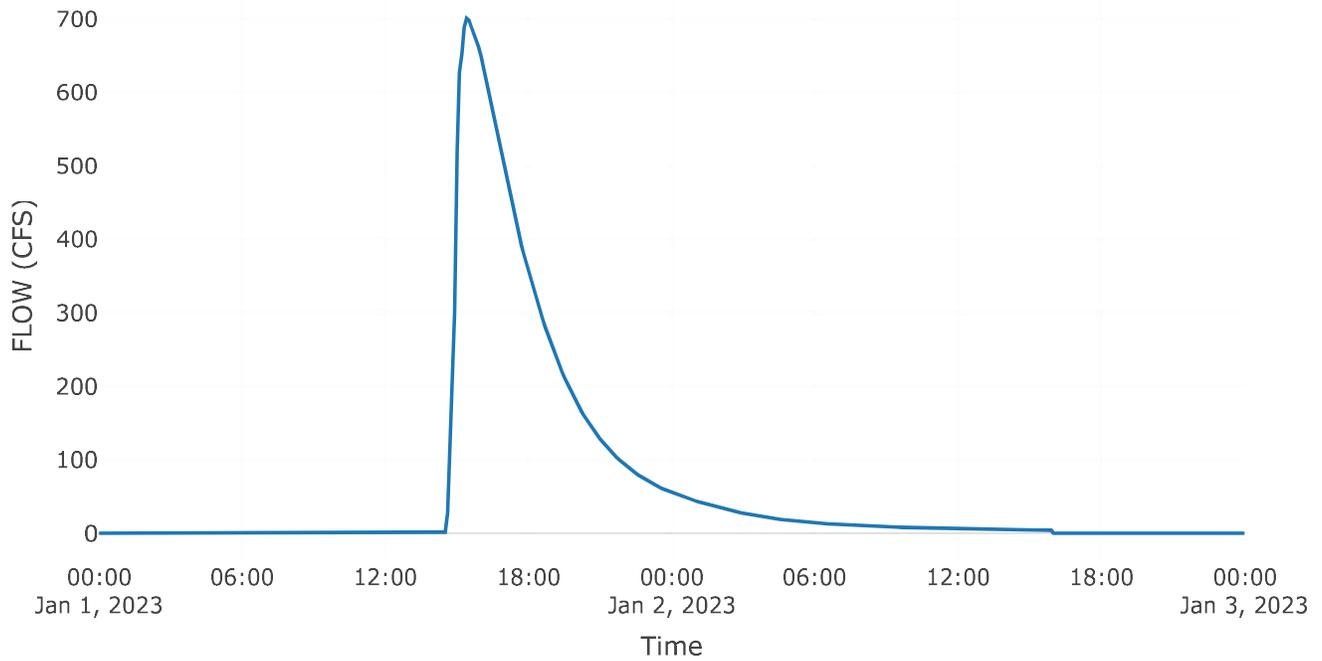
**Source: XS ID 72 - EX\_100****Downstream** : Iron Mountain Basin**Flow Method** : Gage Flow**Flow Gage** : XS ID 72 - EX\_100**Results: XS ID 72 - EX\_100**

Peak Discharge (CFS)	701.48
Time of Peak Discharge	01Jan2023, 15:24

### Cumulative Outflow



### Outflow

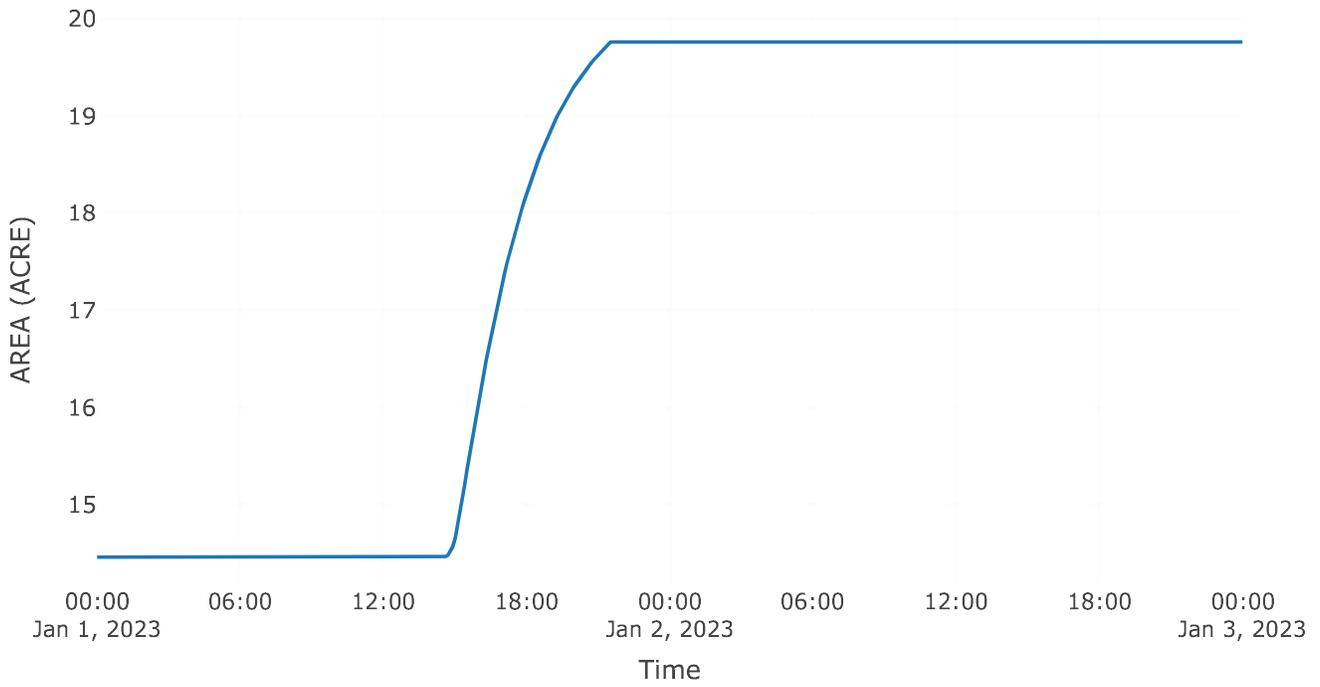


## Reservoir: Iron Mountain Basin

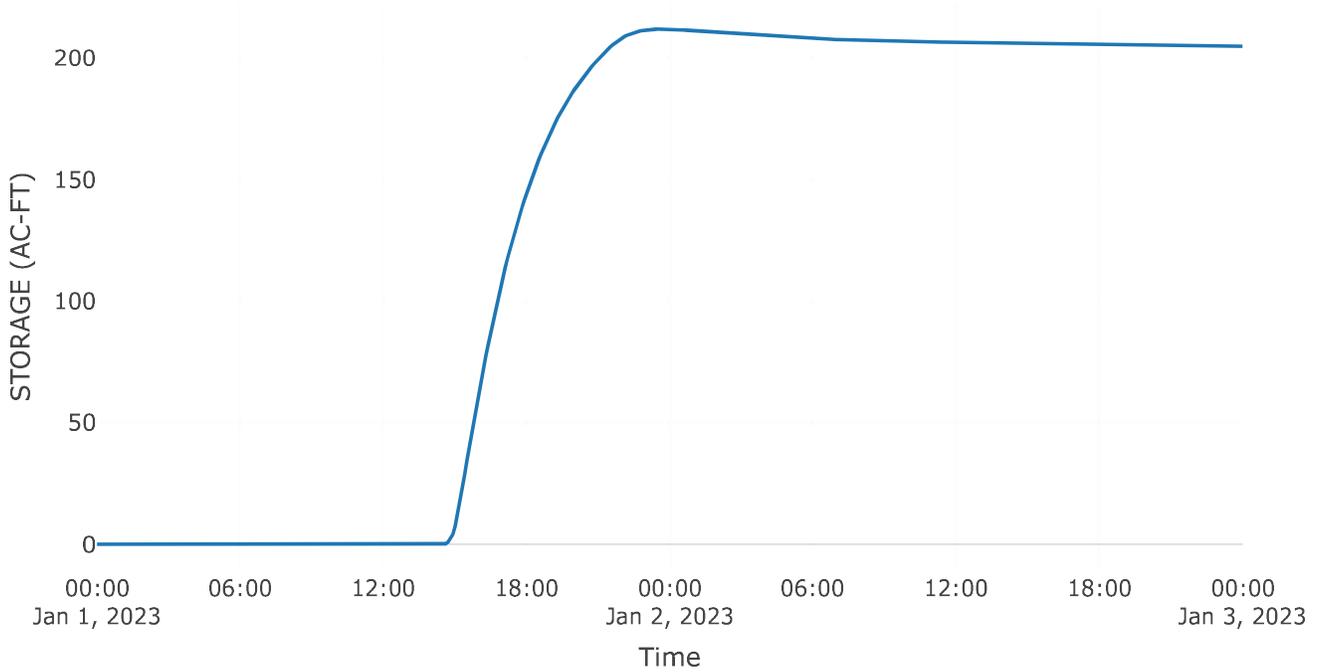
### Results: Iron Mountain Basin

Peak Discharge (CFS)	60.14
Time of Peak Discharge	01Jan2023, 23:36
Peak Inflow (CFS)	701.48
Time of Peak Inflow	01Jan2023, 15:24
Inflow Volume (AC - FT)	241.35
Maximum Storage (AC - FT)	211.82
Peak Elevation (FT)	4322.37
Discharge Volume (AC - FT)	36.52

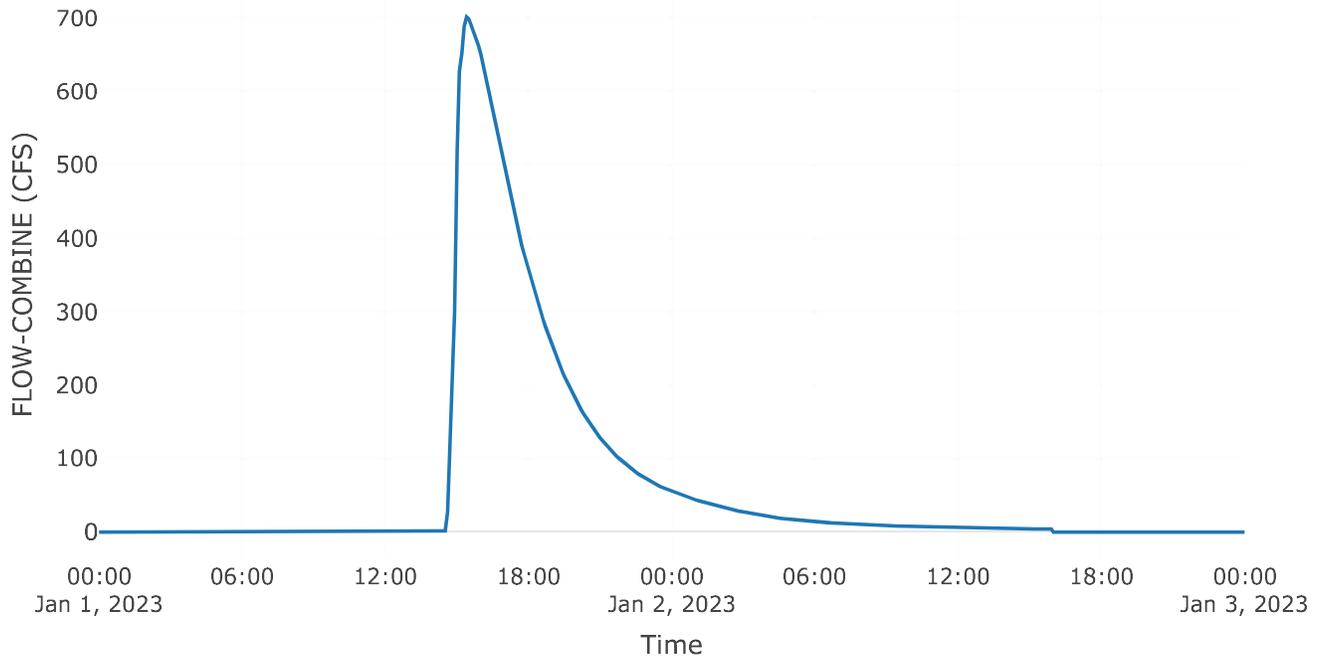
### Reservoir Area



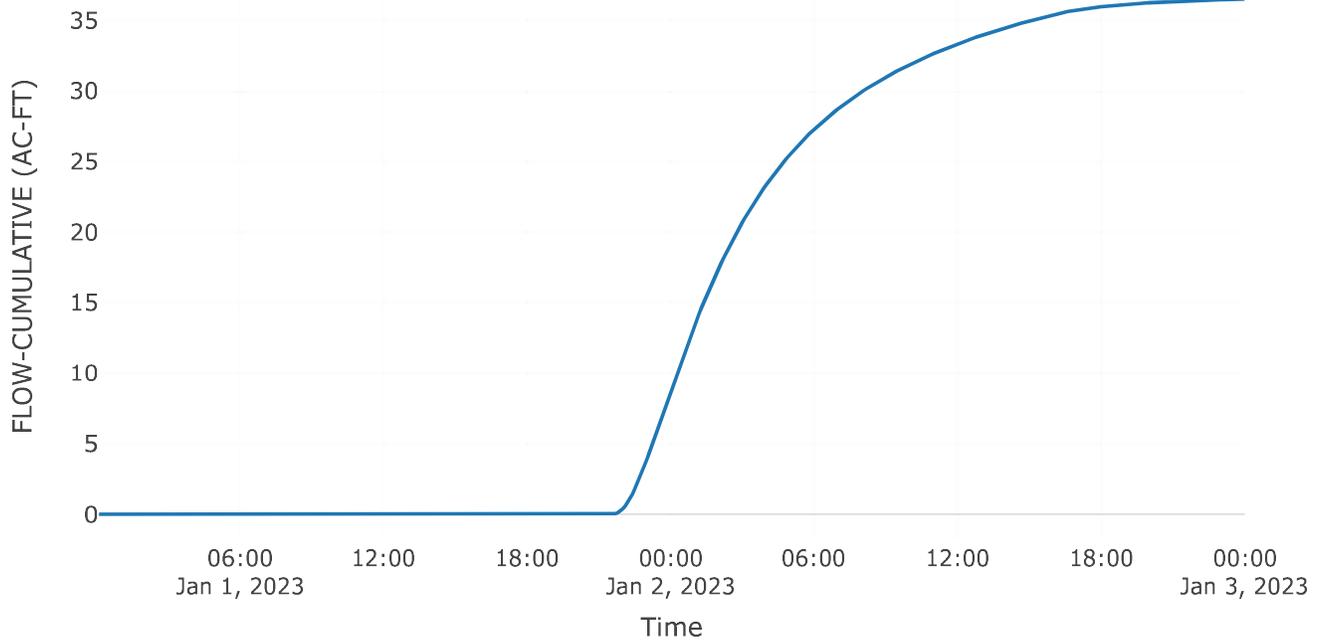
### Storage



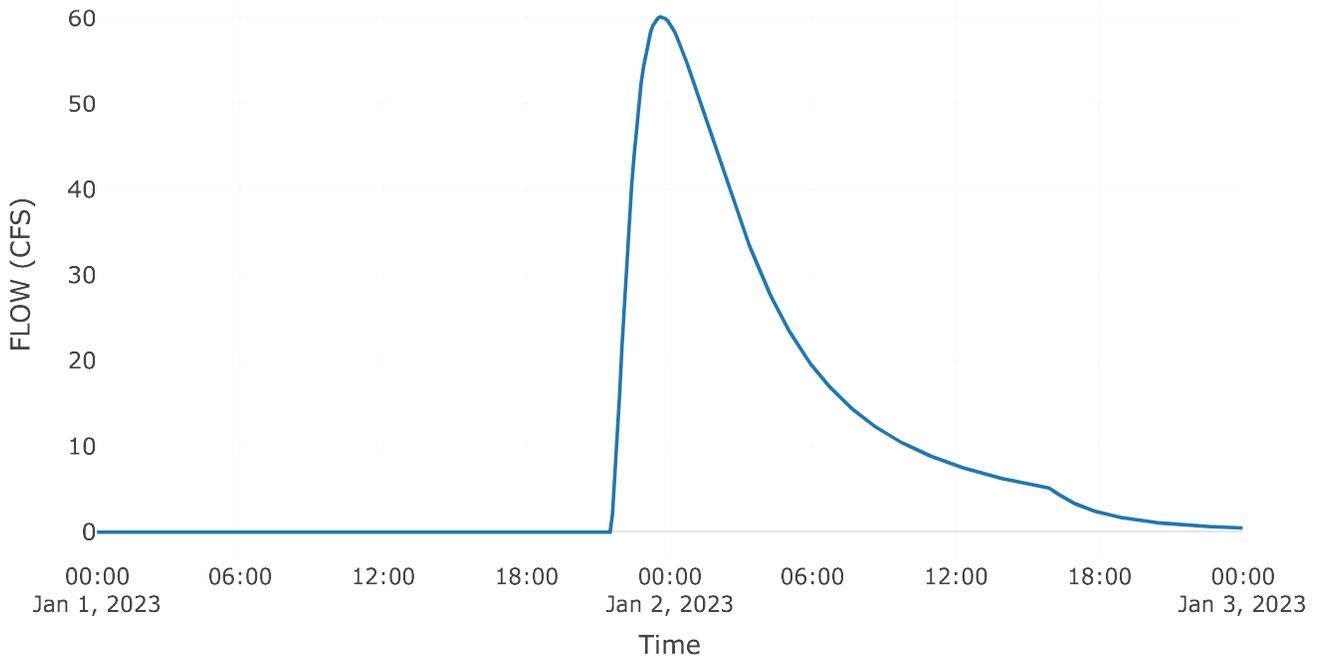
### Combined Inflow



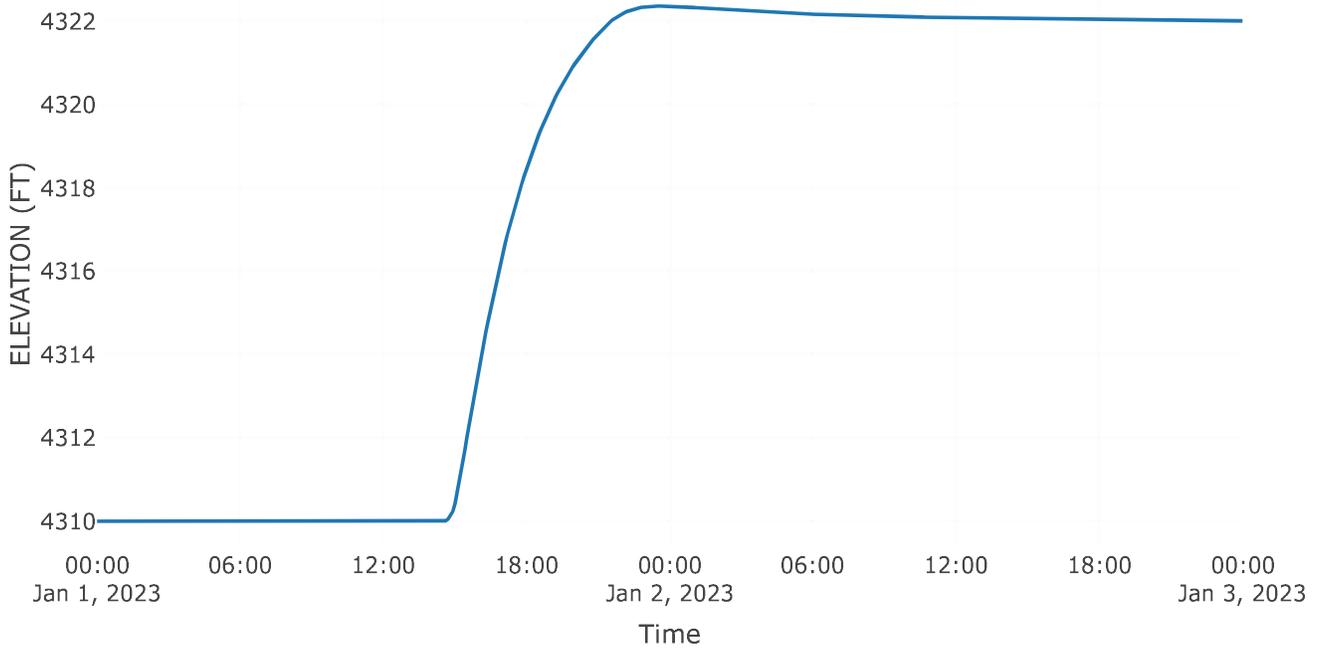
### Cumulative Outflow



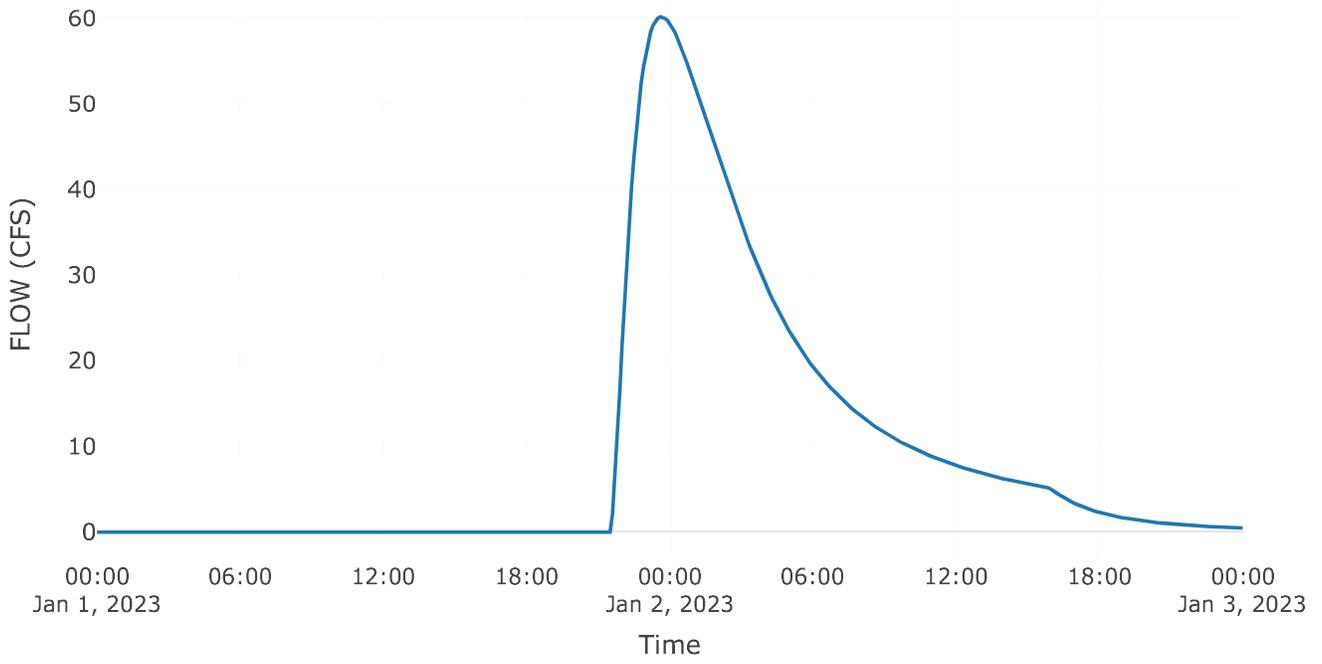
### Spillway 1



### Pool Elevation



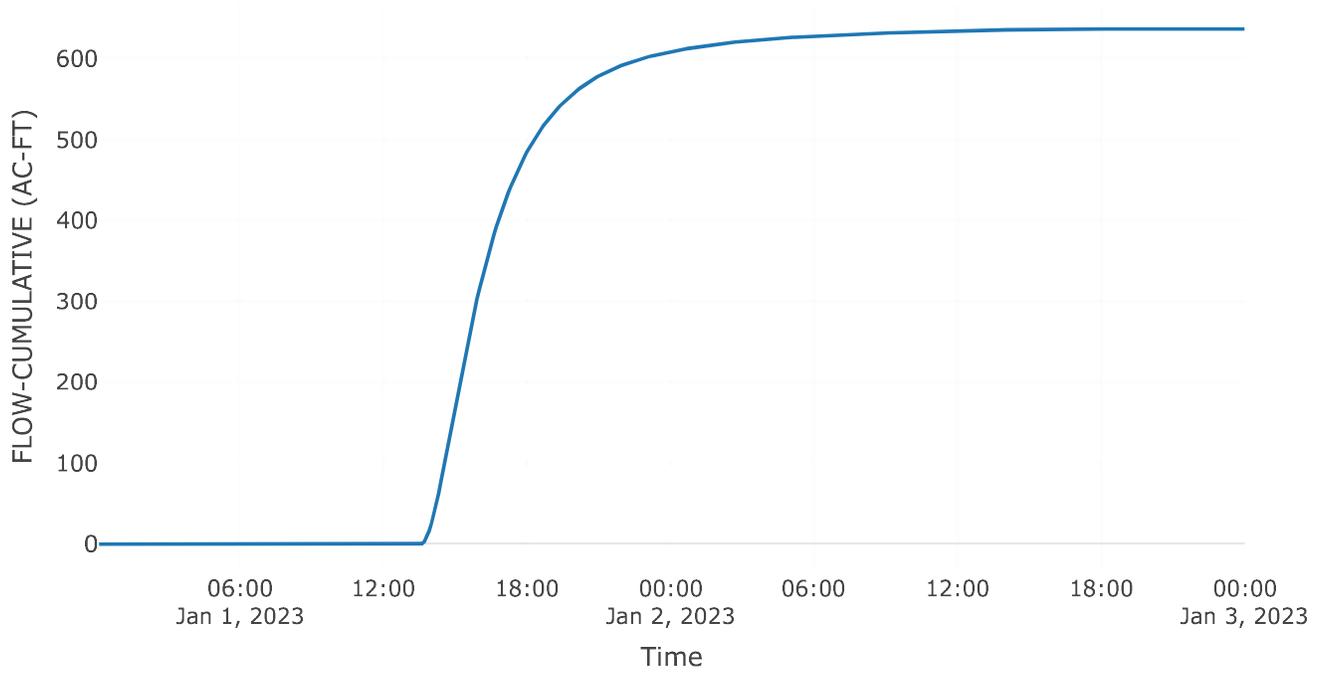
### Outflow



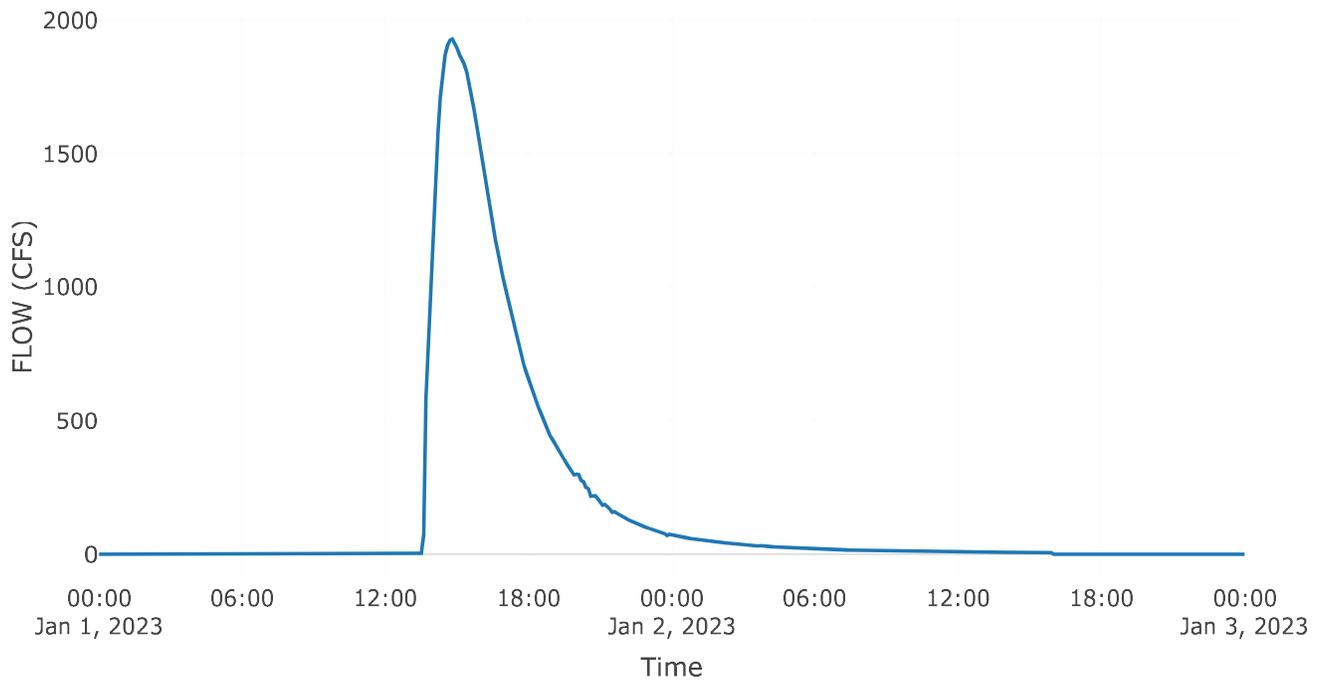
**Source: XS ID 74 - EX\_100****Downstream** : Geurts Basin\_Upper**Flow Method** : Gage Flow**Flow Gage** : XS ID 74 - EX\_100**Results: XS ID 74 - EX\_100**

Peak Discharge (CFS)	1929.73
Time of Peak Discharge	01Jan2023, 14:48

### Cumulative Outflow



### Outflow

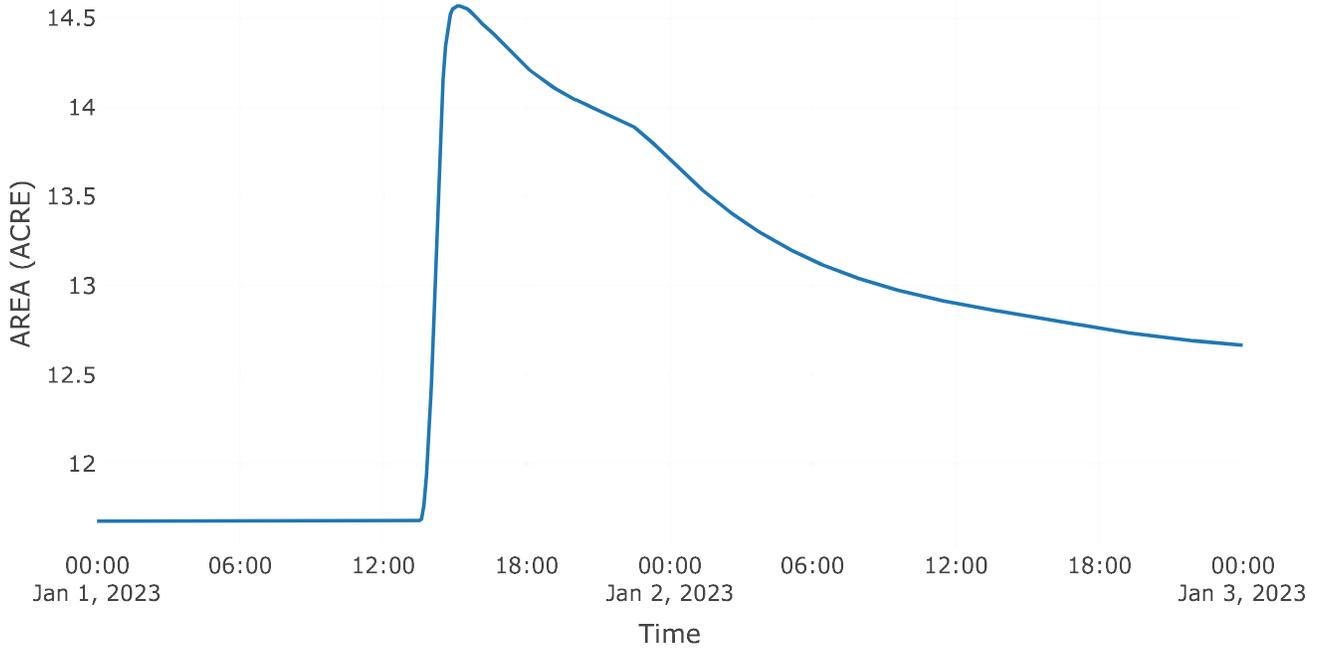


## Reservoir: Geurts Basin\_Upper

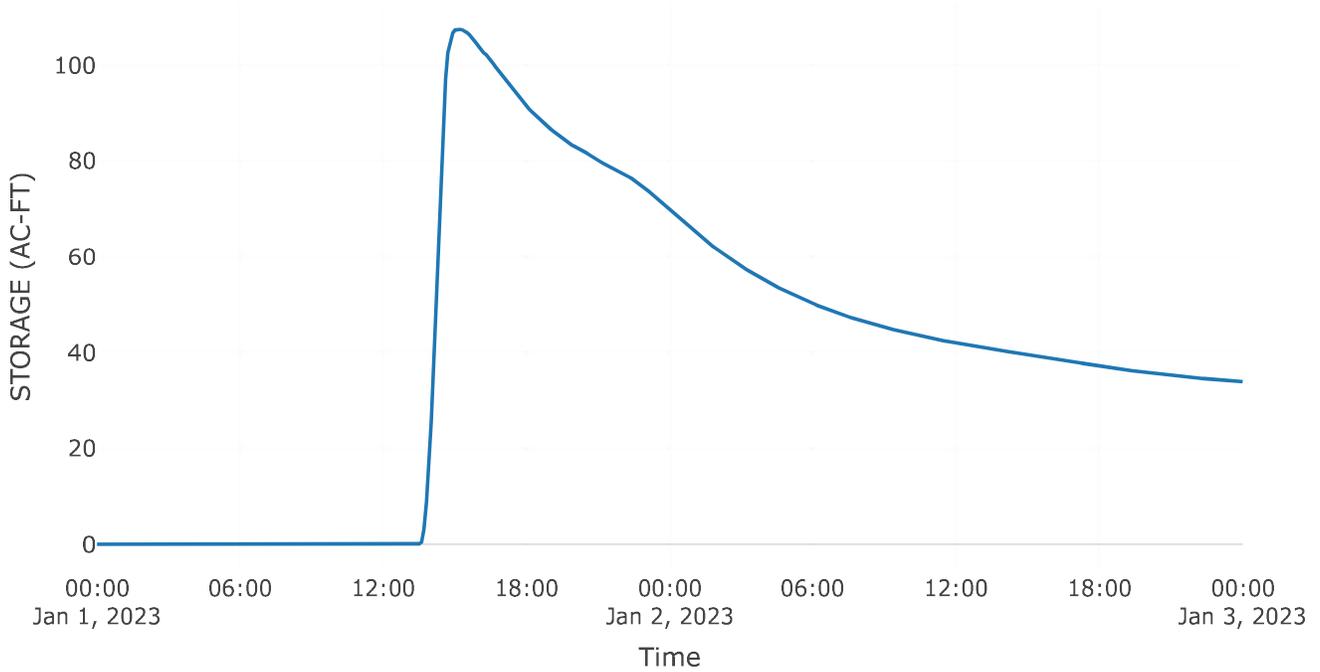
### Results: Geurts Basin\_Upper

Peak Discharge (CFS)	1868.56
Time of Peak Discharge	01Jan2023, 15:06
Peak Inflow (CFS)	1929.73
Time of Peak Inflow	01Jan2023, 14:48
Inflow Volume (AC - FT)	636.87
Maximum Storage (AC - FT)	107.54
Peak Elevation (FT)	4347.15
Discharge Volume (AC - FT)	602.79

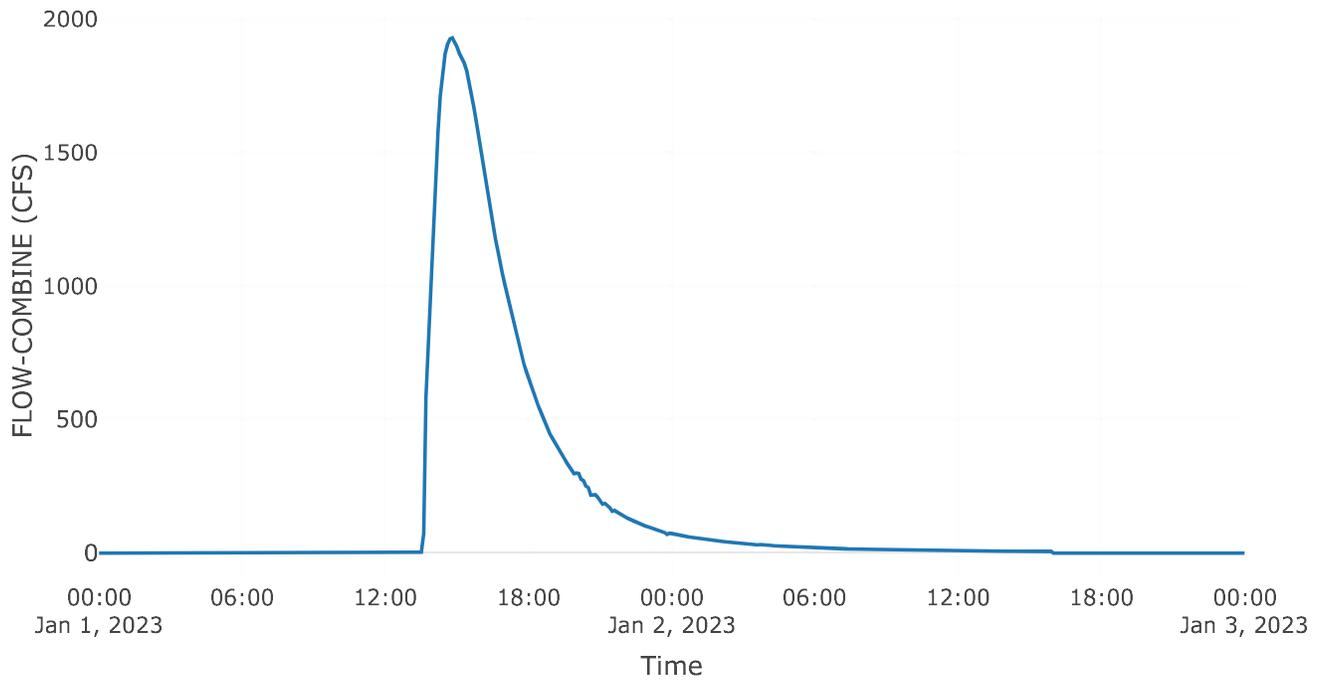
### Reservoir Area



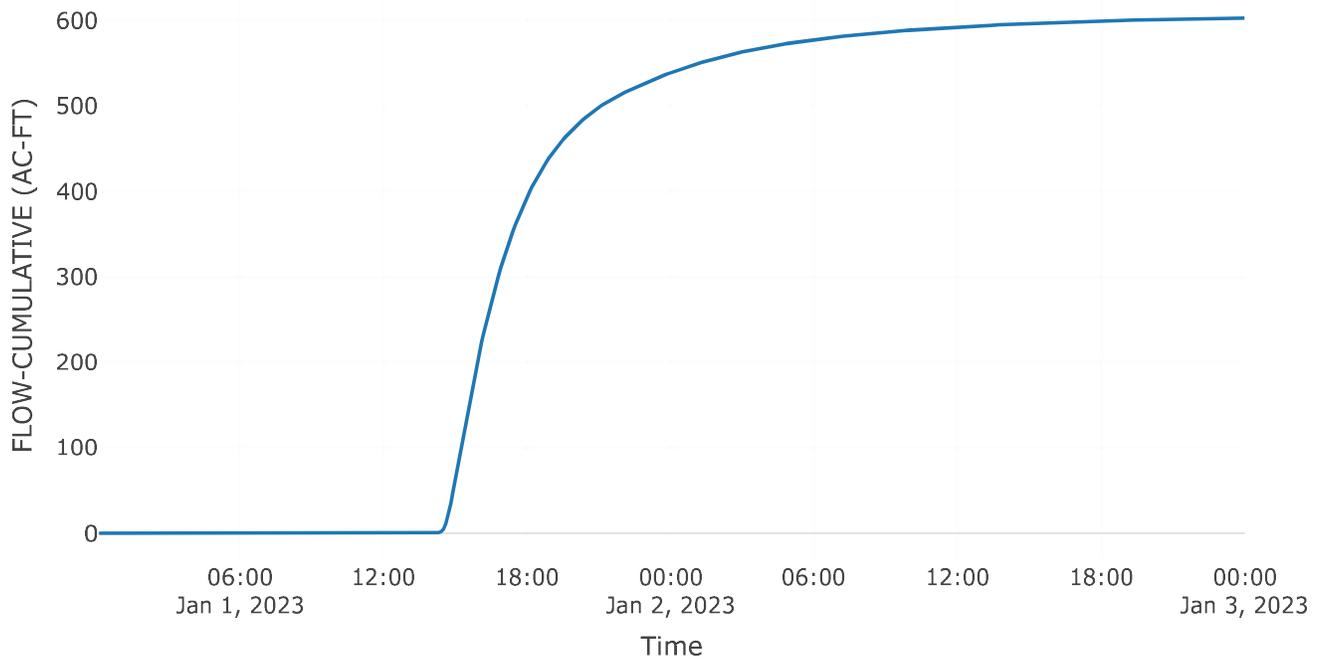
### Storage



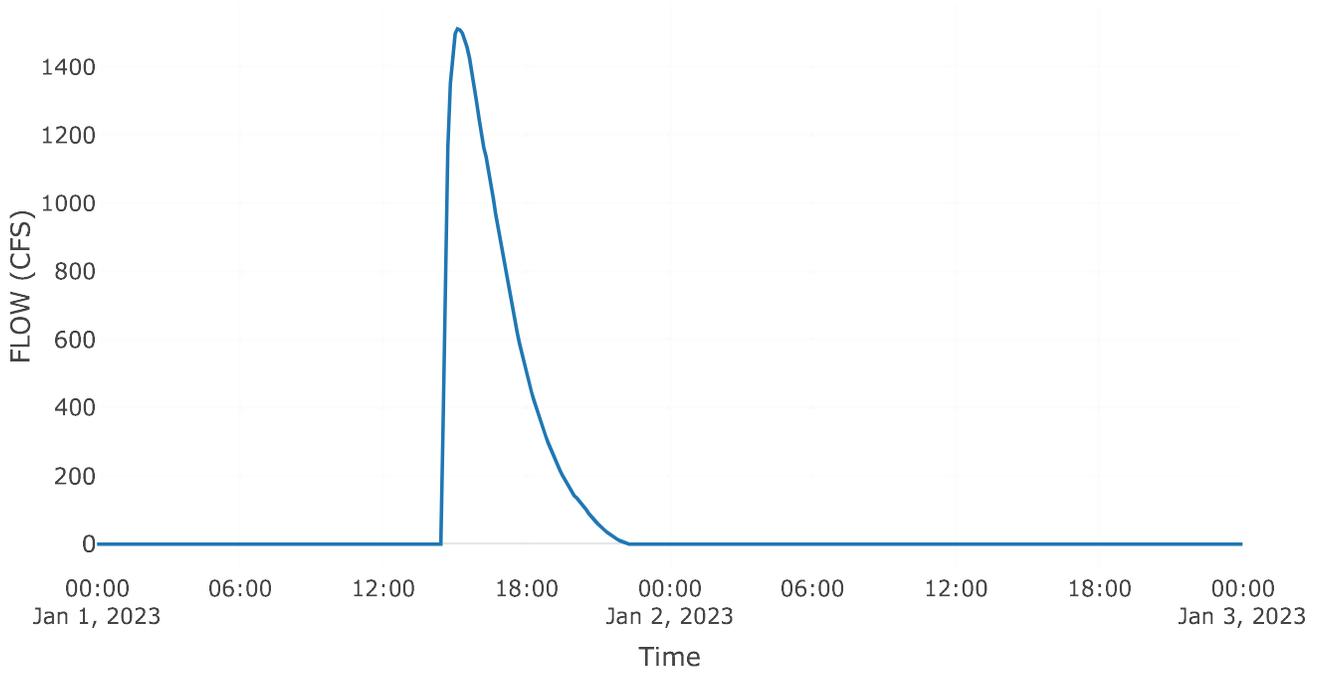
### Combined Inflow



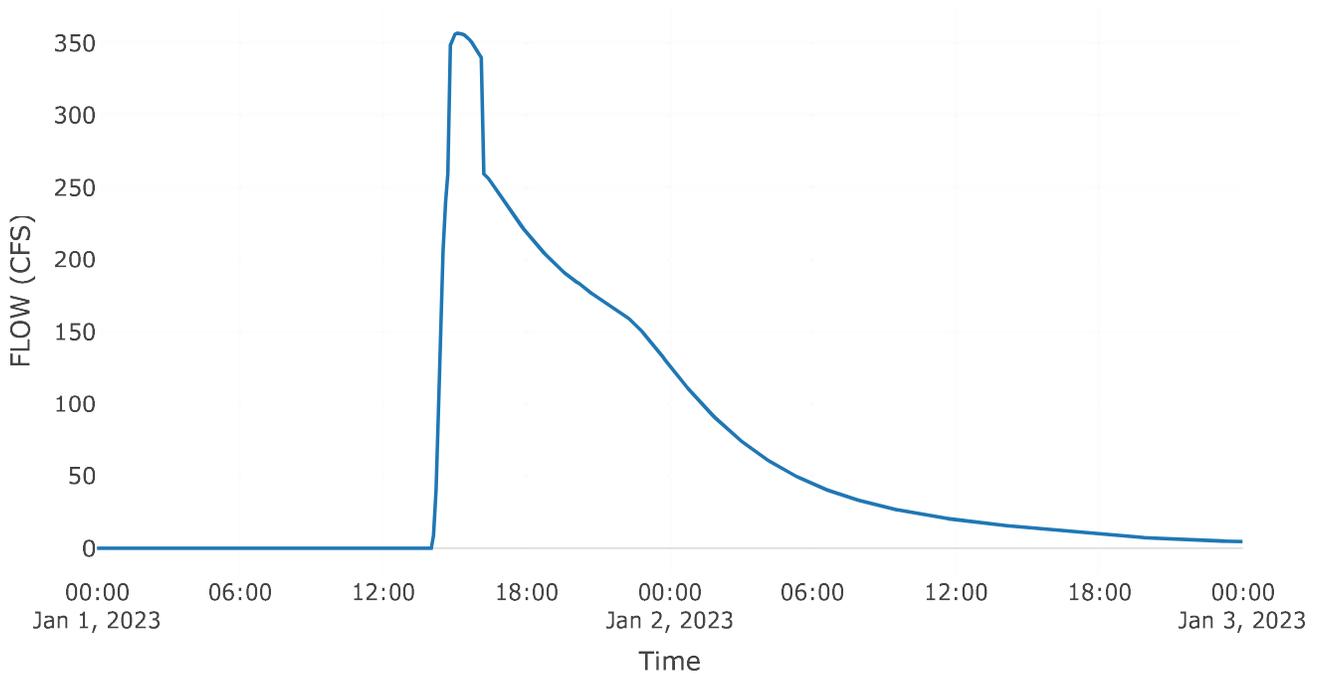
### Cumulative Outflow



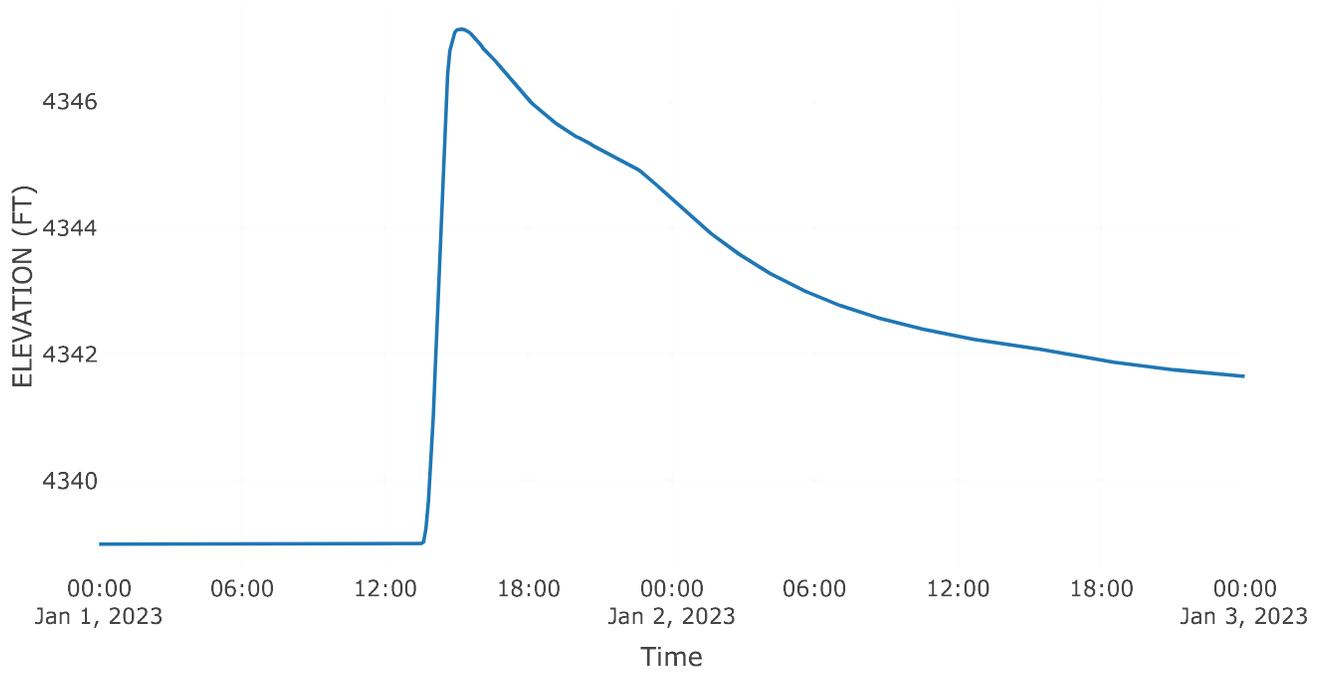
### Spillway 1



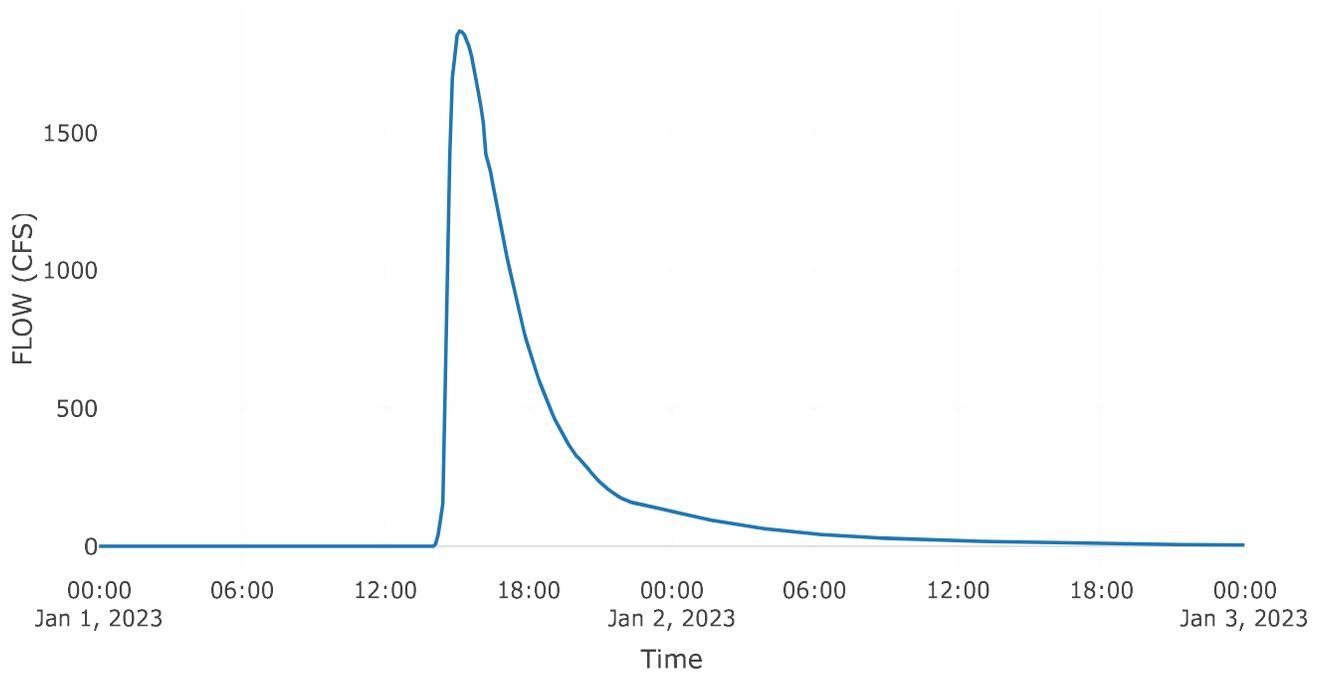
### Outlet 1



### Pool Elevation



### Outflow

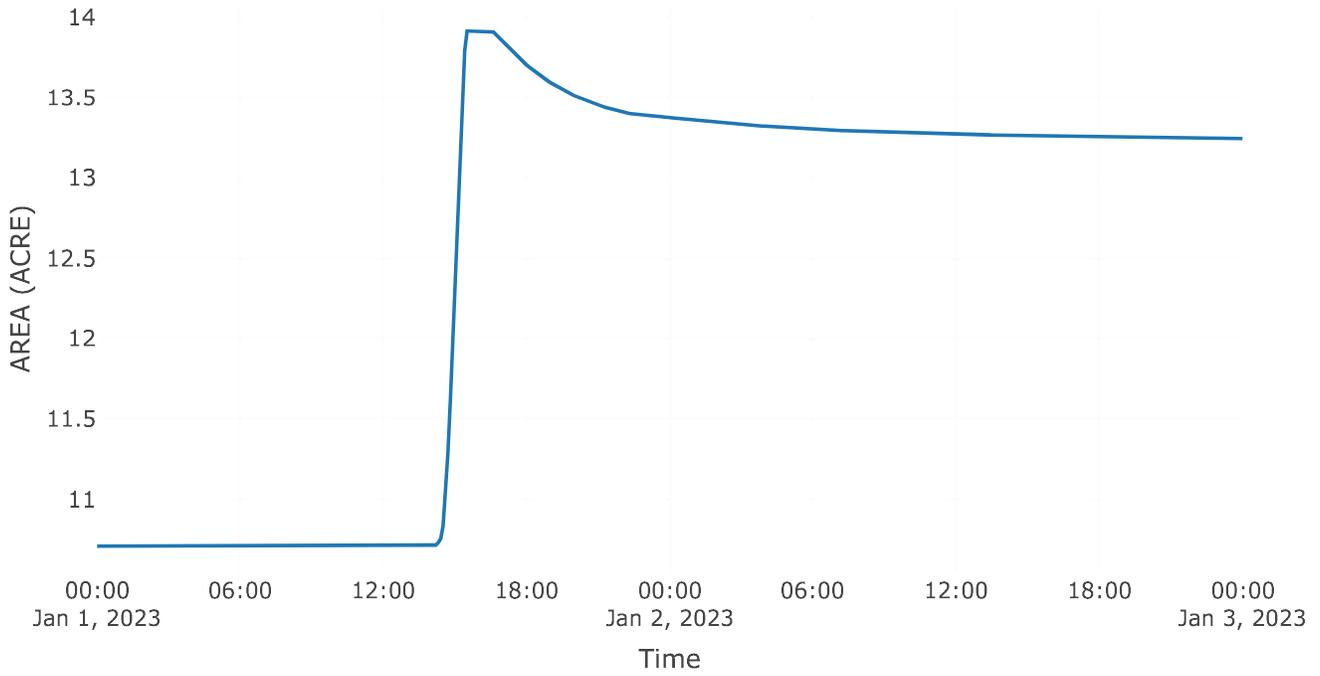


## Reservoir: Geurts Basin\_Lower

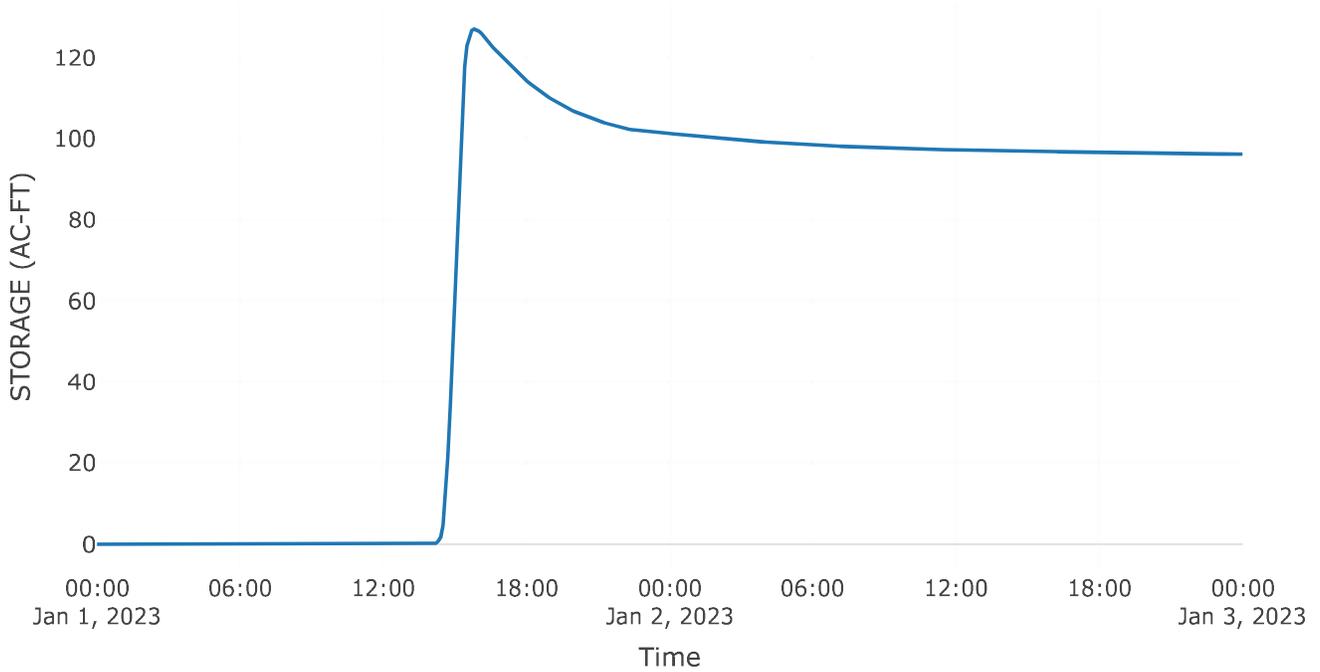
### Results: Geurts Basin\_Lower

Peak Discharge (CFS)	1689.59
Time of Peak Discharge	01Jan2023, 15:48
Peak Inflow (CFS)	1868.56
Time of Peak Inflow	01Jan2023, 15:06
Inflow Volume (AC - FT)	602.79
Maximum Storage (AC - FT)	127.09
Peak Elevation (FT)	4340.31
Discharge Volume (AC - FT)	506.63

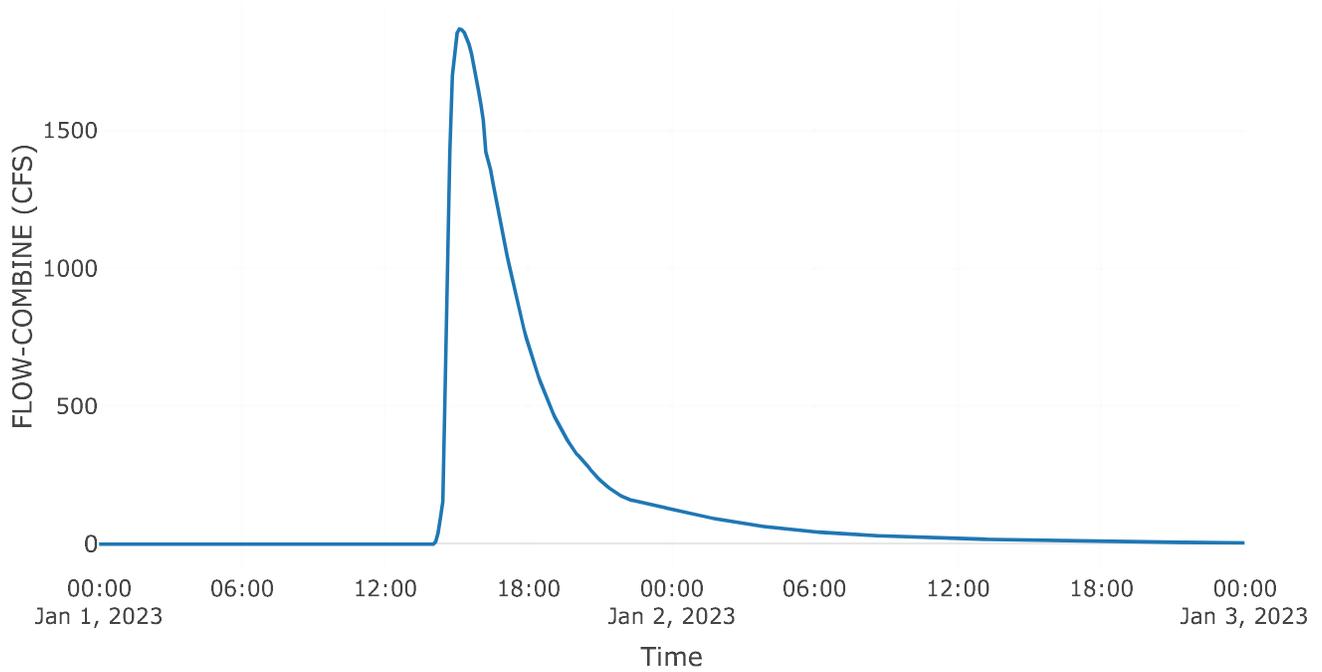
### Reservoir Area



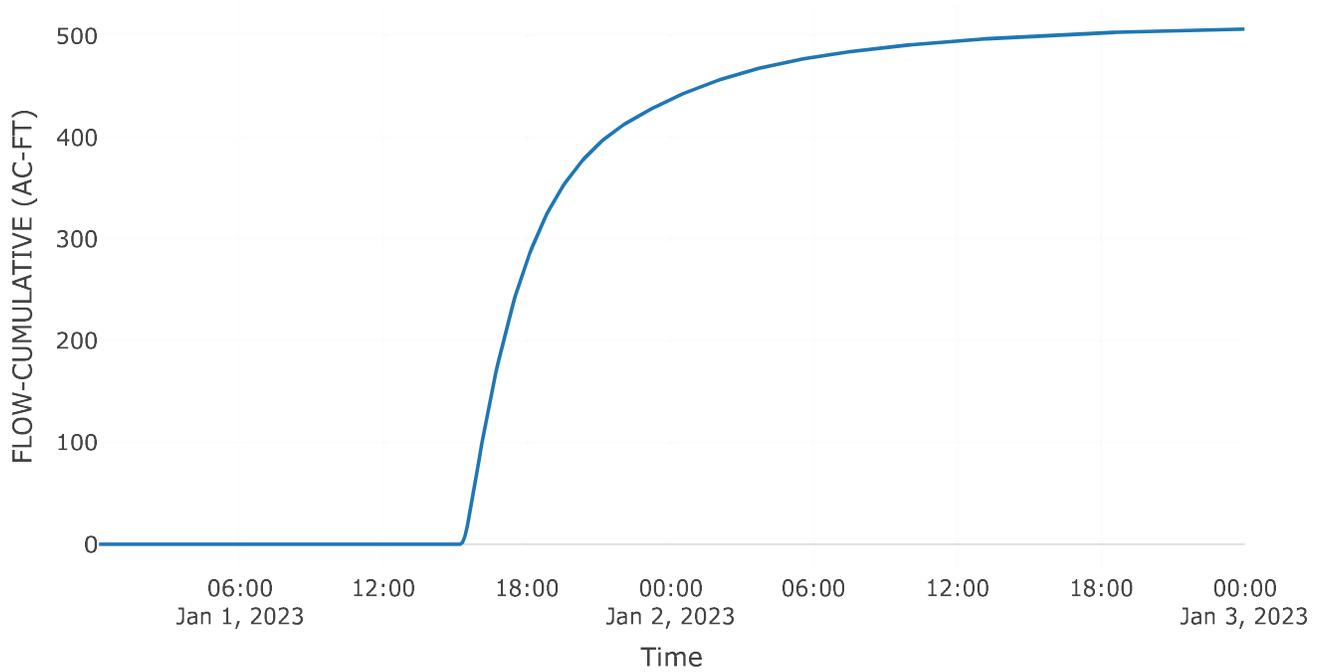
### Storage



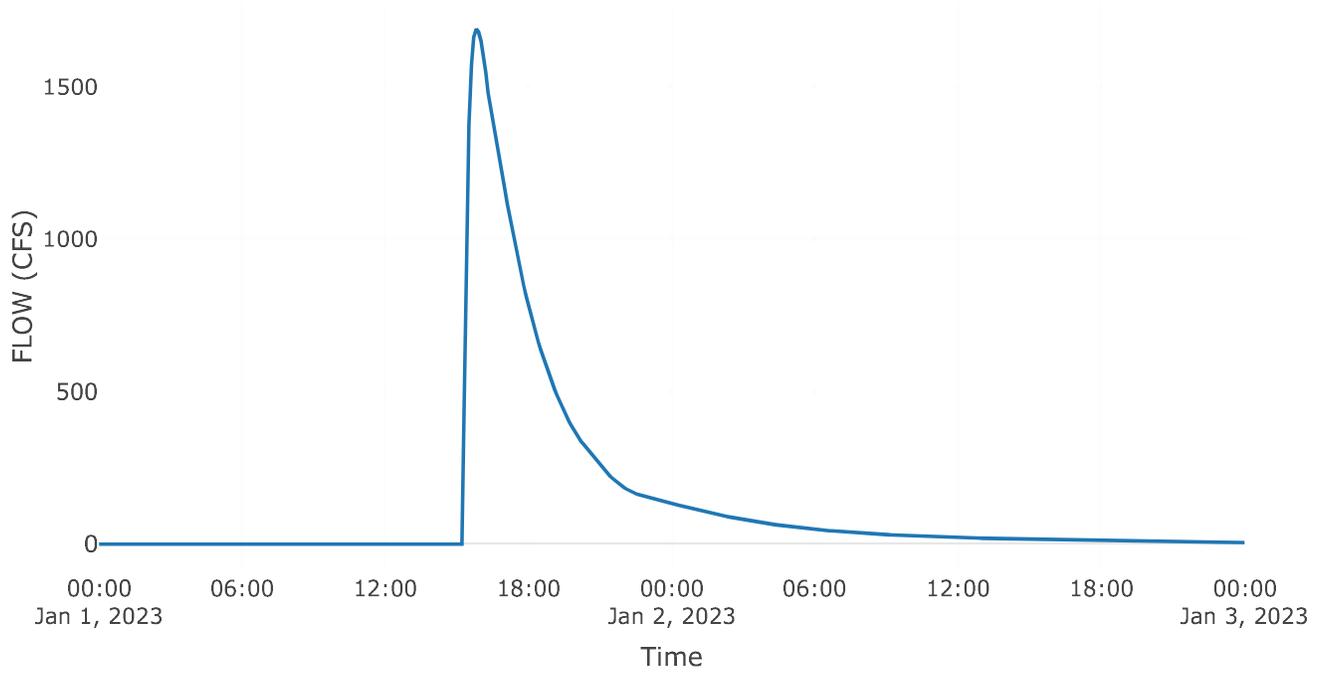
### Combined Inflow



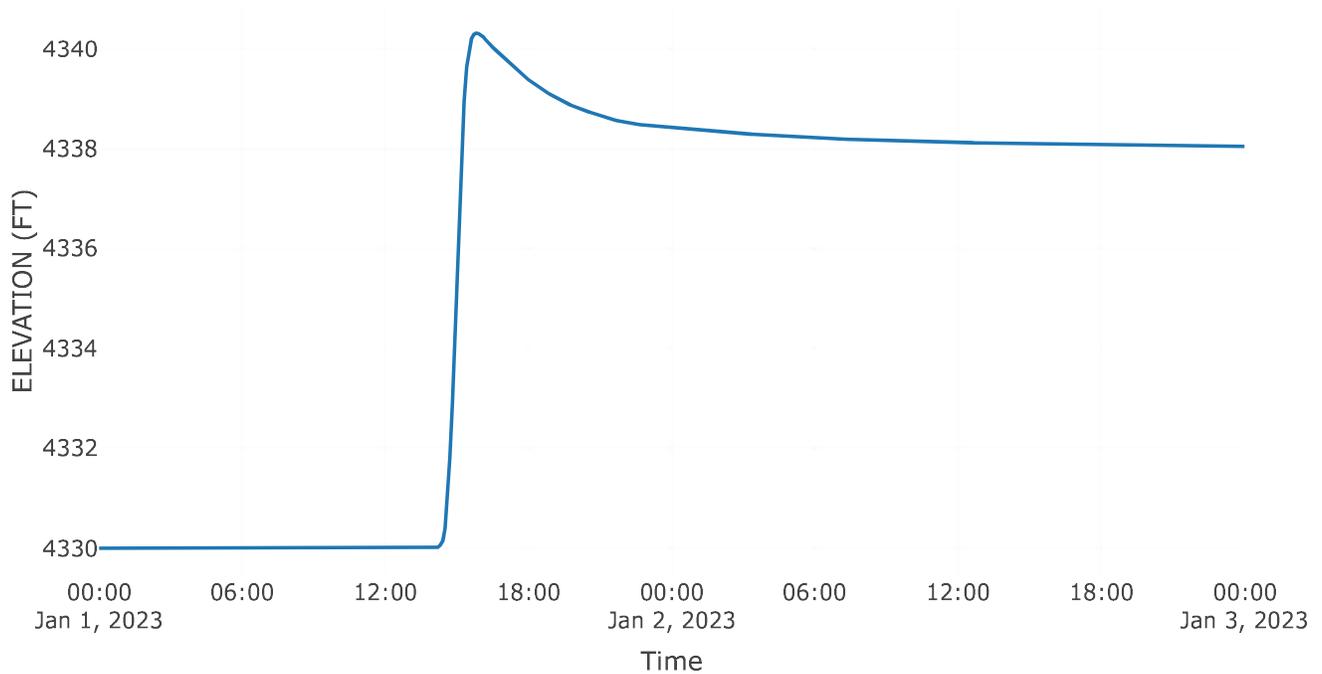
### Cumulative Outflow



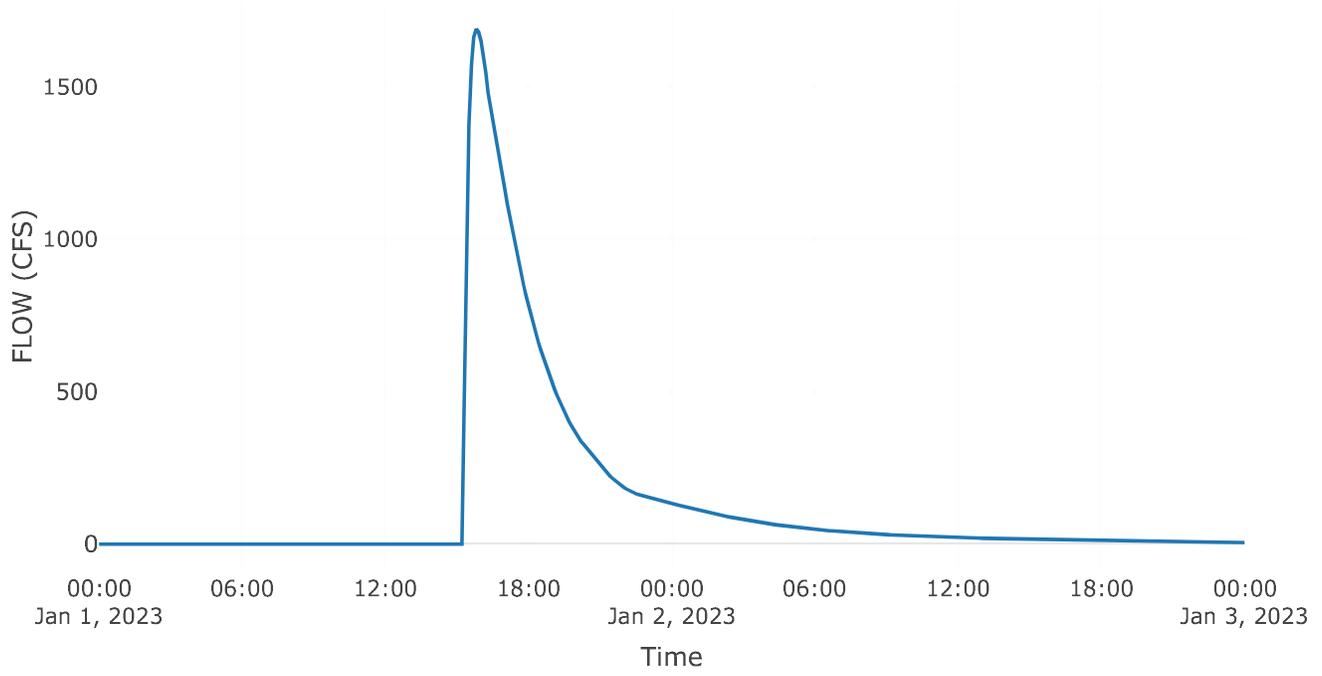
### Spillway 1



### Pool Elevation



### Outflow



# **Attachment C**

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*15% Design Drawings*







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 NOT FOR CONSTRUCTION  
 JUNE 2024

REV	DATE	DESCRIPTION

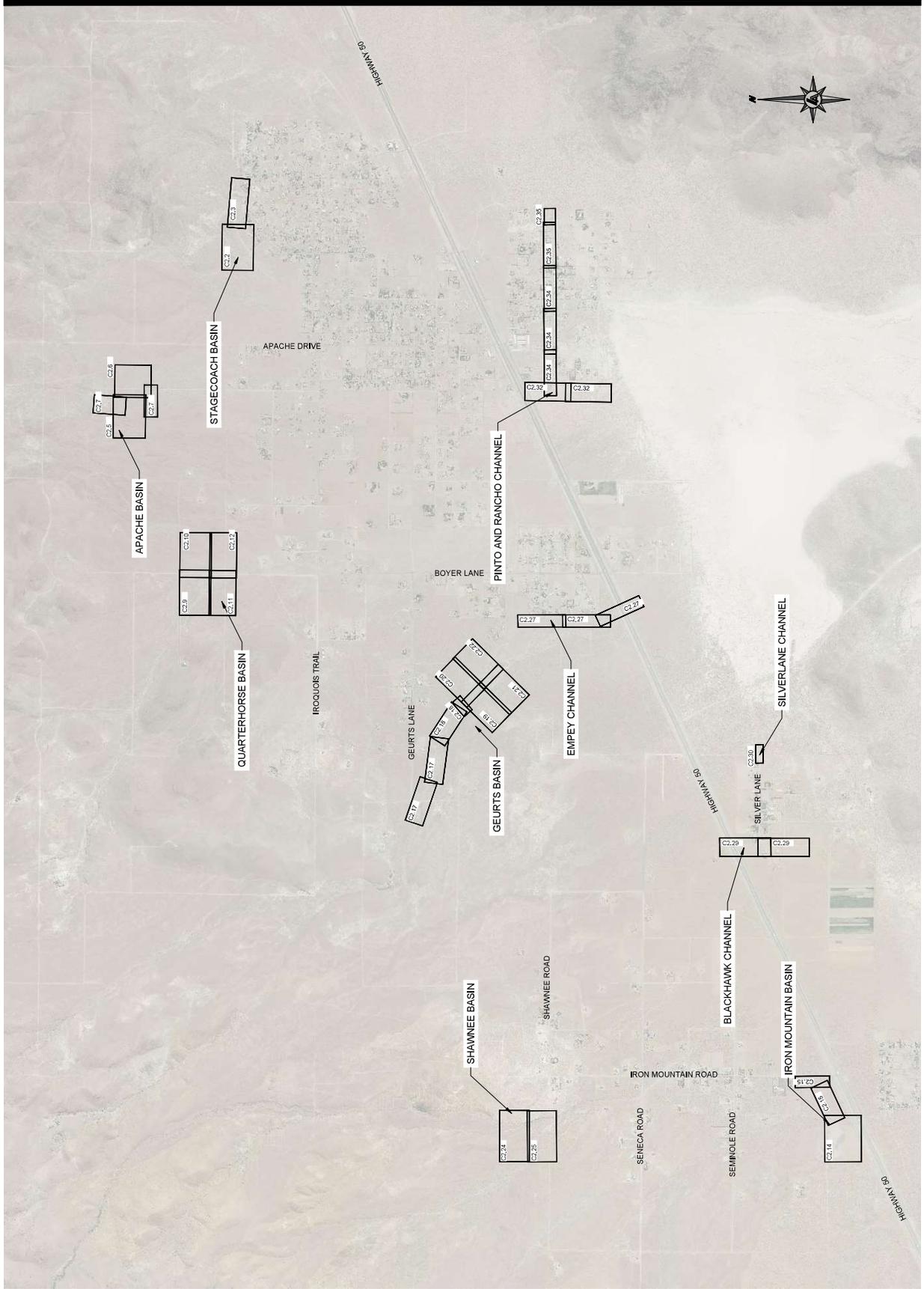
BY: \_\_\_\_\_

SCALE: 1"=100' OF ORIGINAL DRAWING

IF NOT ONE INCH ON THE SHEET, ADJUST SCALE ACCORDINGLY

**C0.3**

DRAWN BY: RHM  
 DESIGNED BY: RHM  
 CHECKED BY: JMO  
 JOB NO.: 103923000





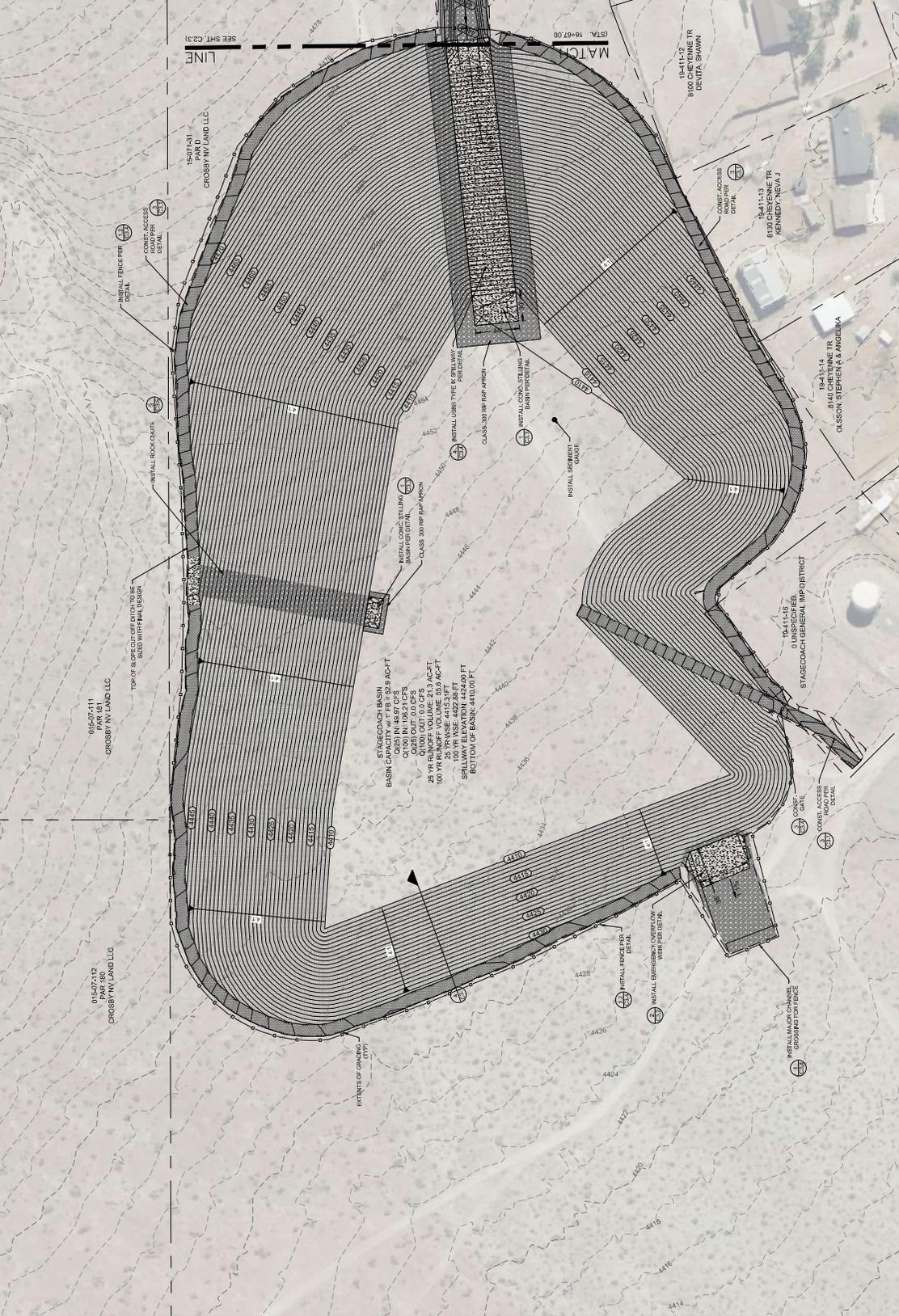


7/16/2024

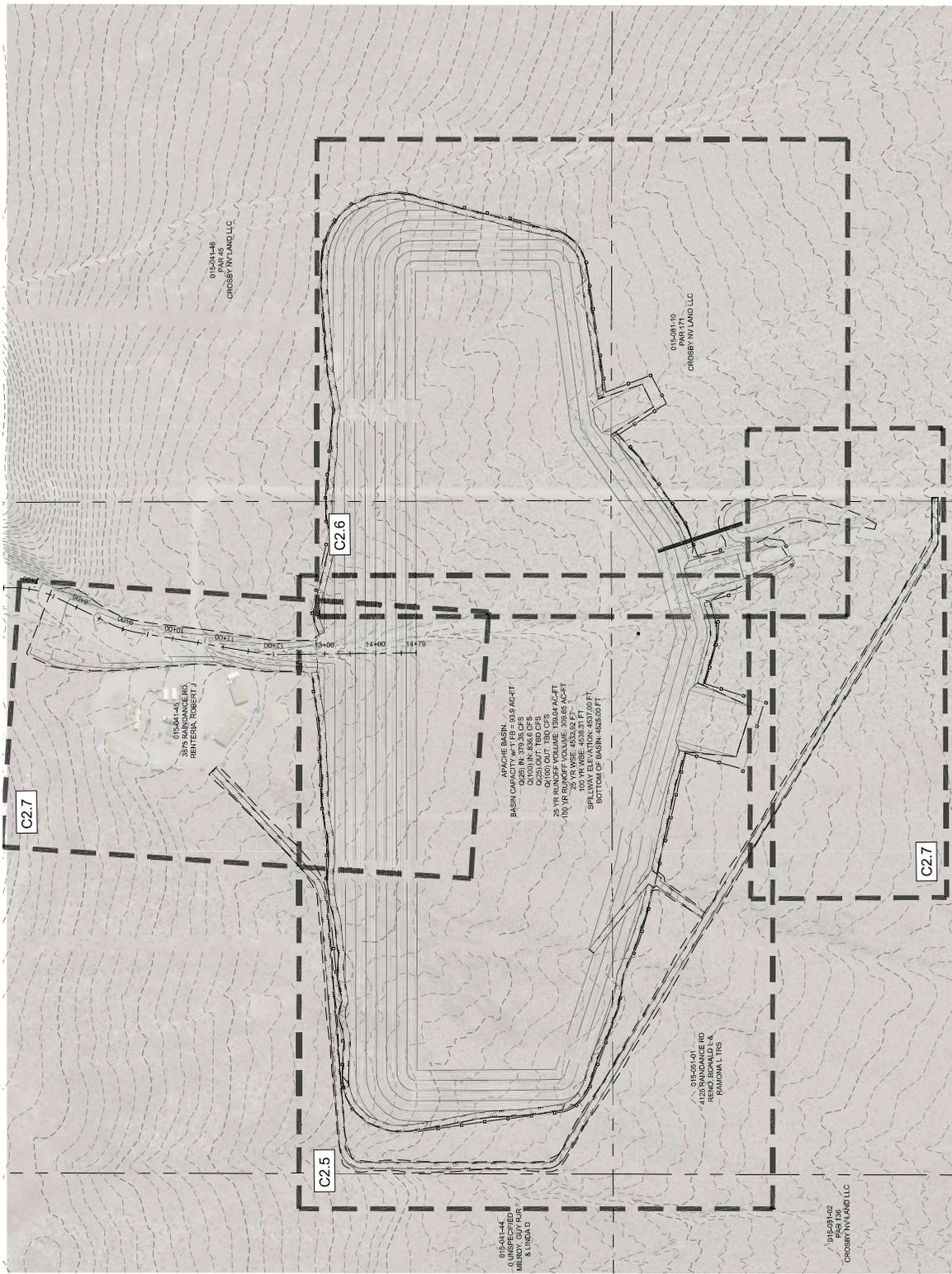
STAGECOACH-ADM  
 STAGECOACH BASIN  
 LYON COUNTY  
 NEVADA

REV.	DATE	DESCRIPTION

**C2.2**  
 DRAWN BY: IKM  
 DESIGNED BY: IKM  
 CHECKED BY: MMG  
 JOB NO.: 10522.000











7/16/2024

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LYON

STAGECOACH

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 APACHE BASIN

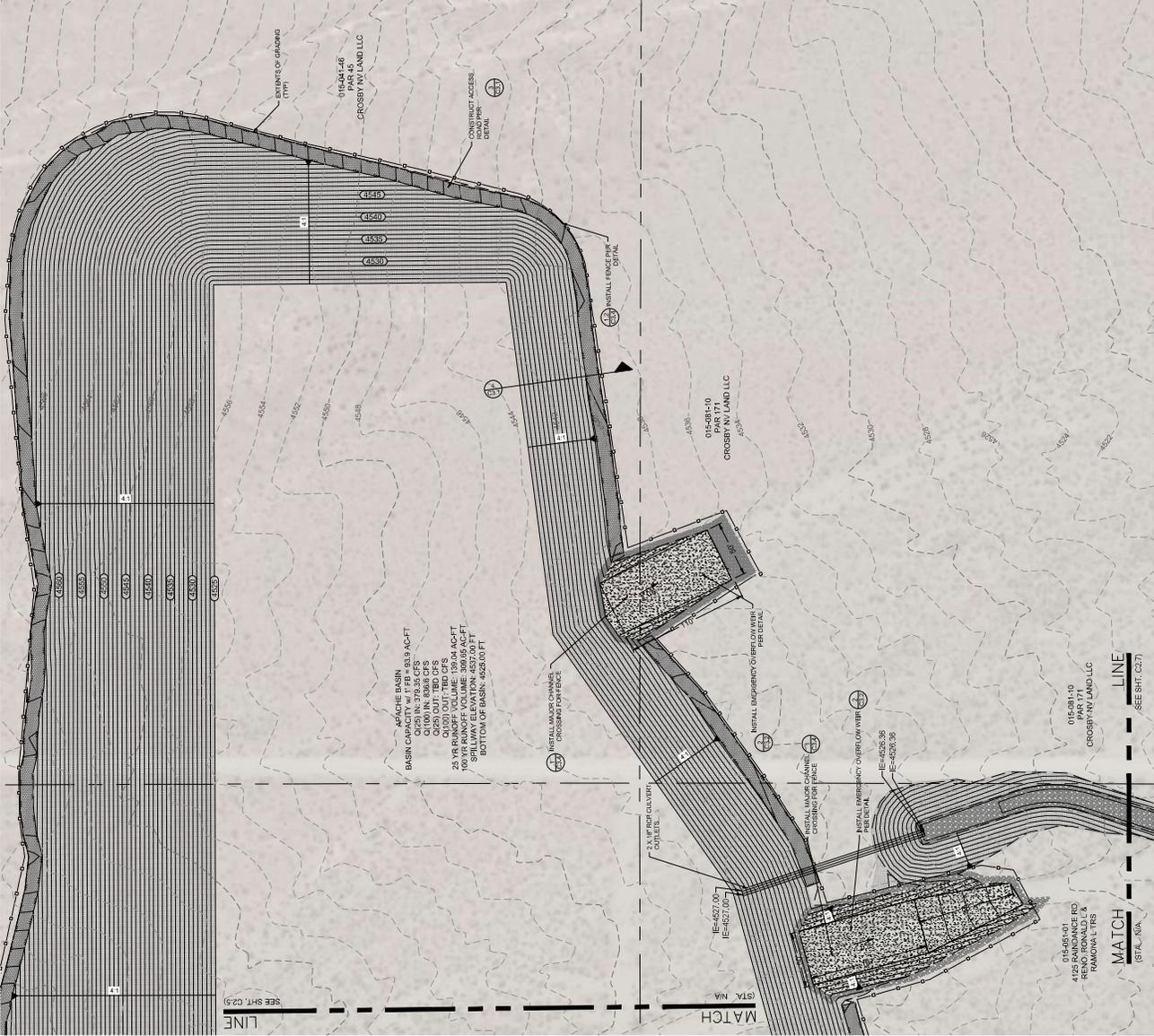
LYON COUNTY

15% SUBMITTAL  
 NOT FOR CONSTRUCTION  
 JUNE 2024

REV.	DATE	DESCRIPTION

**C2.16**

DRAWN BY: IKM  
 CHECKED BY: MMG  
 JOB NO.: 10522.000



APACHE BASIN  
 BASIN CAPACITY W/ 1' FB = 83.9 AC-FT  
 25 YR RAINFALL VOLUME = 190.04 AC-FT  
 100 YR RAINFALL VOLUME = 300.95 AC-FT  
 25 YR RAINFALL VOLUME = 190.04 AC-FT  
 100 YR RAINFALL VOLUME = 300.95 AC-FT  
 BOTTOM OF BASIN = 4525.00 FT

015-081-10  
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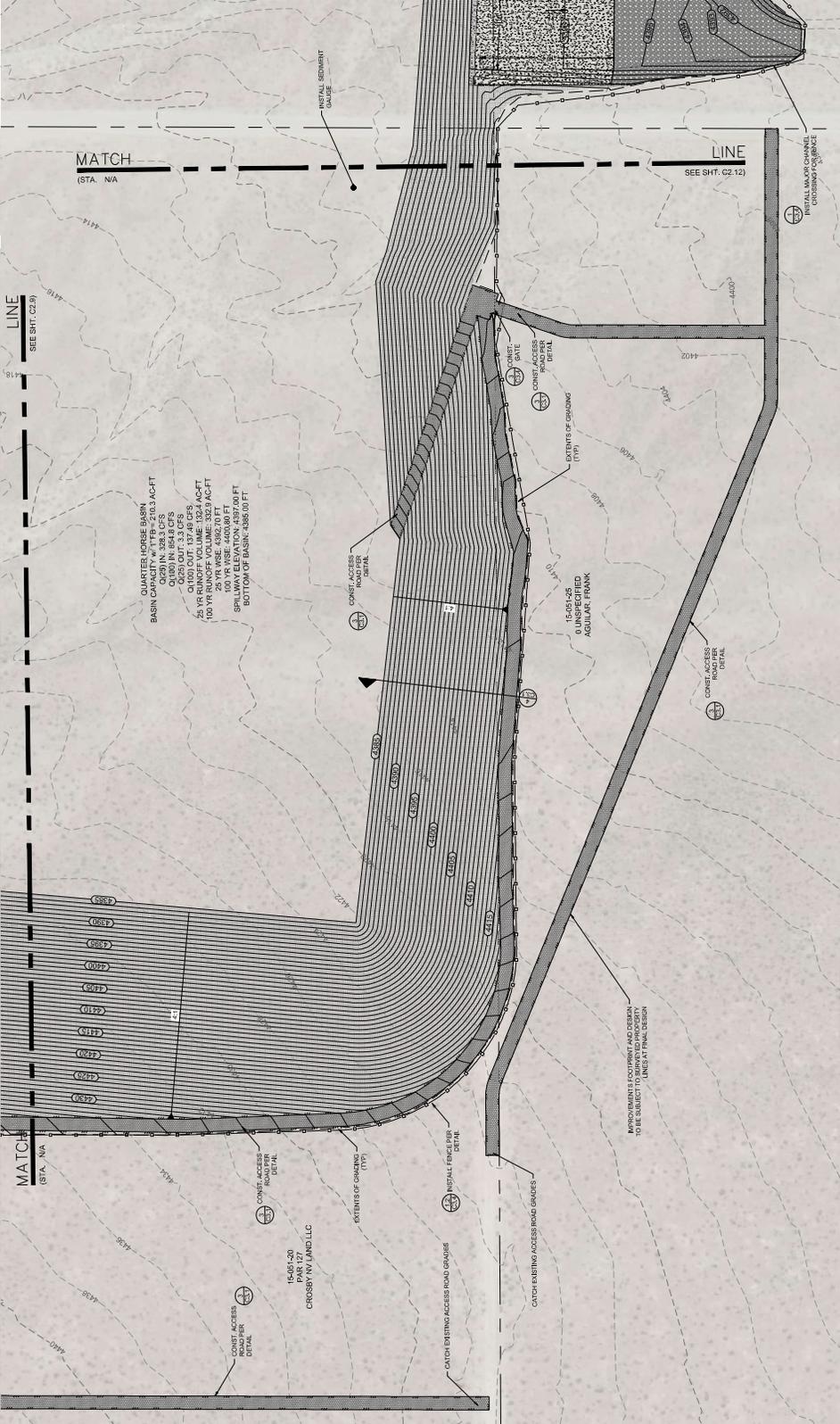


**STAGECOACH-ADMP**  
**QUARTERHORSE BASIN**

REV.	DATE	DESCRIPTION

DATE: JUNE 2024  
 NOT FOR CONSTRUCTION  
**15% SUBMITTAL**

**C2.11**  
 DRAWN BY: IKM  
 DESIGNED BY: MMG  
 JOB NO.: 10522.000







7/16/2024

STAGECOACH-ADMP  
 LYON COUNTY  
 IRONMOUNTAIN BASIN INDEX  
 STAGECOACH  
 LYON  
 NEVADA

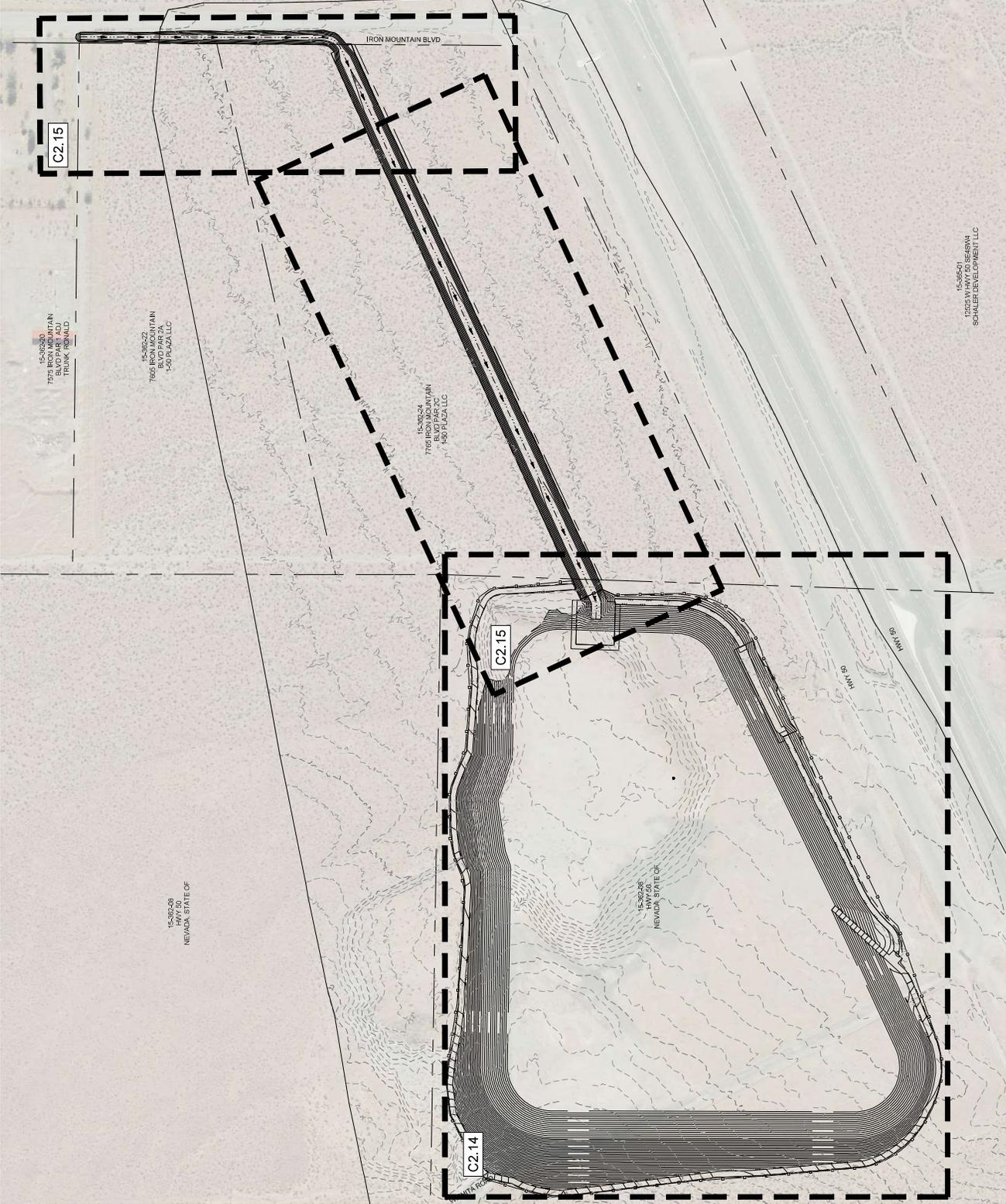
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 NOT FOR CONSTRUCTION  
 JUNE 2024

REV.	DATE	DESCRIPTION

BASED UPON ORIGINAL DRAWING  
 IF NOT ONE INCH ON THIS SHEET  
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**C2.13**

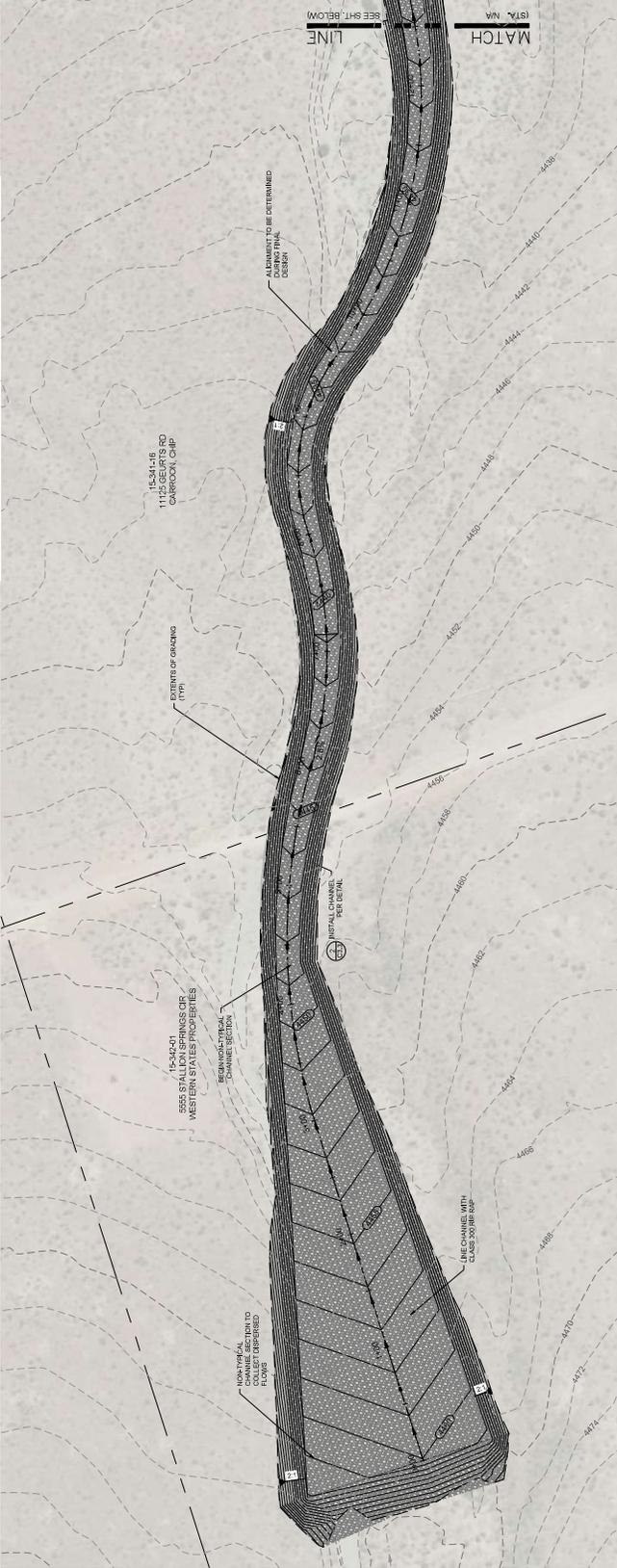
DRAWN BY: IKM  
 DESIGNED BY: IKM  
 CHECKED BY: MMG  
 JOB NO.: 10522.000



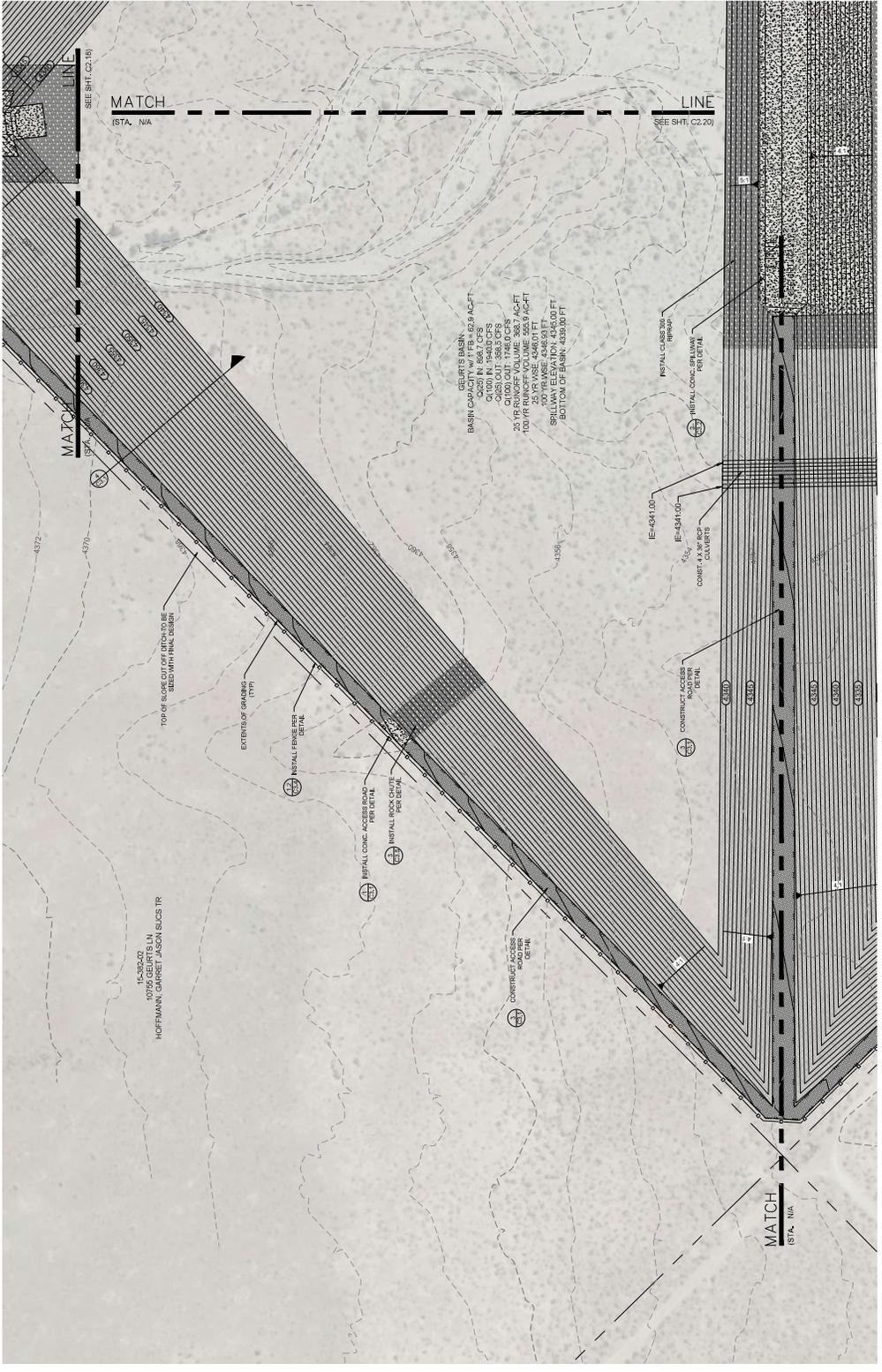
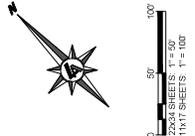


















7/16/2024

STAGECOACH-ADMP  
 GEURTS BASIN  
 LYON COUNTY  
 STAGECOACH NEVADA

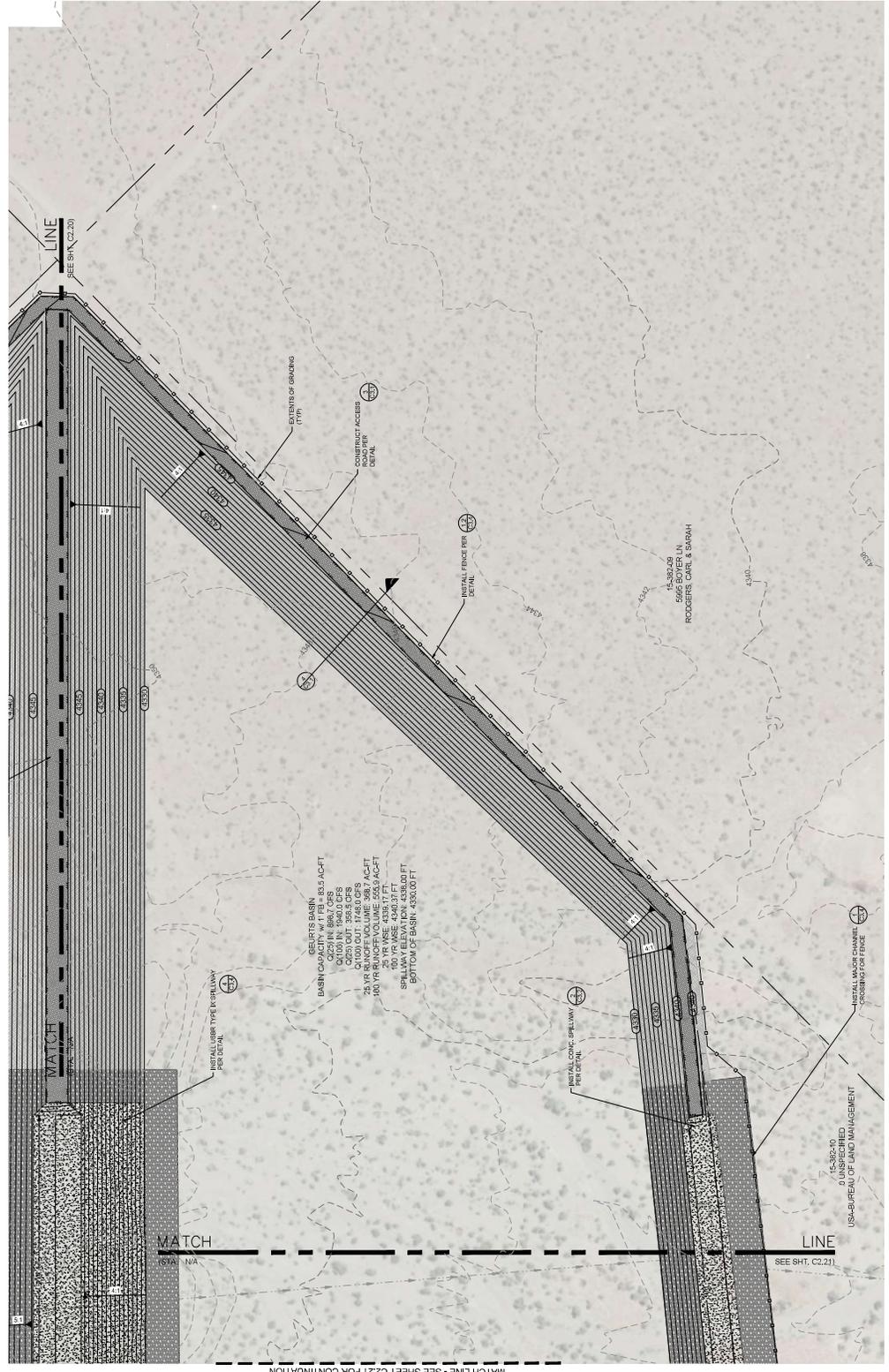
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**15% SUBMITTAL**  
 NOT FOR CONSTRUCTION  
 JUNE 2024

DATE: 6/20/24  
 DRAWN BY: IKM  
 CHECKED BY: MMG  
 JOB NO.: 10527000



0 50 100  
 2234 SHEETS: T=50  
 11411 SHEETS: T=100







7/16/2024

STAGECOACH-ADMP  
 SHAWNEE BASIN  
 LYON COUNTY  
 STAGECOACH  
 LYON

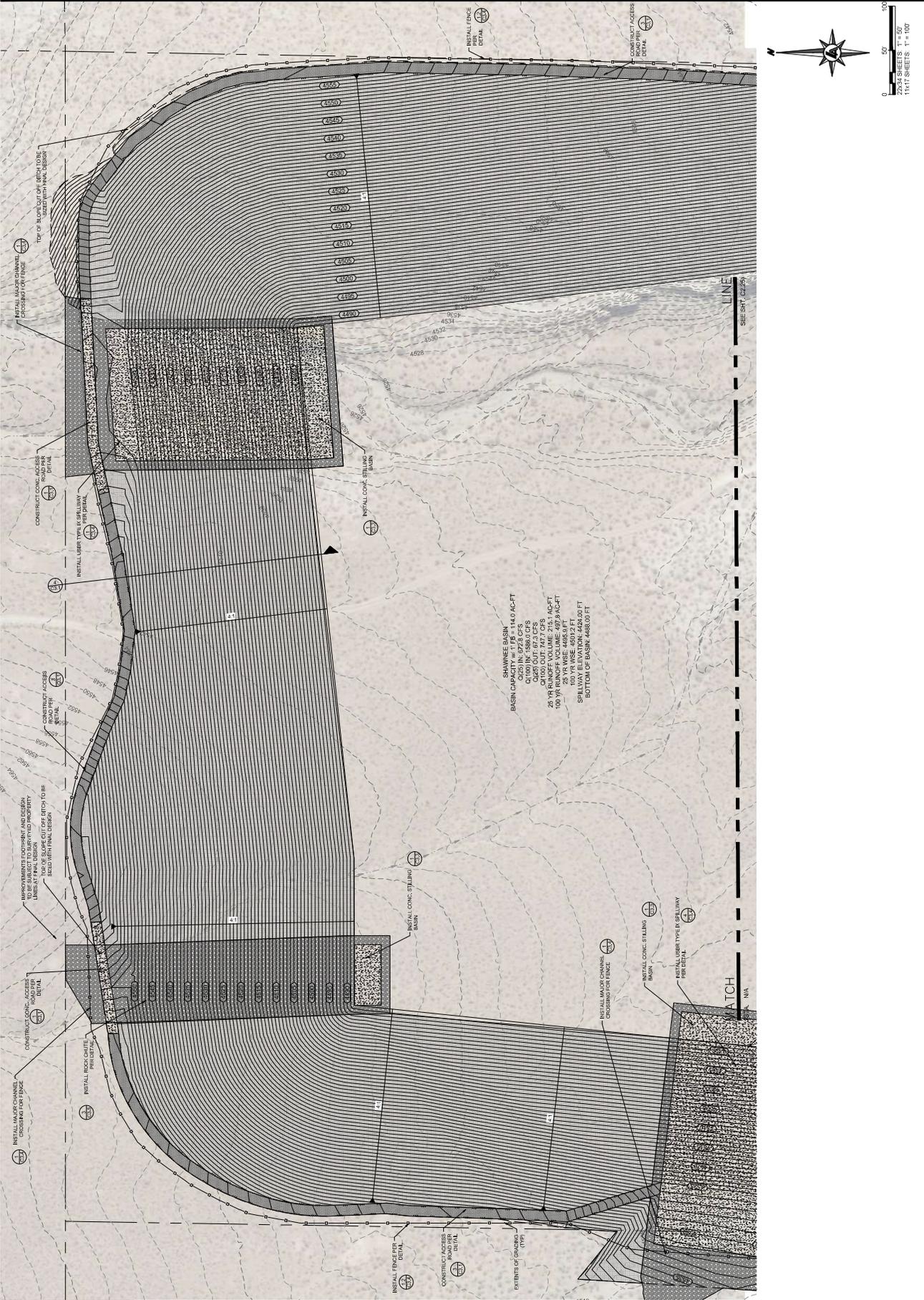
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 JUNE 2024

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 IF NOT ONE INDICATED THIS SHEET  
 SHALL BE AS SHOWN

**C2.24**

DRAWN BY: IKM  
 DESIGNED BY: IKM  
 CHECKED BY: MMG  
 JOB NO.: 1052200



**SHAWNEE BASIN** 1140 AC-FT  
 BASIN CAPACITY 1140 AC-FT  
 100 YR WISE 4980.22 FT  
 25 YR WISE 4485.91 FT  
 20 YR RUNOFF VOLUME 715.1 AC-FT  
 100 YR WISE 4980.22 FT  
 25 YR WISE 4485.91 FT  
 20 YR WISE 4485.91 FT  
 BOTTOM OF BASIN 4488.00 FT







7/16/2024

NEVADA

STAGECOACH-ADMP  
 EMPY CHANNEL  
 LYON COUNTY

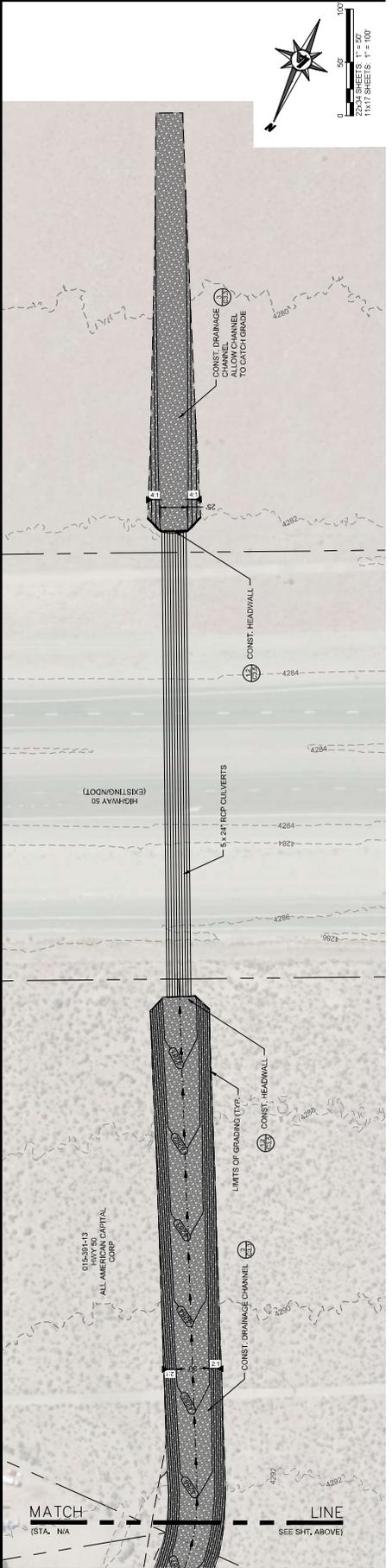
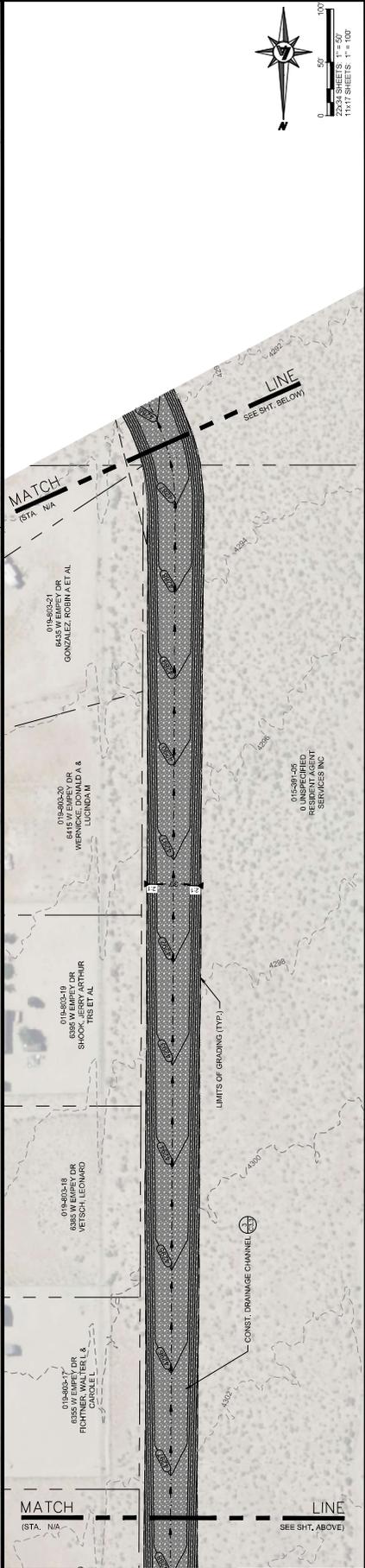
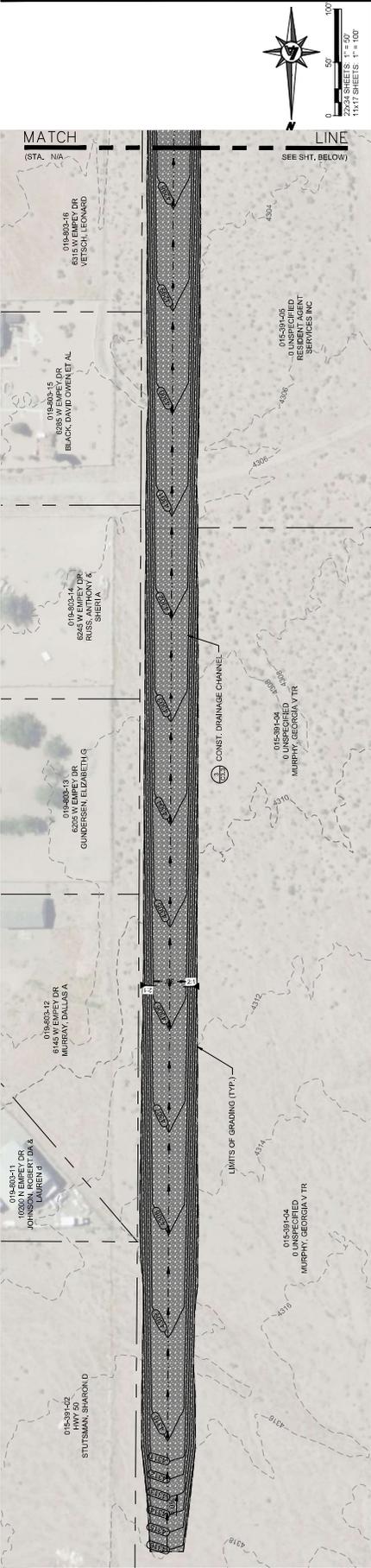
STAGECOACH

LYON

15% SUBMITTAL  
 NOT FOR CONSTRUCTION  
 JUNE 2024

REV.	DATE	DESCRIPTION

IF NOT ONE INCH ON THIS SHEET, SCALE SHALL BE AS SHOWN.  
**C2.27**  
 DRAWN BY: IKM  
 DESIGNED BY: IKM  
 CHECKED BY: MMG  
 JOB NO.: 10522.000









7/16/2024

NEVADA

STAGECOACH-ADMP  
 LYON COUNTY  
 SILVERLANE CHANNEL

STAGECOACH

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 NOT FOR CONSTRUCTION  
 JUNE 2024

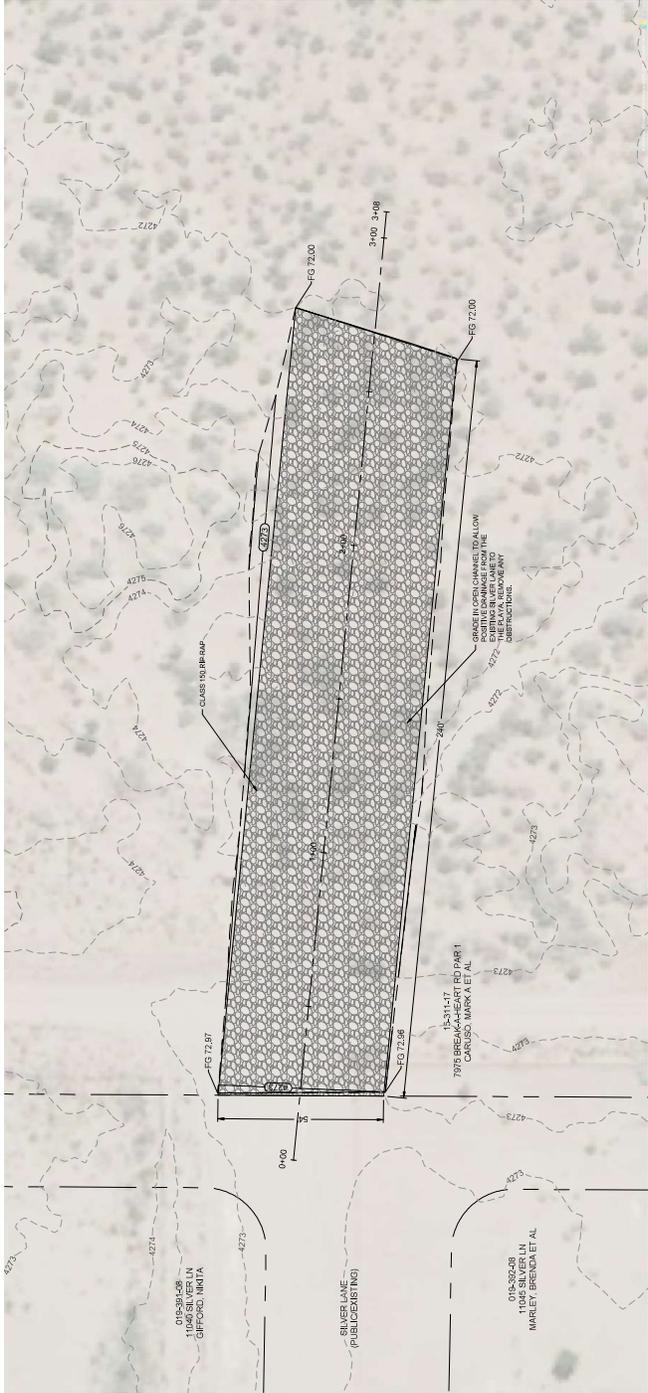
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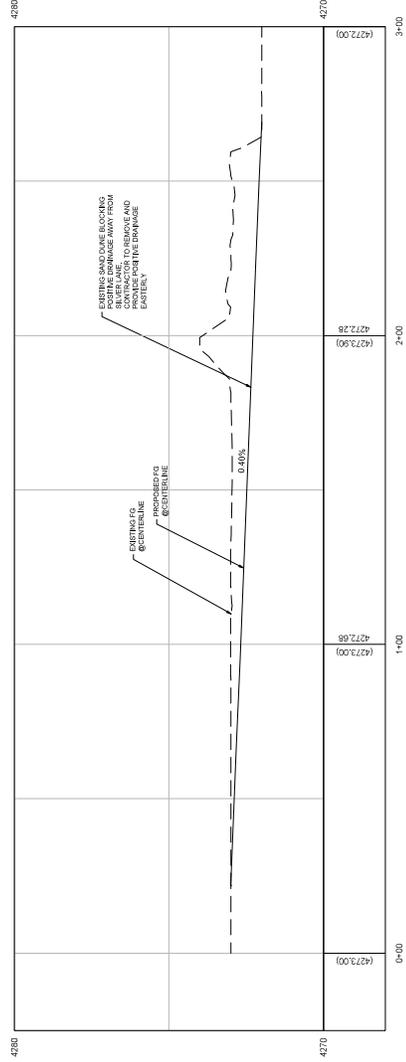
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 SHOWS SCALE ACCURACY

**C2.30**

DRAWN BY: IKM  
 DESIGNED BY: MMG  
 CHECKED BY: MMG  
 JOB NO.: 10522.000



SILVER LANE CHANNEL



HORIZONTAL SCALE  
 1" = 20'

VERTICAL SCALE  
 1" = 2'

11x17 SHEETS: 1" = 40'  
 22x34 SHEETS: 1" = 20'  
 11x17 SHEETS: 1" = 4'



7/16/2024

NEVADA

STAGECOACH-ADMP  
 PINTO CHANNEL INDEX

LYON COUNTY

STAGECOACH

LYON

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 JUNE 2024

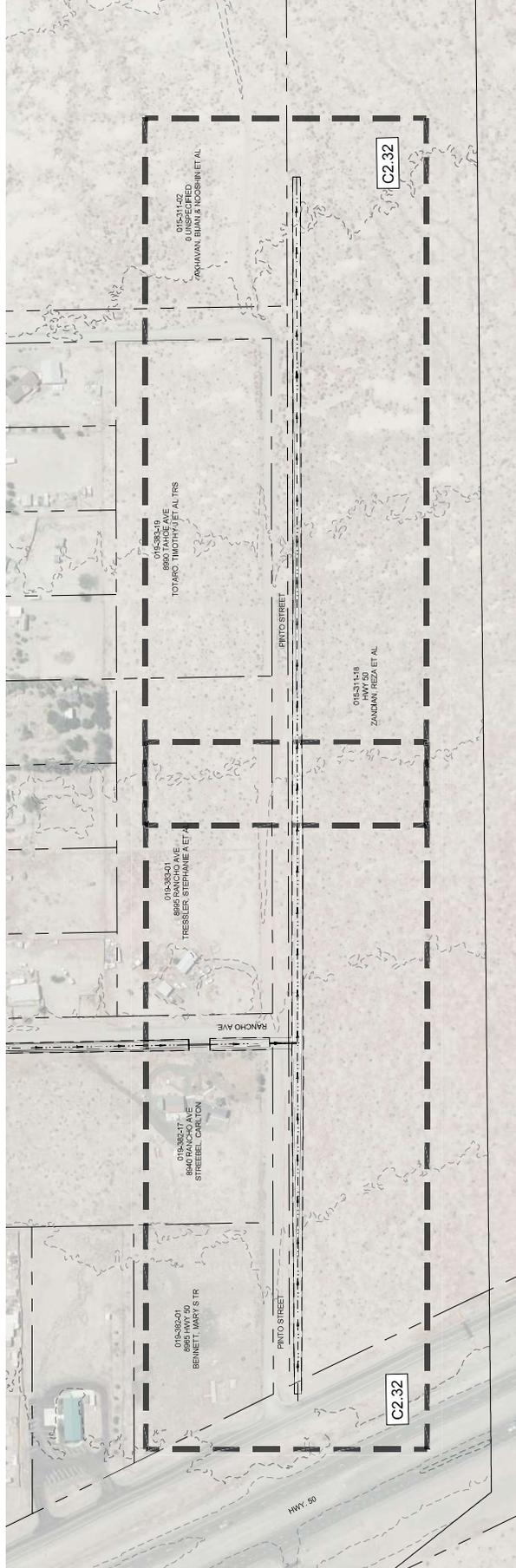
REV.	DATE	DESCRIPTION

BASED UPON ORIGINAL DRAWING

IF NOT ONE INCH ON THIS SHEET,  
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 DESIGNED BY: MMG  
 CHECKED BY: MMG  
 JOB NO.: 10522.000







7/16/2024

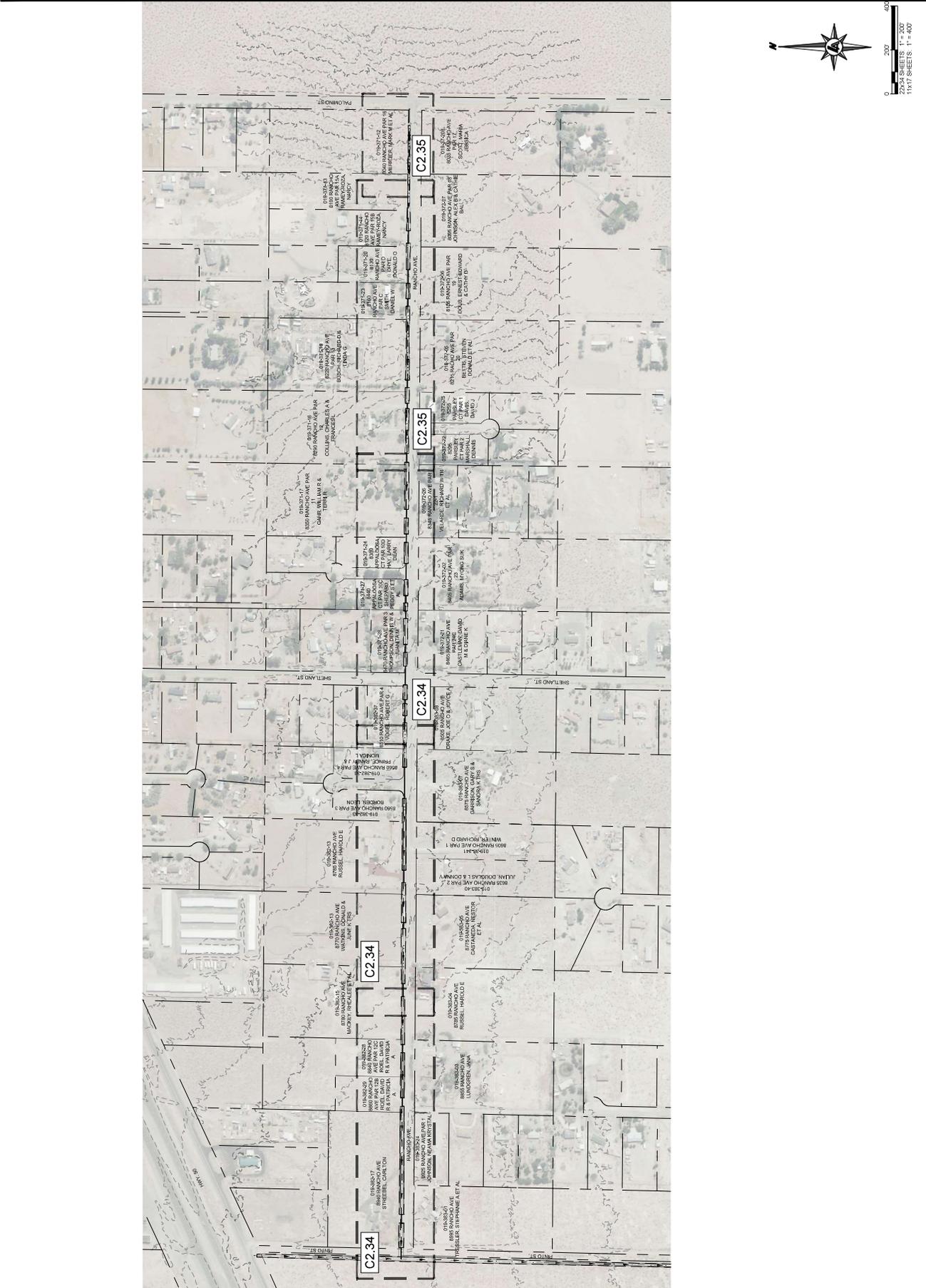
STAGECOACH-ADMP  
 RANCHO CHANNEL INDEX  
 LYON COUNTY  
 STAGECOACH  
 LYON

REV.	DATE	DESCRIPTION

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 TIME: 10:00 AM  
 PROJECT: STAGECOACH-ADMP  
 SHEET: RANCHO CHANNEL INDEX  
 DRAWN BY: [Name]  
 CHECKED BY: [Name]  
 DATE: 06/24/2024

**C2.33**  
 DRAWN BY: [Name]  
 CHECKED BY: [Name]  
 DATE: 06/24/2024

IF NOT ONE INCH ON THIS SHEET,  
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0 100 200  
 20x34 SHEETS 1" = 200'  
 11x17 SHEETS 1" = 400'

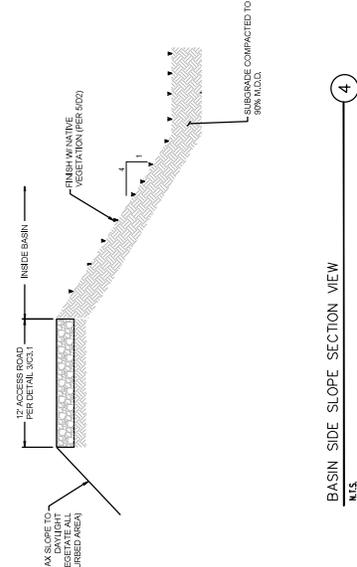
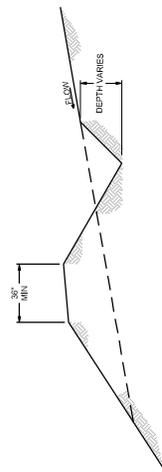
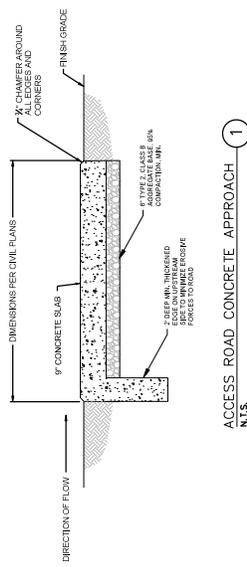
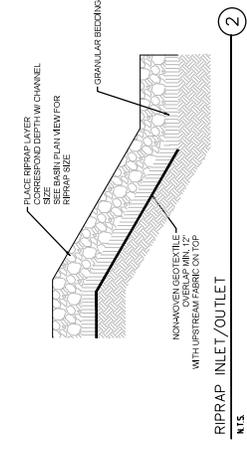
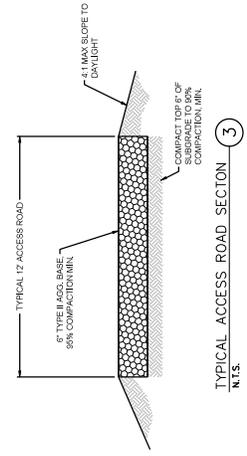


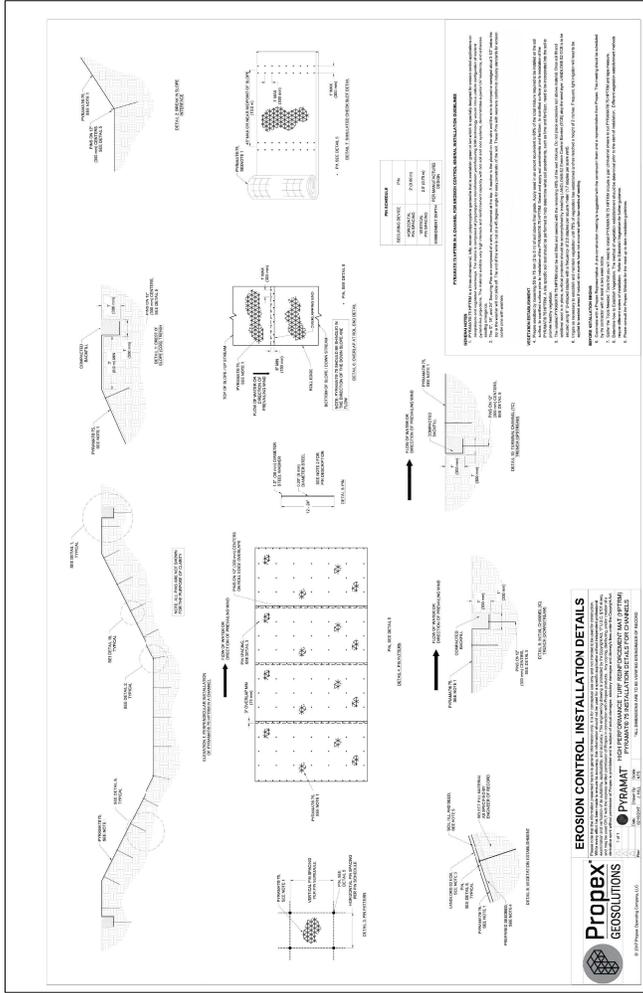


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 NOT FOR CONSTRUCTION  
 JUNE 2024

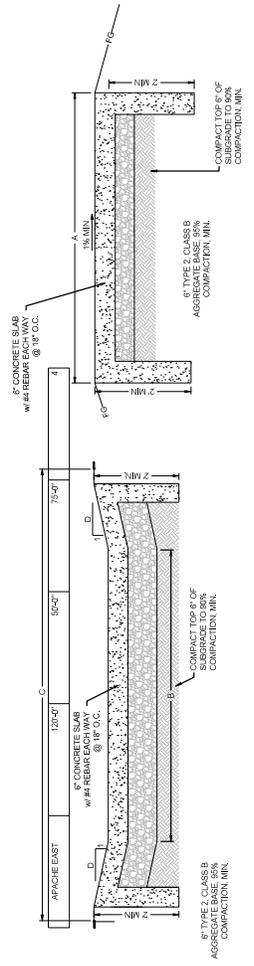




1  
 PYRAMAT SLOPE STABILIZATION  
 N.E.S.

SPILLWAY PARAMETERS				
SPILLWAY	A	B	C	D
IRONHOLDFURN	250'-0"	188'-0"	208'-0"	10
SHAWHEE	84'-0"	100'-0"	145'-0"	4
GELATS UPPER	44'-0"	300'-0"	325'-0"	4
GELATS LOWER	250'-0"	300'-0"	324'-0"	4
QUARTER HORSE	125'-0"	100'-0"	150'-0"	4
STAGECOACH SPILLWAY	38'-0"	50'-0"	65'-0"	4
STAGECOACH BR	150'-0"	63'-0"	87'-0"	4
APACHE WEST	125'-0"	100'-0"	135'-0"	4
APACHE CENTRAL	200'-0"	50'-0"	78'-0"	4

2  
 SPILLWAY W/ CONCRETE LINING  
 N.E.S.



NOTES:  
 1. REFER TO BE SPECIFIED AT FINAL DESIGN WITH GEOTECHNICAL RECOMMENDATIONS







## **APPENDIX B**

### **Supporting Data (Digital)**