

Existing Conditions Hydraulic Analysis Report: Montoyas Arroyo, Harvey Jones Channel

February 10, 2026



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**EXISTING CONDITIONS HYDRAULIC ANALYSIS REPORT FOR THE MONTOYAS ARROYO
FROM THE HARVEY JONES CHANNEL INLET TO THE RIO GRANDE**

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EXECUTIVE SUMMARY

The Southern Sandoval County Arroyo Flood Control Authority (SSCAFCA) contracted Bohannon Huston, Inc. (BHI) to perform an existing conditions two-dimensional (2D) hydraulic analysis of the Harvey Jones Channel (HJC) and adjacent irrigation channels within the Village of Corrales, New Mexico. The HJC conveys runoff from the Montoyas and Lomitas Negras arroyos, draining approximately 60 square miles. Although the channel was originally designed to convey up to 8,000 cubic feet per second (cfs) and multiple projects have implemented measures to increase the capacity of the Corrales Road (NM 448) bridge, neighborhood concerns have limited the extent to which the structure can be raised, thereby restricting the amount of conveyance achievable. Consequently, the bridge continues to act as a hydraulic bottleneck.

This study uses inflow hydrographs extracted from the SSCAFCA Montoyas Watershed Park Management Plan [Montoyas WMP] (2021) HEC-HMS model in a 2D HEC-RAS hydraulic model. The Montoyas WMP HEC-HMS model was updated as part of the *Montoyas Arroyo Reach Stabilization Plan (RSP), Broadmoor Blvd to NM Hwy 528* (BHI, 2024) to simulate the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year storm events. This model contains various structures throughout the model domain based on field survey observations, and the downstream boundary condition is governed by Federal Emergency Management Agency's (FEMA) expected base flood elevation (BFE) in the Rio Grande, giving conservative mapping results in the model domain. This model's results will inform future SSCAFCA projects by characterizing hydraulic performance of critical infrastructure, delineating inundation limits under existing conditions, and identifying structures at risk of flooding.

The analysis confirms that the Corrales Road bridge is significantly undersized relative to the channel capacity and *New Mexico Department of Transportation Drainage Design Manual* (NMDOT DDM) (Smith Engineering Company and Occam Engineers Inc., 2018) freeboard requirements. The hydraulic model indicates that the crossing can convey a maximum flowrate of 6,130 cfs; however, before maximum conveyance is achieved, backwater from the bridge overtops the HJC, resulting in extensive flooding of adjacent properties during the 100- and 500-year events.

In addition to the channel overtopping at the Corrales Road bridge, the hydraulic model indicates three additional HJC wall sections overtopping during the 500-year event, as shown in **Figure 6**. Overtopping in the 500-year event was expected, as the HJC was only designed to convey the 100-year storm.

The first additional 500-year overtopping channel wall section occurs near the HJC inlet, where the southern wall is lower than the northern wall and the overtopping water escapes to the south. Additional modeling would be needed to evaluate how raising this wall might shift breakout locations downstream, toward the Corrales Road bridge.

The second and third additional 500-year overtopping channel wall sections occur approximately 250 feet downstream of the HJC confluence with the Dulcelina Channel. In this area, the channel walls on both sides of the HJC are overtopped, causing floodwater to spill onto the adjacent properties to the north and south.

Inundation limits and a list of inundated buildings were produced for the 10-, 50-, 100-, and 500-year events. Field reconnaissance was performed to validate finished floor elevations (FFE), allowing for FFE adjustments where appropriate, based on the number of steps to a building's front door and an assumption of step heights. This data provides the basis for SSCAFCA benefit-cost analysis and future mitigation planning.

Overall, the results confirm that the Corrales Road bridge is the primary driver of flood risk in this reach of the HJC and that major flooding occurs during the 100-year storm event due to the restricted bridge conveyance, while in the 500-year storm event the limited capacity of both the crossing and the channel leads to a substantially greater extent of inundation.

1 INTRODUCTION

The Harvey Jones Channel (HJC) conveys runoff from the Montoyas and Lomitas Negras arroyos, draining approximately 60 square miles. Constructed in 1987 by the Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service (SCS), the HJC was designed to protect the Village of Corrales from flooding by accommodating up to 8,000 cubic feet per second (cfs) of runoff from the upstream watershed. This channel has a history of flooding and sedimentation problems that have been studied and documented for decades.

The *HJC Hydraulic Capacity Analysis* (Bohannon Huston, Inc. (BHI), 1999) found that using U.S. Army Corps of Engineers (USACE) assumptions, only 5,000 cfs would pass through the Corrales Road (NM 448) bridge. This study identified a sedimentation problem in the crossing due to the stilling basin's sill downstream of the crossing. This study also found that the stilling basin's sill could decrease the conveyance capacity of the crossing to only 3,000 cfs, if full.

The *Evaluation of Alternatives for Raising the Bridge at Corrales Road* (BHI, 2003) discussed three (3) alternatives to increase the conveyance capacity of the crossing. The preferred alternative included raising the low chord of the bridge by 2 feet 8 inches. This alternative was said to be able to increase the bridge conveyance to 6,000 cfs. Due to the sensitivity of visual impacts in the area, the bridge and roadway elevation must remain essentially in its current vertical condition, thus, this alternative was not implemented.

Improvements were made to the crossing in 2013 after large floods filled the HJC outfall to the top of the channel walls and large flood waters inundated adjacent properties. The area was analyzed and the design was conceptualized in the *Drainage Analysis Report – Harvey Jones Channel Improvements* (BHI, 2013). The modifications proposed and later implemented included: (1) remove the existing 3 feet concrete adverse sloped sill and construct an earthen sediment pond within the existing HJC outlet, and (2) construct a trapezoidal channel incised within the existing vertical wall HJC, positively sloped to continue toward the sediment pond. This crossing modification was calculated to increase the bridge's conveyance capacity to 6,800 cfs. Though SSCAFCA has not observed an event as significant as previous events since this modification was made, the crossing's capacity is still far lower than the capacity of HJC and the flowrates expected to reach the crossing.

The entire watershed was studied in the *Montoyas Watershed Park Management Plan* (SSCAFCA, 2021) to serve as a planning tool to mitigate the effects of anticipated urban development within the Montoyas Arroyo Watershed by identifying future regional drainage improvements. This study found that even with proposed regional improvements, the peak flow

reaching the Corrales Bridge was still above its existing capacity, thus, the Corrales Bridge must be improved further.

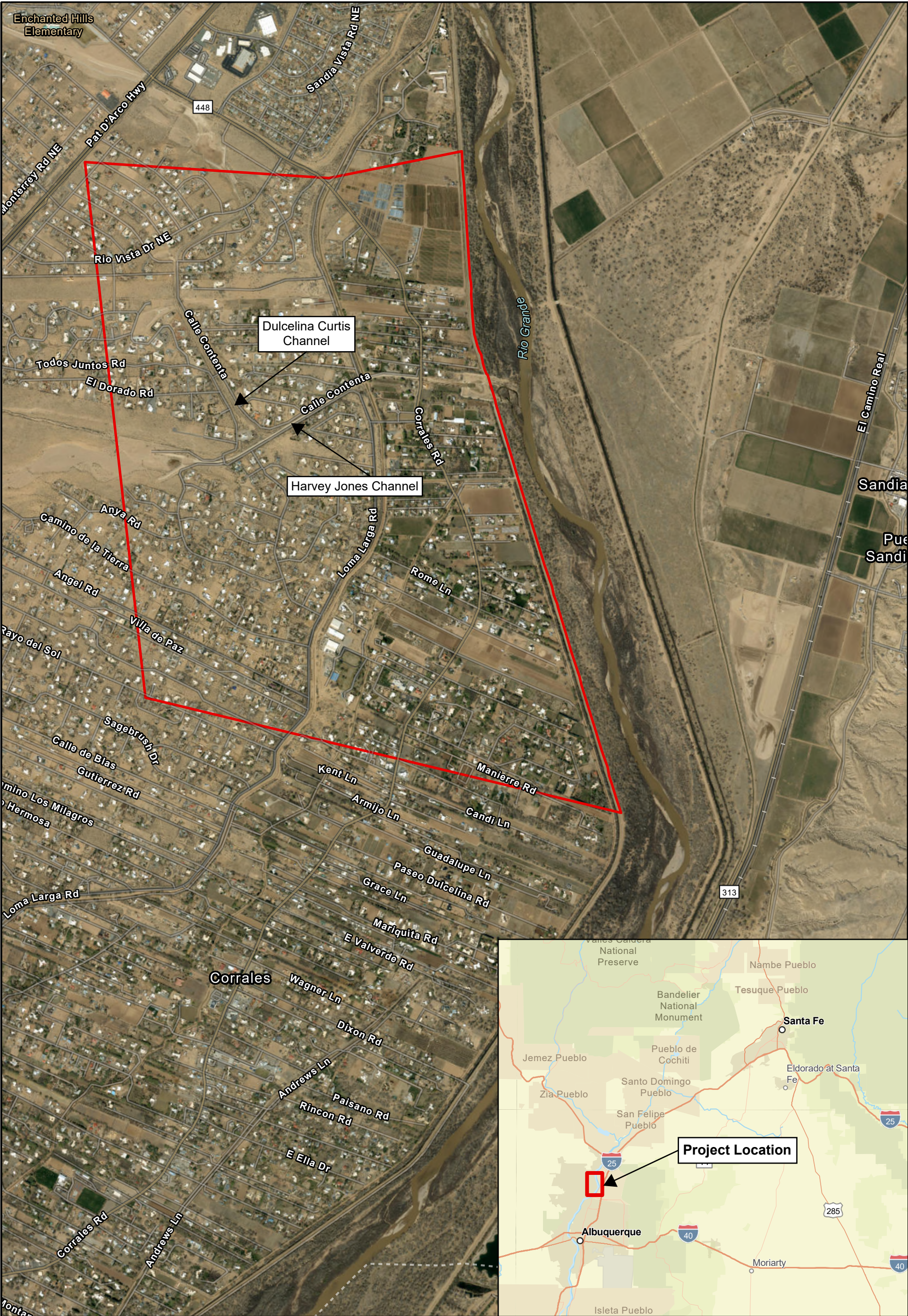
Although multiple projects have examined this reach and implemented measures to increase the bridge's capacity, neighborhood concerns have limited the extent to which the structure can be raised, thereby restricting the amount of conveyance achievable. Consequently, the bridge continues to act as a hydraulic bottleneck.

This capacity limitation results in overtopping and flooding of adjacent properties during larger storm events. To more accurately assess the risks associated with this undersized crossing and to delineate the extent of resulting inundation, a two-dimensional (2D) hydraulic model is required for the area. **Figure 1** illustrates the project location and the hydraulic model domain.

2 DATA GATHERING

Various sources of data and literature were reviewed prior to the start of analysis or field reconnaissance. All relevant pages of previous reports and reference texts are provided in **Appendix A**. The following background information was obtained and reviewed for this project:

1. *Montoyas Watershed Park Management Plan* (SSCAFCA, 2021) – The Watershed Park Management Plan (Montoyas WMP) includes an HEC-HMS hydrologic model developed for the Montoyas watershed. This model served as the basis for the inflow boundary conditions used in hydraulic modeling.
2. *Drainage Analysis Report – Harvey Jones Channel Improvements* (BHI, 2013) – This Drainage Analysis Report provided valuable project context and support for the need to increase the Corrales Bridge Crossing.
3. FIRM Panels 35043C1913D and 35043C1914D (FEMA, 2008) – These Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) panels were reviewed to understand existing mapped flooding extents and to assist in establishing the hydraulic model boundary.
4. FIS Report 35043CV000A (FEMA, 2008) – The FIS report was reviewed to provide additional context on existing conditions, historical flooding, boundary considerations, and an outflow boundary condition stage for the Rio Grande within the project reach.



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 Hydraulic Model Domain



0 800 1,600 3,200 Feet
1 in. = 1,600 ft.

Montoyas Arroyo HJC - Hydraulic Analysis Report

Figure 1
Project Location

5. Base Level Engineering Vector and Raster GIS Data (FEMA, 2019) – Base Level Engineering datasets were used to establish downstream boundary conditions in the Harvey Jones Channel by setting known water-surface elevations (WSELs) equal to the Rio Grande base flood elevation (BFE).
6. New Mexico Building Footprints Dataset (New Mexico Broadband Program, 2024) – Building footprint data were incorporated into the hydraulic model to represent structures and produce conservative inundation limits. These footprints were also used to identify inundated buildings and to reference adjacent WSELs.
7. Middle Rio Grande Conservancy District (MRGCD) Inventory (BHI, 2024) – This inventory was reviewed to identify hydraulic structures within the model domain, including those that could influence flow direction or withstand the modeled events.
8. *Middle Rio Grande Flood Protection-Bernalillo to Belen NM – Interim Feasibility Report - Main Report-EIS* (USACE, 1979 Vol 3D) – This document provided historical context regarding previous studies in the area. The review confirmed that Cochiti Dam operations were incorporated into prior hydrologic analyses.
9. Base Level Engineering Hydrologic Engineering Center's River Analysis System (HEC-RAS) Models (FEMA, 2019) – These models were used to determine the initial extents of this HEC-RAS model and surface data acquisition.
10. LOMR 12-06-1884P-350094 (FEMA, 2013) – Reviewed for project context about previous work in the area.
11. LOMR 21-06-1075P-350146 (FEMA, 2023) – Reviewed for project context about previous work in the area.
12. LOMA 12-06-0079A-350146 (FEMA, 2012) – Reviewed for project context about previous work in the area.

3 FIELD RECONNAISSANCE

A site visit to the project area was conducted by SSCAFCA and BHI on July 14th and 15th, 2025. The purpose of the site visit was to review preliminary model results, observe field conditions, and perform a windshield survey of the buildings identified as inundated under existing conditions. The survey focused on determining whether structures contained basements or had finished floor elevations (FFE) above adjacent ground.

Based on the discussions and observations made during the site visit, several refinements were incorporated into the hydraulic model. These refinements included adding detail to hydraulic structures to better represent control points within the reach and more accurately capture observed structural dimensions. The site visits also served as critical sources of information for validating model behavior throughout its development. All field observations and notes are documented in **Appendix B**, which contains a map marked with observation locations as well as the photos and observations from each observation location.

4 HYDROLOGIC ANALYSIS

SSCAFCA provided the existing conditions *Montoyas WMP* (2021) HEC-HMS model (version 4.2.1) to BHI as part of the *Montoyas Arroyo Reach Stabilization Plan* (RSP), *Broadmoor Blvd to NM Hwy 528* (BHI, 2024), which represents existing conditions development and hydrologic parameters. The existing conditions *Montoyas WMP* (SSCAFCA, 2021) HEC-HMS model incorporates sediment bulking factors in each subbasin. This model included the 100- and 500-year storm events. BHI added the 2-, 5-, 10-, 25-, and 50-year storm events to the HEC-HMS model using NOAA Atlas 14 data, collected in February 2024 for the RSP, using the same reference location as the *Montoyas WMP* (SSCAFCA, 2021). Cumulative precipitation depths for each storm event are listed in **Table 1**. The complete NOAA Atlas 14 data is provided in **Appendix C**. Additional details about SSCAFCA’s model development and the arroyo’s contributing areas are available in the *Montoyas WMP* (SSCAFCA, 2021). The existing conditions HEC-HMS results were extracted and used as input to the hydraulic analysis discussed in Section 5. Inflow hydrographs extracted from the hydrologic analysis are provided in **Appendix D**. The full model has been provided as a digital appendix (**Appendix F**).

Table 1. Cumulative Precipitation Depths

Storm Event	Cumulative Precipitation Depth (in)
2-year, 24-hr	1.34
5-year, 24-hr	1.68
10-year, 24-hr	1.95
25-year, 24-hr	2.32
50-year, 24-hr	2.6
100-year, 24-hr	2.9
500-year, 24-hr	3.61

5 HYDRAULIC ANALYSIS

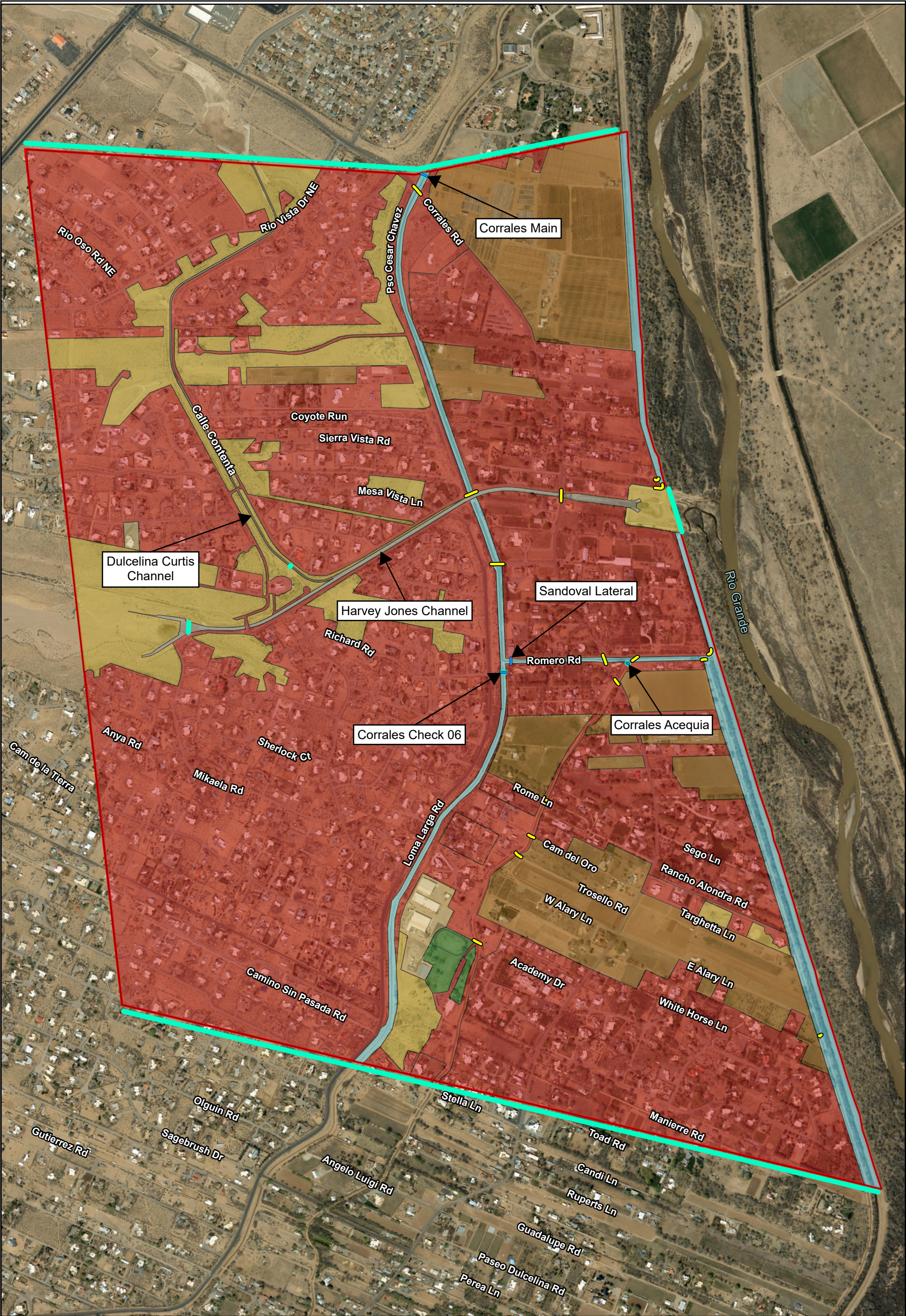
The existing hydraulic conditions were evaluated in the USACE HEC-RAS modeling software, version 6.6. This software was used to create a 2D model of the project area, simulating the 2-, 10-, 25-, 50-, 100-, and 500-year storm events. The model was used to identify flooding problem areas by delineating inundated areas and buildings, and to quantify the capacity of the existing Corrales Road bridge based on freeboard criteria as defined by the *New Mexico Department of Transportation Drainage Design Manual (NMDOT DDM)* (Smith Engineering Company and Occam Engineers Inc. [OEI], 2018). The full model has been provided as a digital appendix (**Appendix G**).

5.1 MODEL INPUTS

Input to the model includes definition of (1) the grid elements that coalesce to form the computational mesh, (2) topographic information from which the computational mesh extracts elevations, (3) essential hydraulic structures by entering documented or observed characteristics, (4) upstream and downstream boundary conditions, (5) hydraulic roughness zones, and (6) the computational parameters for each simulation. Hydraulic Structures, boundary conditions, MRGCD gauge locations, the model domain, and Manning’s “n”-values that were input to the model are illustrated in **Figure 2**.

5.1.1 COMPUTATIONAL MESH

The initial grid point spacing was set to 50-feet x 50-feet. Breaklines, refinement regions, and structure centerlines were enforced with a near spacing from 5- to 25-feet to better capture terrain high- and low-points, increase resolution in areas of hydraulic complexity, align cells to be perpendicular to flow, and/or ensure flow continuity.



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- Hydraulic Structure
- Boundary Condition
- MRGCD Gauge
- Model Domain

Manning's "n"-value	
Residential; Vegetated Residential	n = 0.045
Concrete	n = 0.013
Farmland	n = 0.04
Undeveloped Sand	n = 0.032
Park	n = 0.05
Irrigation	n = 0.03
Commercial	n = 0.02



0 500 1,000 2,000 Feet
1 in. = 1,000 ft.

**Montoyas Arroyo
HJC - Hydraulic
Analysis Report**
*Figure 2
Hydraulic Model
Input*

5.1.2 TOPOGRAPHIC DATA

Topography for the existing conditions hydraulic analysis was developed from Mid-Region Council of Governments (MRCOG) light detection and ranging (LiDAR) mapping collected in 2018 (BHI, 2022). The 2018 MRCOG LiDAR mapping is understood to have errors of up to 1.7 feet. Considering the project reach covers approximately 2 square miles, the relative error along the study area could be significant and have a substantial impact on the results of the analysis and associated inundation mapping. As such, a field survey was conducted in February 2025 to collect check points for comparison with the LiDAR elevations. The elevational corrections obtained from the BHI field survey were used to adjust the MRCOG 2018 LiDAR mapping and create a corrected Digital Elevation Model (DEM) from which the 2D hydraulic model's computational mesh was horizontally defined. The terrain has a 1-foot cell size and is in the North American Datum of 1983 (NAD 83), New Mexico Central horizontal coordinate system, and the North American Vertical Datum of 1988 (NAVD 88).

5.1.2.1 Terrain Modifications

Various terrain modifications were implemented to better reflect the existing terrain and hydraulic structures. Building footprints were downloaded in January 2025 from New Mexico Resource Geographic Information System (NM RGIS). The New Mexico Broadband Program published this building footprint layer in September of 2024 as publicly available digital, vector data. These building footprint polygons were added as terrain modifications in the 2D model, protruding from the existing terrain data. Within the building footprints, the terrain was raised 10 feet relative to the existing DEM elevations.

Hydraulic structure characteristics, including channel walls and crossing inverts, were input based on field survey data (Section 5.1.3), which often differed from the existing DEM. To reconcile these differences, the DEM was adjusted where necessary to align with surveyed structure inverts. Transitions were constructed to smoothly connect the DEM with the surveyed crossings, reducing potential for instabilities at structure faces.

5.1.3 STRUCTURE DATA

Culvert and bridge structures in the 2D hydraulic modeling domain that were assumed to withstand storms in the range of simulated events and affect hydraulic conditions were modeled based on structural attributes obtained from detailed field surveys conducted by BHI in February 2025. Survey points were collected to create structure centerlines and crossing inverts were entered

per the field survey. The inlet configuration into each crossing was determined from available survey points and/or aerial imagery. Structures were assigned hydraulic roughness according to the survey-reported material and *Sediment and Erosion Design Guide [Design Guide]* (SSCAFCA, 2008) as listed in **Table 2**.

Table 2. Manning’s “n”-values for Structures

Material	Manning’s “n”	Source
Reinforced Concrete Pipe	0.013	Design Guide, Table 3.1
Reinforced Concrete Box	0.015	Design Guide, Table 3.1
Corrugated Steel	0.025	Design Guide, Table 3.1
Corrugated Steel Inside Concrete	0.025	Design Guide, Table 3.1

5.1.4 BOUNDARY CONDITIONS

Flowrates from the HEC-HMS model were used as the inflows into the model in Harvey Jones Channel and the Dulcelina Curtis Channel, while MRGCD data was used to simulate baseflow in less significant irrigation canals.

5.1.4.1 Channel Inflows

Two inflow boundary conditions were placed in positions consistent with junctions named A_111_J4 and U_901_J from the HEC-HMS model in the main reach of HJC and the Dulcelina Curtis Channel, respectively. The inflow boundary conditions were governed by a flow hydrograph extracted from the HEC-HMS model results for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year storms, at their respective model junctions. The full download of all MRGCD gauge data at each of the locations and their determined reasonable maximum to serve as a constant irrigation inflow are provided in **Appendix E**.

Table 3 summarizes peak flowrates in each storm event. In addition to the two main channel inflows, baseflows were added to the model per MRGCD irrigation ditch stream gauges. These baseflow conditions are discussed in Section 5.1.4.2.

Table 3. Boundary Condition Summary

Storm	Peak Channel Inflow (cfs)		Constant MRGCD Gauge Inflow (cfs)			Peak Outflow (cfs)		
	Harvey Jones Channel	Dulcelina Curtis Channel	Corrales Main Canal	Corrales Check 06	Corrales Acequia	Harvey Jones Channel	Northern	Southern
2-year	278	111	38	93	24	326	0	101
5-year	662	267	38	93	24	790	0	101
10-year	1,420	438	38	93	24	1,585	0	101
25-year	2,899	739	38	93	24	3,201	0	101
50-year	3,964	951	38	93	24	4,361	0	101
100-year	6,429	1,174	38	93	24	6,131	0	143
500-year	14,201	1,731	38	93	24	6,130	2,284	1,152

5.1.4.2 Irrigation Inflows

Numerous irrigation ditches are present in the model domain. During irrigation months, the ditches are typically at or near capacity. Baseflows were added to the irrigation ditches using flow data from MRGCD gauges.

Within the model domain, MRGCD operates four (4) gauges that record irrigation ditch flow rates and stages over time. Time series data for each of these gauges was downloaded from the MRGCD Data Viewer on July 23, 2025. The data was processed, and the maximum observed flowrate (excluding outliers) was recorded for each gauge. All modeled irrigation channel flowrates are listed in **Table 3** and all additional gauge data evaluated for the model is provided in **Appendix E**. This maximum flow rate was modeled as a constant inflow hydrograph at each MRGCD gauge location in the model, apart from Sandoval Lateral.

For the Sandoval Lateral gauge, sensitivity analyses were conducted on the surrounding geometry. The most stable and reasonable results were achieved by routing flow from the Corrales Main Canal to Sandoval Lateral via a terrain modification, rather than explicitly modeling an outflow from Corrales Main Canal followed by an inflow into Sandoval Lateral a few cells downstream. Because the gauge data captures minor gains and losses (seepage, groundwater returns, evaporation, etc.) that are not directly accounted for in the model input/domain, implementing those internal boundary conditions resulted in significant increases to the model's volume accounting error. Because implementation of these boundary conditions does not conserve modeled flow continuity nor meaningfully affect the simulation results, the version of the model with the boundary conditions were used to prepare the inundation mapping. The Corrales Main flowrate is only approximately 5 cfs higher than the Sandoval Lateral average so it is considered to be within a reasonable discharge range.

5.1.4.3 Model Outflows

Three (3) outflow boundary conditions convey water out of the model. The primary outflow boundary condition at the mouth of HJC delivers all contained inflows to the Rio Grande. Adjacent to the Rio Grande, there is a levee with top elevation greater than the BFE, thus it is assumed that the only location water may rise high enough to exit the model was at the HJC outlet to the Rio Grande. This outflow boundary condition was governed by a stage hydrograph set to a constant elevation of 5,032.50 feet. This elevation conservatively estimates the 100-year stage expected at the Rio Grande outflow, based on BFEs at cross sections upstream and downstream of the HJC

outlet, per the Flood Insurance Study (FEMA, 2008) and GIS data downloaded in April 2025 (FEMA, 2023). Sensitivity analyses were conducted on this boundary condition, and it was concluded that the boundary condition is far enough downstream and the approximate BFE is low enough compared to the upstream channel sections that it does not adversely impact the model results in storm events that are not the 100-year. The second and third outflow boundary conditions run along the northern and southern edges of the model domain. These boundary conditions are governed by normal depth approximation with a nominal energy grade slope of 1%. Since the boundary conditions span the entire northern and southern edges of the model domain, this slope was not measured but the effect of the boundary condition slope was tested. Sensitivity analyses were conducted on the outflow boundary conditions, and it was concluded that the energy grade slope did not have a significant effect on the overall inundation limits. **Table 3** summarizes peak flowrates in each storm event.

5.1.5 HYDRAULIC ROUGHNESS

Hydraulic roughness was accounted for using Manning’s “n” roughness coefficients that were determined using publicly available aerial photography collected in the fall of 2020, field observations in July 2025, experience with similar channel/overbank conditions, and published values (Arcement and Schneider, 1989; NMDOT, 2018; SSCAFCA, 2008; USACE [HEC-RAS online documentation](#)). **Table 4** lists each roughness category classified within the 2D model domain, the associated “n”-value, source, and roughness name from the respective source.

Sensitivity analyses were conducted on the Manning’s “n”-value for residential and vegetated residential roughness categories. Tested values ranged from 0.025 to 0.06. While inundation limits changed depending on the Manning’s “n”-value in residential areas, 0.045 was deemed a suitable value, resulting in reasonable inundation limits, given the material and form roughness observed within the model domain.

Table 4. 2D Model Manning’s “n”-values

Category	Manning’s “n”	Source	Name
Residential	0.045	Arcement and Schneider (1989)	*Composite Value
Concrete	0.013	NMDOT Table 502-1	“Concrete (trowel finish)”
Farmland	0.04	HEC-RAS Manual Table 3-1	“Floodplains – Mature Field Crops”
Undeveloped Sand	0.032	NMDOT Table 502-1	“Earth with vegetation”
Vegetated Residential	0.045	HEC-RAS Manual Table 3-1	Low value for “Light brush and trees, in summer” to account for streets
Park	0.05	HEC-RAS Manual Table 3-1	“Light brush and trees, in summer”
Irrigation	0.03	NMDOT Table 502-1	“Earth (with vegetation)”
Commercial	0.02	HEC-RAS Manual Table 3-1	Between “Floodplains – Short grass” and “Asphalt”

* SSCAFCA Design Guide does not include guidance for overbank n-values (only includes guidance regarding the selection of main channel n-values) but does include reference to Arcement and Schneider (1989). Based on Table 3 of that document:

n1 (irregularity) = 0.008 for moderate irregularity

n2 = 0.0 (not applicable for variation of flood-plain cross section)

n3 = 0.012 for minor obstructions with < 15% of cross-sectional area

n4 = 0.025 for moderately dense vegetation with shallow flow depths

Composite n = 0.008+0.0+0.012+0.025 = 0.045, consistent with best estimate based on engineering judgement and previous experience in the SSCAFCA jurisdictional area

5.1.6 SIMULATION PARAMETERS

The models were executed for all storm events using the Diffusion Wave (DW) equation set with a varying computation interval of 0.09 – 0.4 seconds to satisfy a maximum Courant Number of 1.5 in Dulcelina Curtis Channel and the channel flow and breakouts of HJC. The Courant Number of irrigation channels was not regulated. The simulation window was set to a 12-hour duration to capture the peak of the HEC-HMS output hydrographs through the model. The hydrograph output interval was set to 5 minutes, while the mapping and detailed output interval was set to 10 minutes. Sensitivity analyses were conducted on simulation equation set as well as timestep to maximize the timestep, decreasing time required to run the simulation, while maintaining computational detail and accuracy. **Table 5** summarizes the range of inflow discharges at the HJC inflow and how those relate to the ultimate timestep.

Table 5. Range of HJC Inflows per Timestep

Timestep (sec)	Minimum Q (cfs)	Maximum Q (cfs)
0.09	6,118	6,121
0.10	4,872	6,131
0.12	1,898	6,103
0.15	586	4,361
0.20	276	4,182
0.29	147	899
0.40	0	653

5.2 MODEL VALIDATION

Proper flow routing and model resolution were validated by comparing the HEC-HMS junction hydrographs “HJC_gauge,” “HJC_J,” and “GaugingStation” within the HJC reach to the corresponding hydrographs generated in the 2D hydraulic model. For the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year storm events, the HEC-RAS hydrographs matched the HMS hydrographs nearly identically at all timesteps, confirming that model resolution is sufficient and that no significant volume loss occurred upstream of the gauge location during these events.

For the 500-year event, the rising and falling limbs matched closely, but the peak discharge differed. This difference is attributed to flow spilling over the channel walls in the hydraulic model before the location of the HEC-HMS junction. While the 100-year event spilled out of HJC, that overtopping did not occur upstream of the gauge location, so the hydrographs match. This hydraulic behavior cannot be represented in a HEC-HMS model. The presence of this spilled flow underscores the accuracy of the model and the importance of the 2D modeling and inundation mapping, as this out-of-bank flow represents the portion of runoff that threatens adjacent properties. Compared hydrographs are shown in **Figure 3** through **Figure 5**.

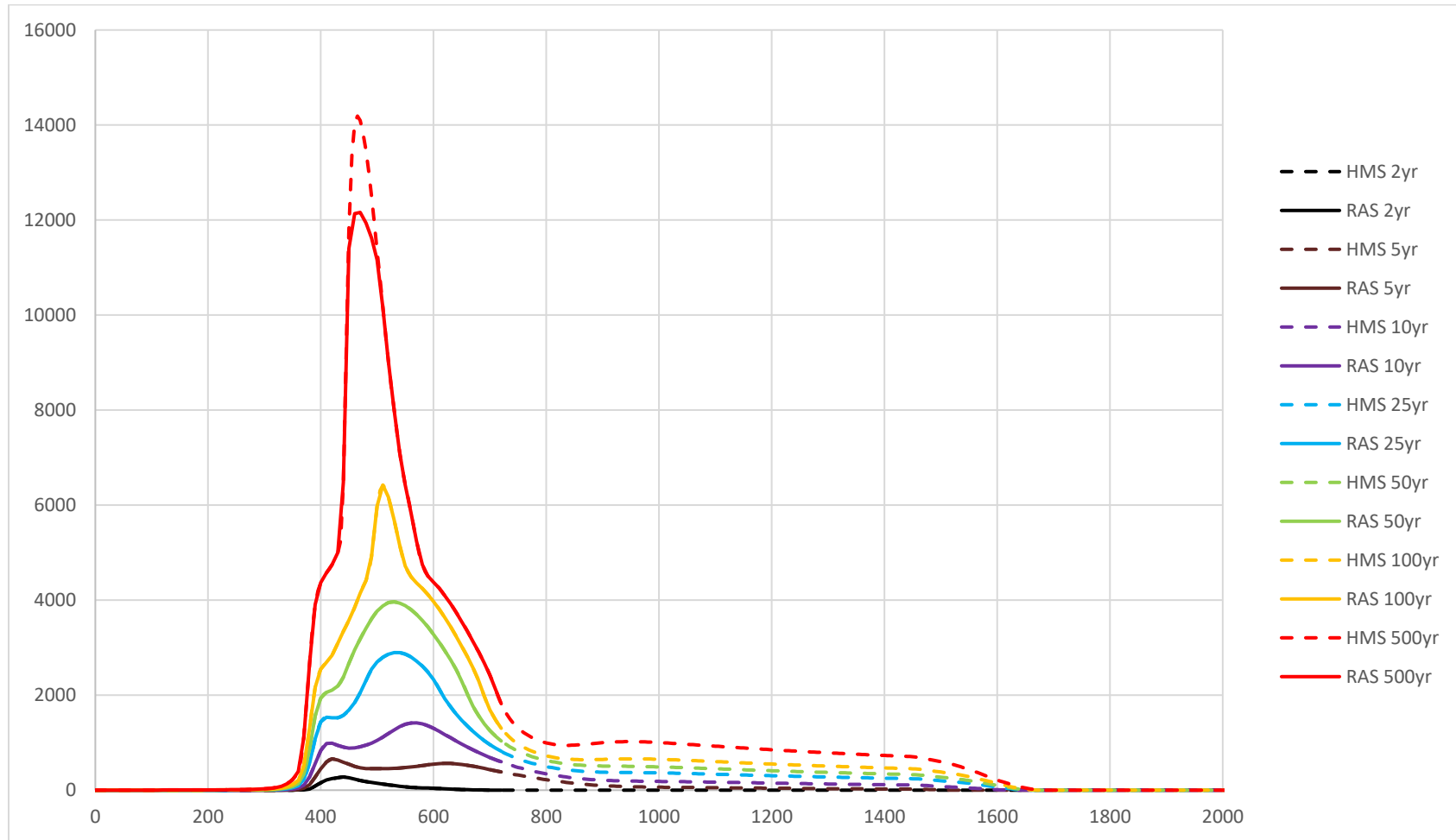
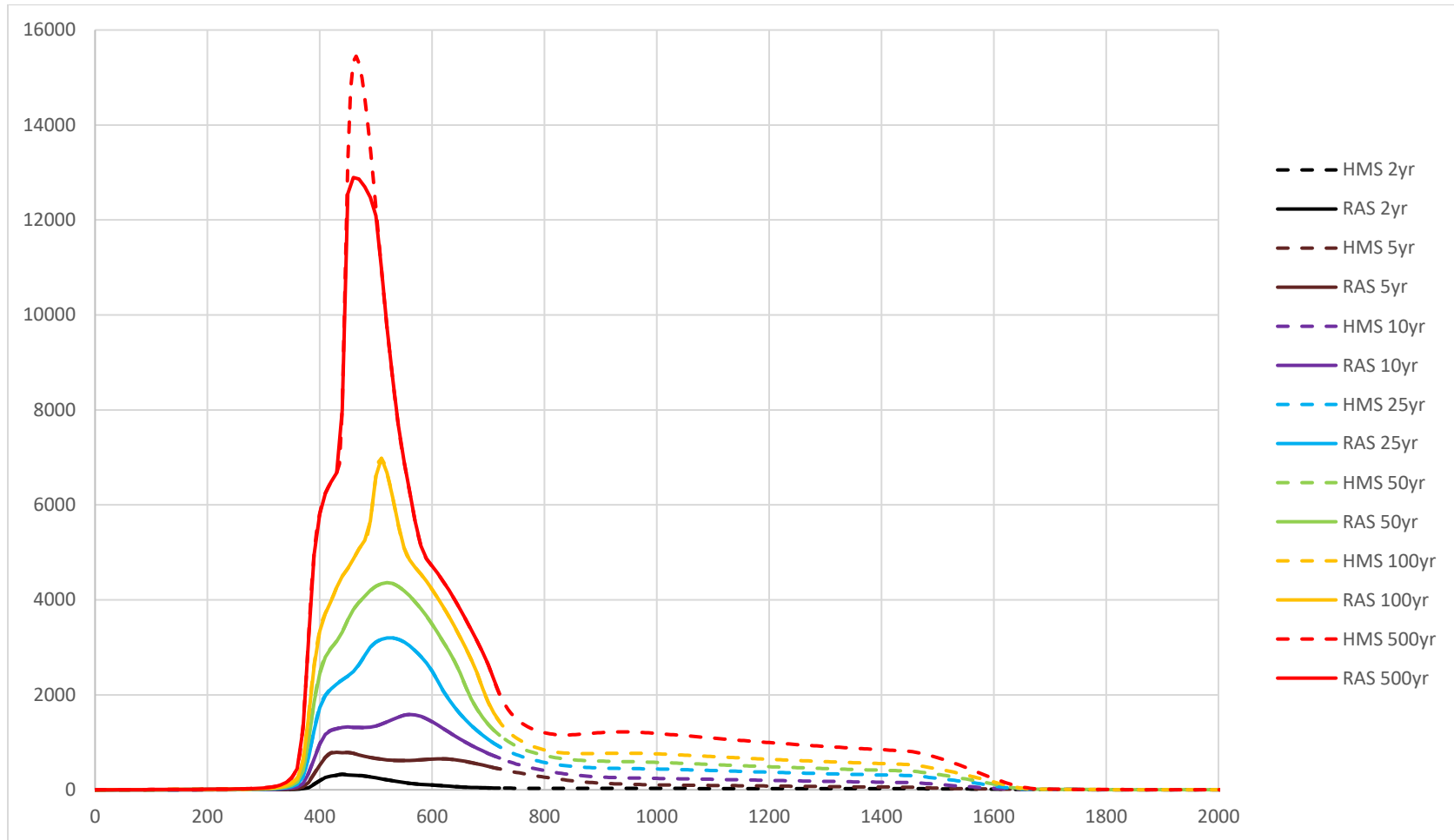


Figure 3. Model Validation Hydrograph at "HJC_gauge"



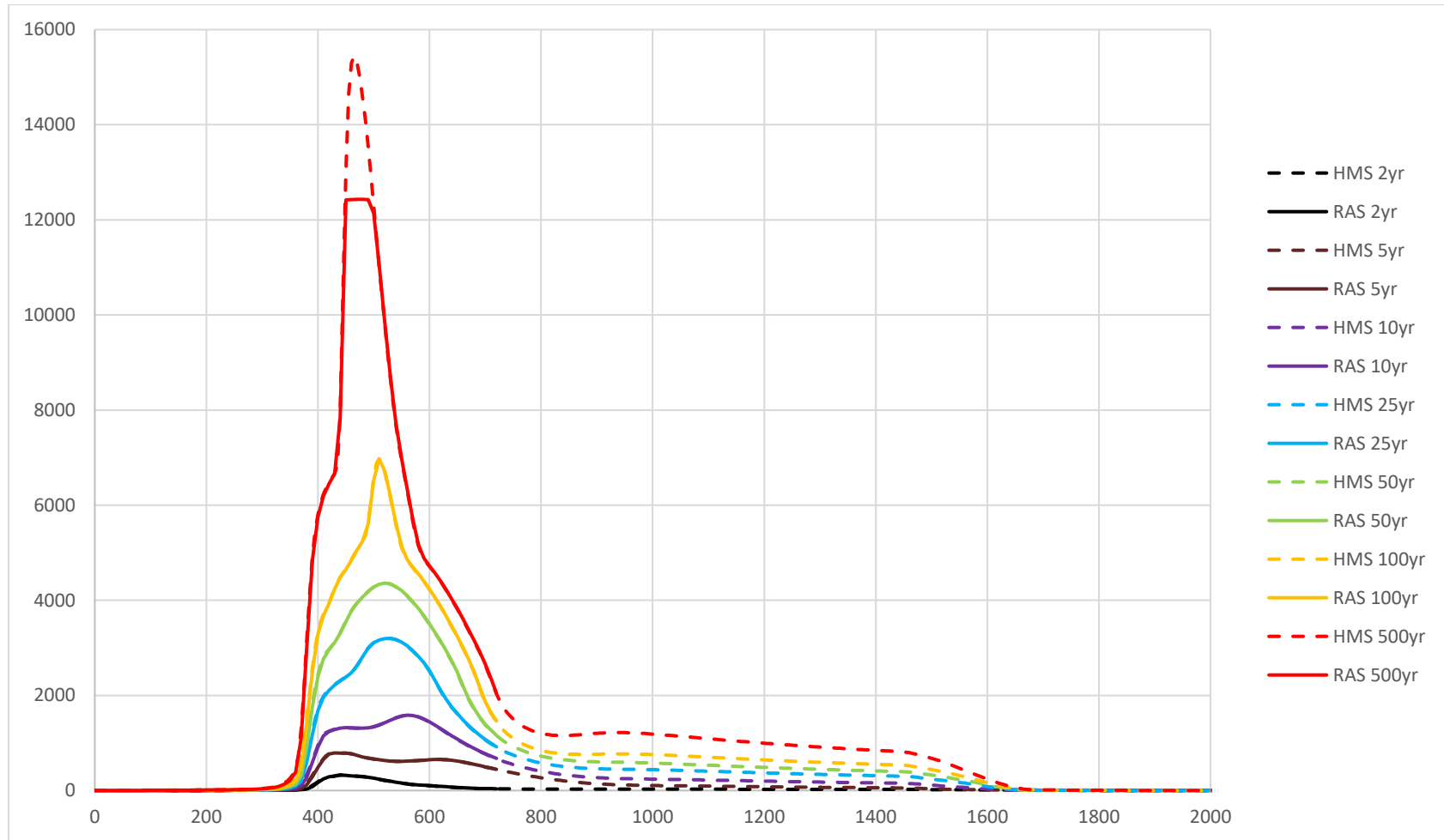


Figure 5. Model Validation Hydrograph at "GaugingStation"

5.3 MODEL RESULTS

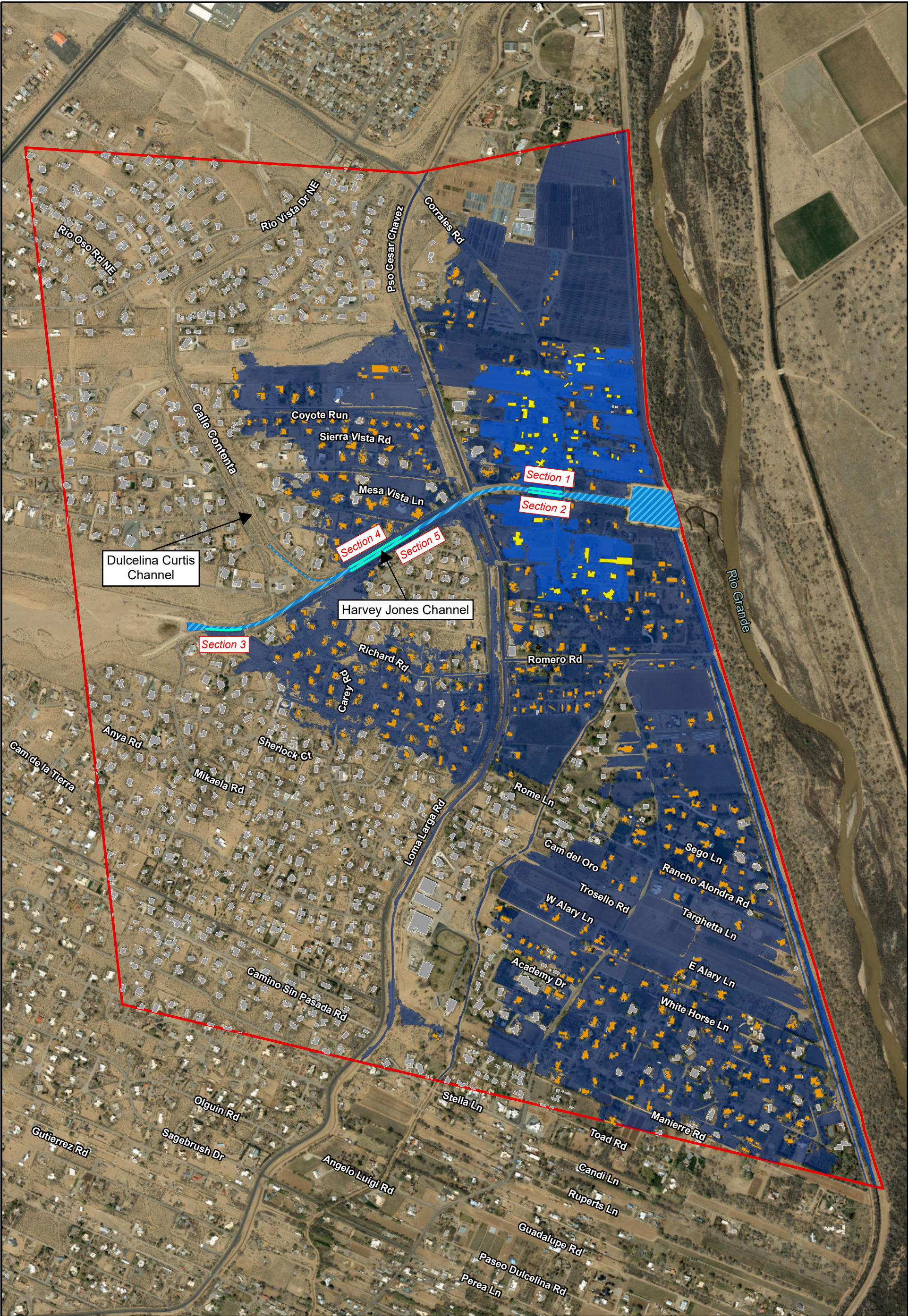
The 2D HEC-RAS model was used to evaluate hydraulic behavior throughout the HJC and Dulcelina Curtis Channel for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year storm events. The model results were used to (1) quantify infrastructure deficiencies at the Corrales Road bridge, (2) produce inundation limits for the 10-, 50-, 100-, and 500-year events, and (3) identify structures at risk for the 10-, 50-, 100-, and 500-year events.

5.3.1 HJC OVERTOPPING

The hydraulic model indicates that there is overtopping of HJC in the 100- and 500-year events. The 100-year storm has two (2) overtopping wall sections, while the 500-year storm has five (5) overtopping wall sections. **Figure 6** shows and labels all distinct overtopping wall sections. In both the 100- and 500-year storms, the channel walls upstream of Corrales Road bridge are overtopping (Section 1 overtops to the north and Section 2 overtops to the south). During both the 100- and 500-year storm events, the Corrales Road crossing capacity limitations create a backwater condition, causing this overtopping upstream of the crossing and results in extensive flooding of adjacent properties. The Corrales Road bridge capacity is discussed in greater detail in Section 5.3.1.1.

In addition to the HJC overtopping upstream of the Corrales Road bridge, the hydraulic model indicates three (3) other breakout locations during the 500-year event. These wall sections are overtopped because the HJC was only designed for the 100-year flood; thus, it is undersized in the 500-year event. The breakout across Section 3 occurs near the HJC inlet, where the southern wall is lower than the northern wall, so water only escapes to the south. The breakouts across Section 4 to the north and Section 5 to the south, occur approximately 250 feet downstream of the HJC confluence with the Dulcelina Channel.

All breakout properties by section (see **Figure 6**) are listed in **Table 6**.





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- Model Domain
- Overtopping Wall Section
- 100-yr Inundated Building
- 500-yr Inundated Building
- Building Footprint

- Inundation Boundary
 - 10-yr
 - 50-yr
 - 100-yr
 - 500-yr



0 500 1,000 2,000 Feet

1 in. = 1,000 ft.

**Montoyas Arroyo
HJC - Hydraulic
Analysis Report**

**Figure 6
Inundation Mapping**

Table 6. HJC Breakout Properties

Overtopping Wall Section	Overtopping Trigger		100-year Overtopping				500-year Overtopping			
	Flowrate (cfs)	WSEL (ft)	Peak Flowrate in HJC (cfs)	Peak Flowrate over HJC Wall (cfs)	Peak WSEL over HJC Wall (ft)	Volume over HJC Wall (ac-ft)	Peak Flowrate in HJC (cfs)	Peak Flowrate over HJC Wall (cfs)	Peak WSEL over HJC Wall (ft)	Volume over HJC Wall (ac-ft)
Section 1	6,129	5,037.2	6,977	536	5,037.8	16	12,431	4,074	5,038.7	477
Section 2	6,116	5,037.1	6,977	317	5,037.9	9	12,431	2,240	5,038.7	265
Section 3	10,566	5,069.5	6,983	N/A			13,436	1,932	5,070.8	93
Section 4	12,747	5,054.9	6,428	N/A			14,084	604	5,055.1	25
Section 5	12,747	5,054.9	6,428	N/A			14,084	404	5,055.1	16

5.3.1.1 Corrales Road Bridge

The Corrales Road bridge is known to have a lower hydraulic capacity than HJC upstream and downstream of the crossing as discussed in Section 1. This analysis confirms that the Corrales Road bridge is significantly undersized relative to the channel capacity and *NMDOT DDM* (Smith Engineering Company and OEI, 2018) freeboard requirements. This constriction causes backwater effects during the 100-year and 500-year storm events, resulting in overtopping and spillover to the north and south of HJC upstream of the Corrales Road bridge before the Corrales Road bridge is overtopped. Although the most recent design modification increased the crossing capacity, it still does not meet the requirements specified in the *NMDOT DDM* (Smith Engineering Company and OEI, 2018).

The Corrales Road bridge was modeled in HEC-RAS as a storage area and 2D flow area (SA/2D) connection using survey data to define structure geometry. The existing crossing consists of two bridge openings, each approximately 22 feet wide and 8 feet tall, separated by a central pier.

Table 7 summarizes the discharge conveyed by the bridge for each storm event, the associated headwater elevation from the HEC-RAS model, and the resulting freeboard. The results for the 100- and 500-year events indicate the Corrales Road bridge has a capacity of 6,130 cfs. Per the *NMDOT DDM* (Smith Engineering Company and OEI, 2018), a minimum of 2 feet of freeboard below the low chord (5,032.7 feet) is required for both existing and new bridges. Negative freeboard values indicate the WSEL is above the bridge low chord.

Table 7. Corrales Road bridge Freeboard

Storm Event	Flowrate (cfs)	Headwater Elevation (ft)	Freeboard (ft)
2-yr	330	5,032.5	0.2
5-yr	790	5,032.5	0.2
10-yr	1,590	5,032.6	0.1
25-yr	3,200	5,032.9	-0.2
50-yr	4,360	5,033.5	-0.8
100-yr	6,130	5,037.3	-4.6
500-yr	6,130	5,037.3	-4.6

5.3.2 INUNDATION LIMITS

Inundation limits were generated for the 10-, 50-, 100-, and 500-year storm events, with the analysis focused on the HJC. Inundated areas from the model were excluded from the final inundation extents for two primary reasons: (1) irrigation canals were omitted unless water depths exceeded baseflow conditions, and (2) hydraulically disconnected areas were removed based on assessments of WSEL versus terrain profiles, velocity vectors, and hydrographs. The 10-year storm event, which does not result in overtopping the HJC, was used to define baseflow conditions in the irrigation canals. As a result, only those canal areas showing depths greater than the 10-year scenario were included in the final inundation limits. The final inundation limits are shown on **Figure 6** and provided as a digital appendix (**Appendix H**).

5.3.3 INUNDATED BUILDINGS

Inundated buildings were identified for both the 100-year and 500-year storm events. Although 10-year and 50-year storms were simulated, no out-of-bank flow from the HJC occurred during those events; therefore, no building inundation was recorded for those frequencies.

Buildings were considered inundated when the WSEL layer intersected the building footprint outlines obtained from the NM RGIS. For each building identified as touching the WSEL, both a WSEL and terrain elevation value were assigned for use in benefit-cost analyses (BCA) conducted by SSCAFCA.

- Water Surface Elevation (WSEL): The WSEL value was extracted from the HEC-RAS results and represents the maximum water surface elevation within one foot of the building footprint boundary.
- Terrain Elevation: The terrain elevation was determined as the minimum ground elevation within the building footprint, providing a conservative estimate of inundation depth (i.e., the largest possible depth difference between the WSEL and ground).

Additionally, potential BFE adjustments were applied based on field observations and Google Earth imagery. Where applicable:

- Each step leading up to a front door or a raised foundation was assumed to increase the FFE by 0.5 feet per step.
- Other elevation features such as ramps or porches were estimated based on aerial or street-view imagery.

The inundated buildings shapefile has been provided as a digital appendix (**Appendix H**).

6 CONCLUSION

This hydraulic analysis demonstrates that the existing HJC performs as intended under smaller storm events but becomes overtopped during larger storms due to capacity limitations at the Corrales Road (NM 448) bridge crossing. The 2D model, validated against HEC-HMS hydrology, provides a reliable representation of channel behavior, breakout locations, and floodplain inundation under existing conditions.

The Corrales Road bridge continues to act as a critical constriction within the system. Despite previous capacity improvements, the structure does not meet the *NMDOT DDM* (Smith Engineering Company and OEI, 2018) freeboard requirement and cannot safely convey flows associated with the 100-year or 500-year storm events. Backwater generated by the bridge causes premature overtopping of the HJC channel walls upstream, directing flow toward private property on both the north and south sides of the channel. This overflow, not on-bridge overtopping, is the primary driver of risk to surrounding properties.

Inundation mapping confirmed that no buildings are affected during the 10- and 50-year storm events; however, multiple structures become inundated during the 100- and 500-year events. Field reconnaissance allowed refinement of estimated FFEs, improving the reliability of the structure-specific WSEL and inundation depth data provided to SSCAFCA.

The findings of this study reinforce the need for continued evaluation of long-term mitigation options at or upstream of the Corrales Road bridge. The hydraulic model and accompanying datasets generated through this effort provide SSCAFCA with a detailed baseline for future planning, design, and FEMA regulatory updates.

APPENDICES

APPENDIX A – REFERENCE TEXT EXCERPTS

APPENDIX B – FIELD RECONNAISSANCE

APPENDIX C – NOAA DATA

APPENDIX D – HEC-HMS INFLOW HYDROGRAPHS

APPENDIX E – MRGCD IRRIGATION GAUGE DATA

APPENDIX F – HEC-HMS MODEL (DIGITAL)

APPENDIX F – HEC-RAS MODEL (DIGITAL)

APPENDIX H – INUNDATION SHAPEFILES (DIGITAL)

APPENDIX A – REFERENCE TEXT EXCERPTS

2. Watershed Hydrology

All hydrologic modeling was carried out using the U.S. Army Corps of Engineers HEC-HMS software version 4.2.1.

2.1. Basin Delineation

Orthophotography used for this project consists of tiled images which depict color digital aerial photographs acquired in the spring of 2018 during leaf-off conditions. LiDAR-derived elevation data (2-foot contour interval, 2010) was used to delineate watersheds and sub-basins as well as for calculating hydrologic parameters. Both orthophotography and elevation data are part of the Mid-Region Council of Governments (MRCOG) Digital Orthophotography and Elevation Data Project.

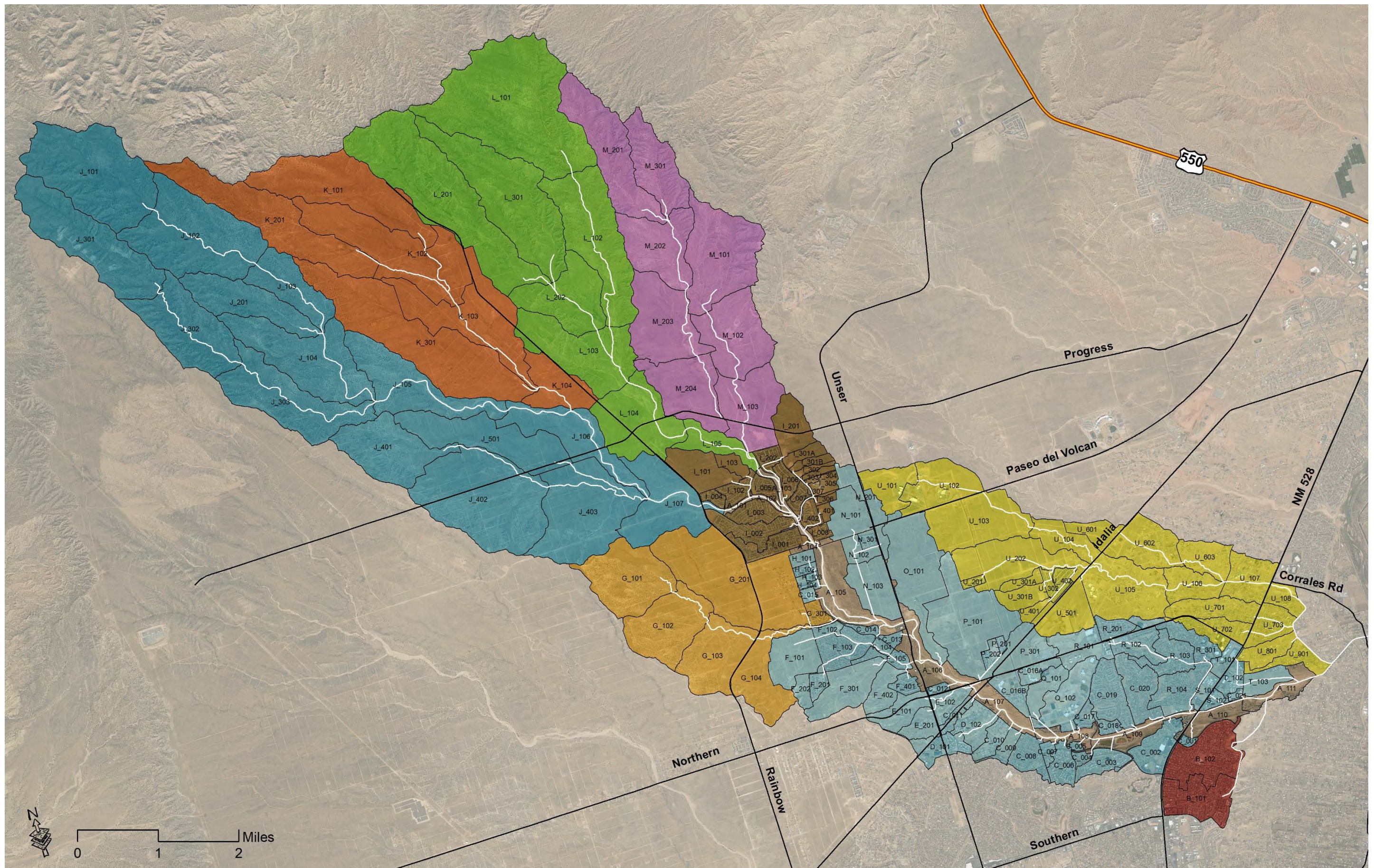
Initial watershed and subbasin boundary delineation was accomplished using HEC Geo-HMS software with a digital elevation model (DEM) created from 2010 MRCOG LiDAR data. Basins were modified to accommodate desired analysis points and achieve basins with relatively uniform land use characteristics. Analysis points were selected for tributary confluences, major existing culverts and road crossings, and existing and proposed pond locations. Questionable boundaries were verified in the field, especially at locations where graded roads influence flow paths, and where a dominant flow path was not immediately obvious from 2-ft contours. An overview map of basin boundaries can be seen in Figure 2.1; major tributaries are shaded in different colors.

2.2. Reach Routing

Routing reaches were delineated, and slopes estimated in Arc-GIS based on 2010, 2-ft contours. Channel reaches were modeled using idealized cross-sections that most closely resembled the natural geometry of the reach (trapezoidal and rectangular). Roughness coefficients (Manning's n-values) were estimated based on orthoimagery and field investigations. In general, the following n-values were used in the model (Table 2.1):

Table 2.1: Roughness coefficients for routing reaches.

Surface Type	Manning's n-value
Concrete pipe	0.013
Road (asphalt)	0.017
Corrugated metal pipe	0.025
Major arroyo, sandy bed and vertical banks	0.020
Natural channel, moderate to heavy vegetation in channel bed and along banks	0.025 - 0.035



2.3. Existing Land Use

The City of Rio Rancho, which covers a portion of the Montoyas watershed (see Figure 1.1), is one of the fastest growing cities in the southwestern U.S. The city was incorporated in 1981 with a population of approximately 10,000 residents. Figure 2.2 shows population growth since 1980. The growth rate slowed following the housing market crash of 2008 but is projected by the City of Rio Rancho to reach a population of approximately 106,000 by the year 2020. Other municipalities in the watershed include the Village of Corrales (see Figure 1.1) and unincorporated areas of Sandoval County.

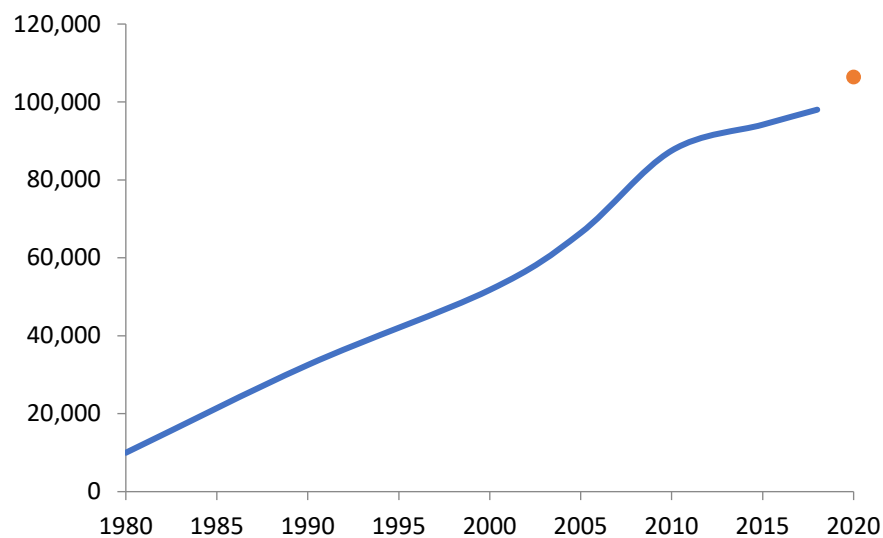


Figure 2.2: Population growth in the city of Rio Rancho (blue) and projected population for the year 2020 (orange). Data source: U.S. Census Bureau (<https://www.census.gov/data.html>) and City of Rio Rancho (<https://rrnm.gov/4185/Community-Overview>).

In 2019, a large portion of the Montoyas watershed was undeveloped. Figure 2.3 shows the extent of urbanization and major land use categories. Urbanization was focused in the lower and central portion of the watershed. Most of the urban development consisted of residential subdivisions (blue). Land use was quantified by manual digitization using orthoimagery and based on GIS data obtained from the City of Rio Rancho.

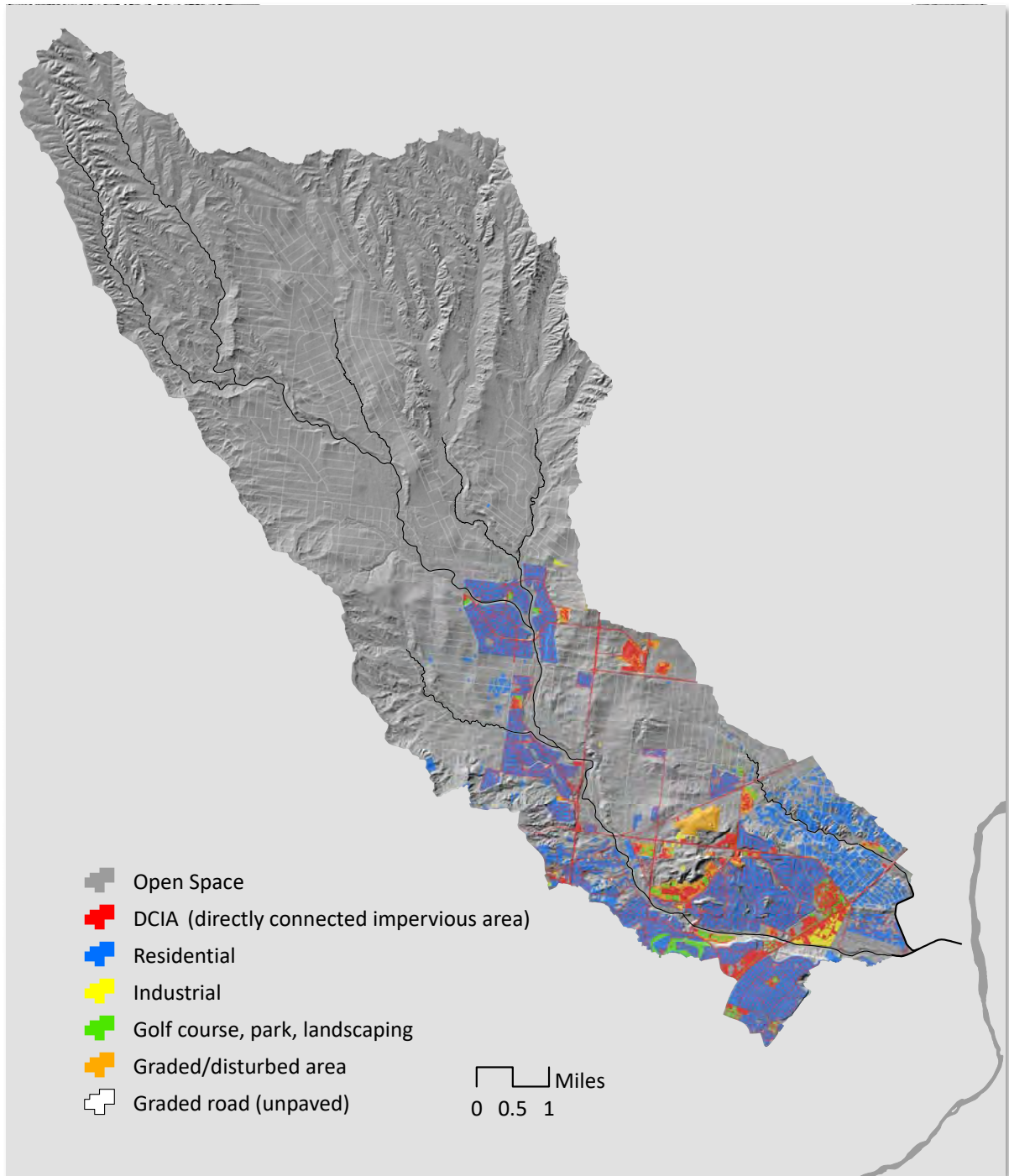


Figure 2.3: Overview map of the Montoyas watershed and major land use types in 2019.

2.4. Existing Conditions Loss Parameters

In accordance with SSCAFCA’s hydrology guidance (SSCAFCA, 2020), the curve number method was used to compute precipitation loss and excess. Curve numbers for pervious areas were estimated based on 2019 land use conditions in the Montoyas watershed (see Figure 2.3). Special emphasis was placed on impervious coverage: directly connected impervious areas (DCIA) were specified explicitly for each subbasin rather than including them in a composite loss calculation. Major sources of DCIA such as commercial areas and paved roads (Figure 2.3, red) were digitized manually. A hybrid approach was adopted for residential areas (Figure 2.3, blue). Residential driveways and five percent of roof areas (the portion of the roof draining onto the driveway) were assumed to be directly connected. Residential roof areas were quantified based on GIS coverage of building footprints maintained by the City of Rio Rancho; an average driveway size of 700 ft² was assumed for each residential lot. Disconnected impervious areas (e.g. 95% of roof areas draining onto pervious landscaping) were included in the composite loss calculation for the corresponding subbasin. Table 2.2 lists land use types and associated loss parameters.

Table 2.2: Land use categories and associate loss parameters for existing conditions 2019

Land Use Type	Data Source	% DCIA	Pervious CN	% of Total Area
Paved roads with curb, residential driveways, other DCIA	CoRR curb coverage, parcels, manual digitization	100	-	3.5%
Disconnected impervious areas	CoRR building footprints, manual digitization	5	98	1.8%
Road ROW	Digitization, buffer around paved roads	0	92	1.1%
Unpaved road	CoRR road centerline, buffer applied in GIS	0	82	3.0%
Industrial	Manual digitization	0	88	0.2%
Graded areas	Manual digitization	0	86	0.8%
Sports fields, city parks, landscaping	Manual digitization	0	74	0.5%
Residential yard	CoRR parcels, manual digitization	0	74	6.3%
Open space	GIS	0	74	82.7%

Table 2.2 illustrates that under existing conditions, a large portion of the Montoyas watershed is undeveloped, with the exception of a network of graded dirt roads. Open space and residential yards account for nearly 90% of the basin area.

2.5. Projected Future Land Use

Given growth projections for the area, anticipated future land use for the basin was considered as part of this planning study. Based on these land use assumptions, expected impacts on watershed processes such as runoff and erosion were evaluated. Figure 2.4 shows a map of anticipated future land use in the basin; land use projections were based on the following assumptions:

- Residential lots in master planned developments and largely developed areas in the lower watershed will develop as platted;
- Areas covered by specific area plans (SAP) will develop as indicated in the planning document, specifically: Broadmoor Drive SAP (CoRR, 2007a), Dos Amigos SAP (CoRR, 2007b), Sierra Vista SAP (CoRR, 2008), Del Norte SAP (CoRR, 2009), Paseo Gateway West SAP (CoRR, 2010a), La Barranca SAP (CoRR, 2010b), Northern Unser SAP (CoRR, 2011);
- The state land tract and lots adjacent to the future Paseo-del-Volcan road alignment will develop as commercial;
- Approximately 650 acres in the upper watershed predominantly under one ownership will develop as a master planned subdivision and a commercial area;
- 60% of remaining residential lots south of the power line easement (see Figure 2.4, dark purple) will develop as platted, 40% will remain undeveloped;
- 15% of remaining residential lots north of the power line easement (see Figure 2.4, light purple) will develop as platted, 85% will remain undeveloped;
- Unplatted areas in the upper portion of the watershed will remain open space.

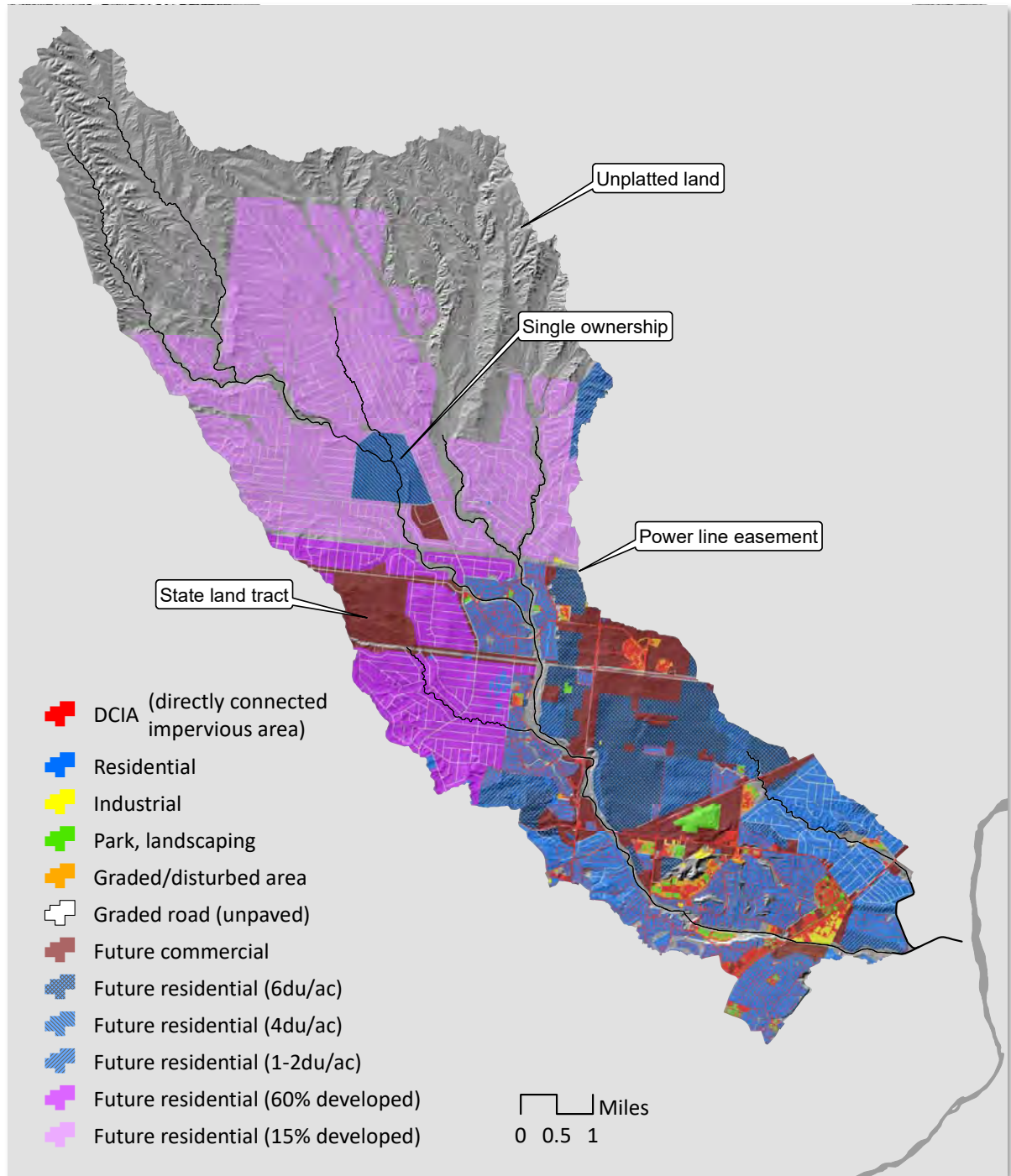


Figure 2.4: Anticipated future land use in the Montoyas watershed.

2.6. Developed Conditions Loss Parameters

The main purpose of the developed conditions model is to estimate runoff from the watershed under future urbanized conditions. The future conditions model helps to identify potential areas of flooding and plan for appropriate mitigation strategies such as land acquisition and drainage infrastructure.

Loss parameters for the developed conditions hydrologic model were estimated based on projected future land use (see Figure 2.4). Table 2.3 shows land use types and associated parameters along with a justification of how the parameters were estimated.

Table 2.3: Land use categories and associated loss parameters for anticipated future land use.

Land Use Type	Data Source	% DCIA	Pervious CN	Justification
Medium density residential (6du/ac)	Manual digitization	26	84	Non-DCIA is composed of: 32% disconnected impervious (CN=98), 14% road ROW (CN=92), 46% residential yard (CN=74), 8% open space (CN=74); statistics based in CoRR High Range subdivision
Medium density residential (4du/ac)	Manual digitization	21	81	Non-DCIA is composed of: 22% disconnected impervious (CN=98), 10% road ROW (CN=92), 43% residential yard (CN=74), 25% open space (CN=74); statistics based on CoRR Northern Meadows subdivision
Low Density Residential (2du/ac)	Manual digitization	3	77	Non-DCIA is composed of: 8% disconnected impervious (CN=98), 8% unpaved road (CN=82), 84% residential yard (CN=74); statistics based on CoRR Unit 17 typical development
Low Density Residential (1du/ac)	Manual digitization	2	76	Non-DCIA is composed of: 7% disconnected impervious (CN=98), 10% unpaved road (CN=82), 83% residential yard (CN=74); statistics based on Village of Corrales typical development
Residential 60% developed	Sandoval County parcels, manual digitization	1.8	76	Assumes 60% of this area is developed with an average density of 2du/ac; remaining area modeled as open space (CN=74)
Residential 15% developed	Sandoval County parcels, manual digitization	0.5	74	Assumes 15% of this area is developed with an average density of 2du/ac; remaining area modeled as open space (CN=74)
Future commercial	CoRR parcels, manual digitization	85	74	Assumes 85% DCIA, remainder is open space (CN=74)

2.7. Transform Method

In HEC-HMS, the Clark unit hydrograph method was selected to transform excess precipitation into a runoff hydrograph for each subbasin. This slight deviation from SSCAFCA's hydrology manual is justified because gridded precipitation data was used during model calibration; the ModClark method is the only transform currently available in HEC-HMS that can be used with gridded rainfall data. Times of concentration were estimated in Arc-GIS based on the watershed DEM using the methodology outlined in TR-55 (USDA, 1986). Subbasin storage coefficients were estimated as 50 percent of the time of concentration. Transform parameters were adjusted during model calibration. A list of model parameters for subbasins and routing reaches is contained in Appendix A.

2.8. Sediment Bulking

Sediment bulking factors of 18% for natural areas and 6% for urbanized areas were added as flow ratios to clearwater discharges in HEC-HMS to account for the increase in runoff volume due to suspended sediment in storm flows. Area averaged bulking factors were used for subbasins containing both urbanized and natural areas.

2.9. Existing Ponds

The Montoyas watershed model contains a total of 59 ponds and dams that attenuate runoff ranging from less than 1 ac-ft to more than 300 ac-ft in storage volume (see Appendix B). Pond parameters and dimensions were collected from corresponding engineering documents and verified in the field where necessary. A comprehensive list of all ponds included in the watershed model is contained in Appendix B. In HEC-HMS, pond routing was simulated using rating curves (elevation-storage and storage-discharge curves). Ponds were assumed to be dry at the start of each simulation.

2.10. Design Storm

The design storm is used as a planning tool. It is a hypothetical storm event based on point precipitation frequency estimates from the NOAA Atlas 14 (NOAA, 2019). Precipitation estimates for the Montoyas watershed and are displayed in Table 2.4.

Table 2.4: Point precipitation frequency estimates for the 100-year recurrence interval in the Montoyas watershed.

Duration	Point precipitation estimate (in)
5 minutes	0.589
15 minutes	1.110
1 hour	1.850
2 hours	2.130
3 hours	2.200
6 hours	2.390
12 hours	2.550
1 day	2.900

The design storm was modeled in HEC-HMS using the built-in frequency storm option with an intensity position of 25 percent (see Figure 2.5), intensity duration of five minutes, and the depth-area reduction for a watershed area of 61 square miles. Temporal and spatial patterns of real-world storm events will likely differ from the design storm and induce a different watershed response.

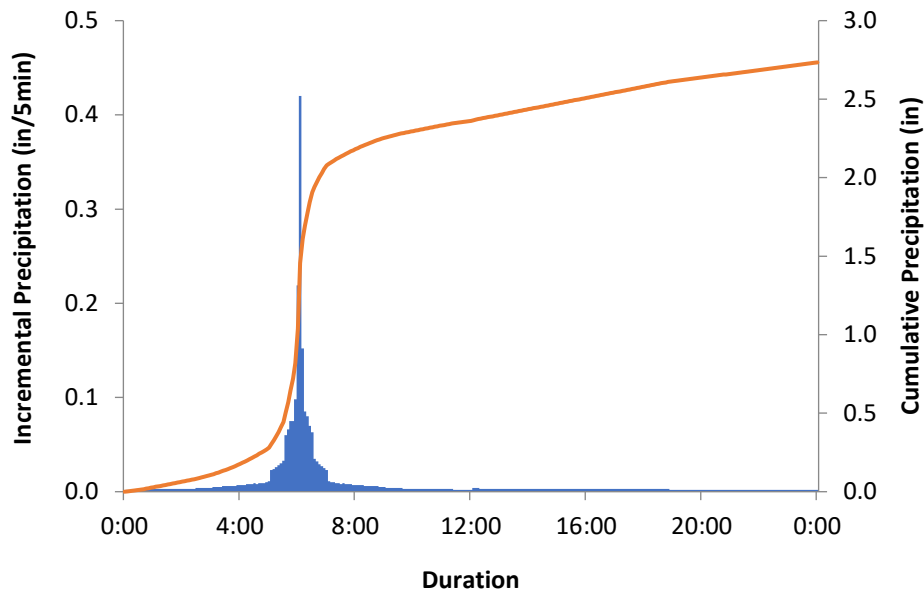


Figure 2.5: Design storm temporal distribution.

2.11. Existing and DEVEX Conditions Results

Figure 2.6 shows design storm hydrographs at the outlet of the watershed for existing land use conditions (left) and developed conditions with existing drainage infrastructure (DEVEX, right). Black lines in Figure 2.6 are results from model runs with a curve number of 74 assigned to all open space, landscaping, and residential yards. This represents the best estimate for a curve number representing intermediate moisture conditions. Grey areas represent model runs with a curve number range of 71-76 for open space, landscaping, and residential yards. This range was selected to provide an uncertainty envelope around the estimated 100-year runoff estimate associated with initial moisture conditions.

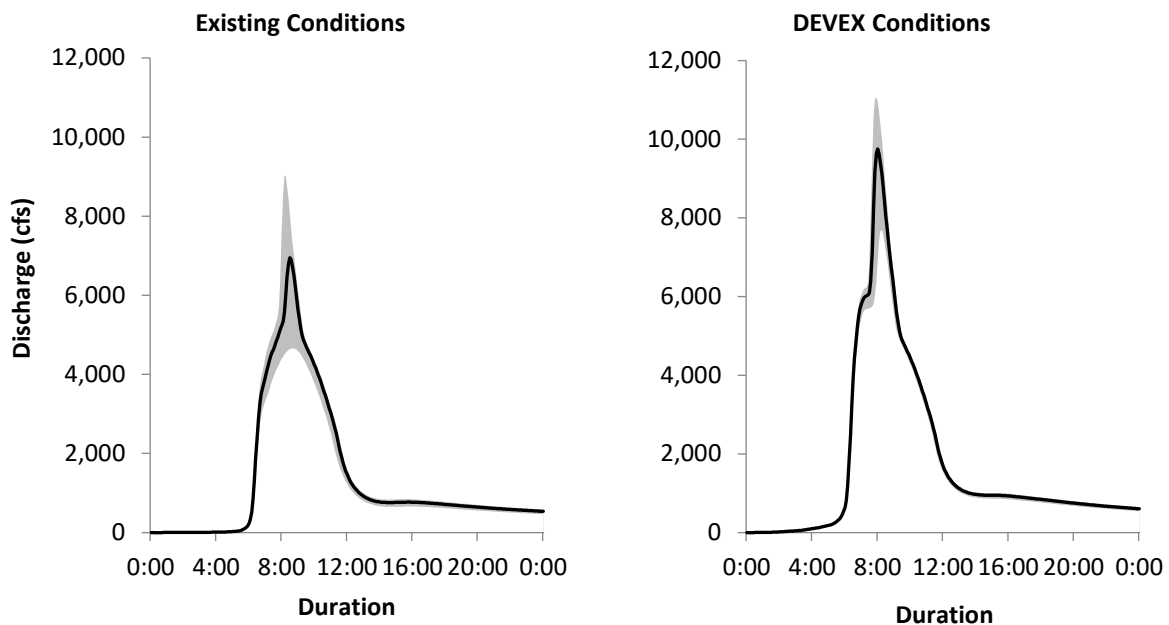


Figure 2.6: Simulated design storm discharge for existing conditions (left) and DEVEX conditions (right).

It is important to note that simulation results only provide a best estimate of the watershed runoff response from the design storm for current and projected future land use conditions. Model results are intended to be used for planning and design of flood control infrastructure but need to be interpreted with the underlying uncertainty in mind.

Based on this analysis, expected peak discharge at the watershed outlet is approximately 7,000 cfs (range: 4,700 – 9,000 cfs) for existing conditions and 9,800 cfs (range: 7,700 – 11,100 cfs) for developed conditions with existing drainage infrastructure. A detailed list of model results is contained in Appendix C.

2.12. Structure Capacities and Major Deficiencies

Crossing structure capacities for locations where 100-year peak flow rates exceed 500 cfs were analyzed based on Existing and DEVEX conditions model runs. Results are summarized in Table 2.5. Figure 2.7 shows crossing structure locations.

Table 2.5: Major crossing structures, capacities and peak discharges under existing and developed conditions with existing infrastructure (DEVEX).

Crossing	Location	HMS_ID	Drainage Area (mi ²)	Existing Q _p (cfs) ^a	DEVEX Q _p (cfs) ^a	Capacity (cfs)
1	Montoyas & King Blvd.	A_103_J4	38	6614	6987	4800
2	Tributary G & Cherry Rd.	G_104_J2	4	1234	2131	1100
3	Montoyas & Unser Blvd.	A_105_J4	44	7202	7992	5200
4	Montoyas & Northern Blvd.	A_106_J5	47	7388	8402	5800
5	Montoyas & Broadmoor Blvd.	A_107_J6	49	7484	8690	7300
6	Montoyas & NM 528	A_109_J5	53	6739	8877	7100
7	High Range Trib. & Loma Colorado Blvd.	U_202_J3	1	471	856	440
8	High Range Trib. & Idalia Rd	U_202_J3	1	471	856	510
9	Aloe Trib. & Idalia Rd.	U_104_J	2	505	1057	1100
10	Lomitas Negras & Saratoga Dr.	U_106_J2	5	1101	1537	1250
11	Lomitas Negras & NM 528	U_107_J	5	1164	1624	1400
12	Rockaway Channel & Turquoise Dr.	R_104_J1	1	673	818	430 ^b
13	Rockaway Channel & Rockaway Blvd.	R_103_R2	1	666	811	790
14	Rockaway Channel & Sundt Rd.	R_104_J	1	879	981	930
15	Harvey Jones Chan. & Corrales Rd.	HJC_J	61	6976	9762	5500
16	Montoyas & Serene Meadows	J_107_J	21	4225	4504	5200
17	Montoyas & Camino de los Montoyas	A_101_J	22	4207	4489	4800
18	Ponce de Leon & Camino de los Montoyas	L_105_J3	15	3128	3222	4100

^a Flow rates from model runs with depth-area reduction factors corresponding to the drainage area contributing to each analysis point

^b Estimated capacity if clean = 780 cfs

	adequate capacity
	< 10% exceedance
	10-25% exceedance
	25-50% exceedance
	> 50% exceedance

Structure capacities were estimated for planning purposes only to establish approximate maximum allowable flow rates at each location. Capacity calculations are based on field investigations or existing planning documents (see Appendix E). Shading in Table 2.5 indicates if structure capacity is sufficient (blue), or the severity of capacity exceedances (green to red). Please note that the peak flow rates reported in Table 2.5 correspond to model runs with depth-area reduction factors appropriate for the contributing area of each analysis point (see drainage area, Table 2.5).

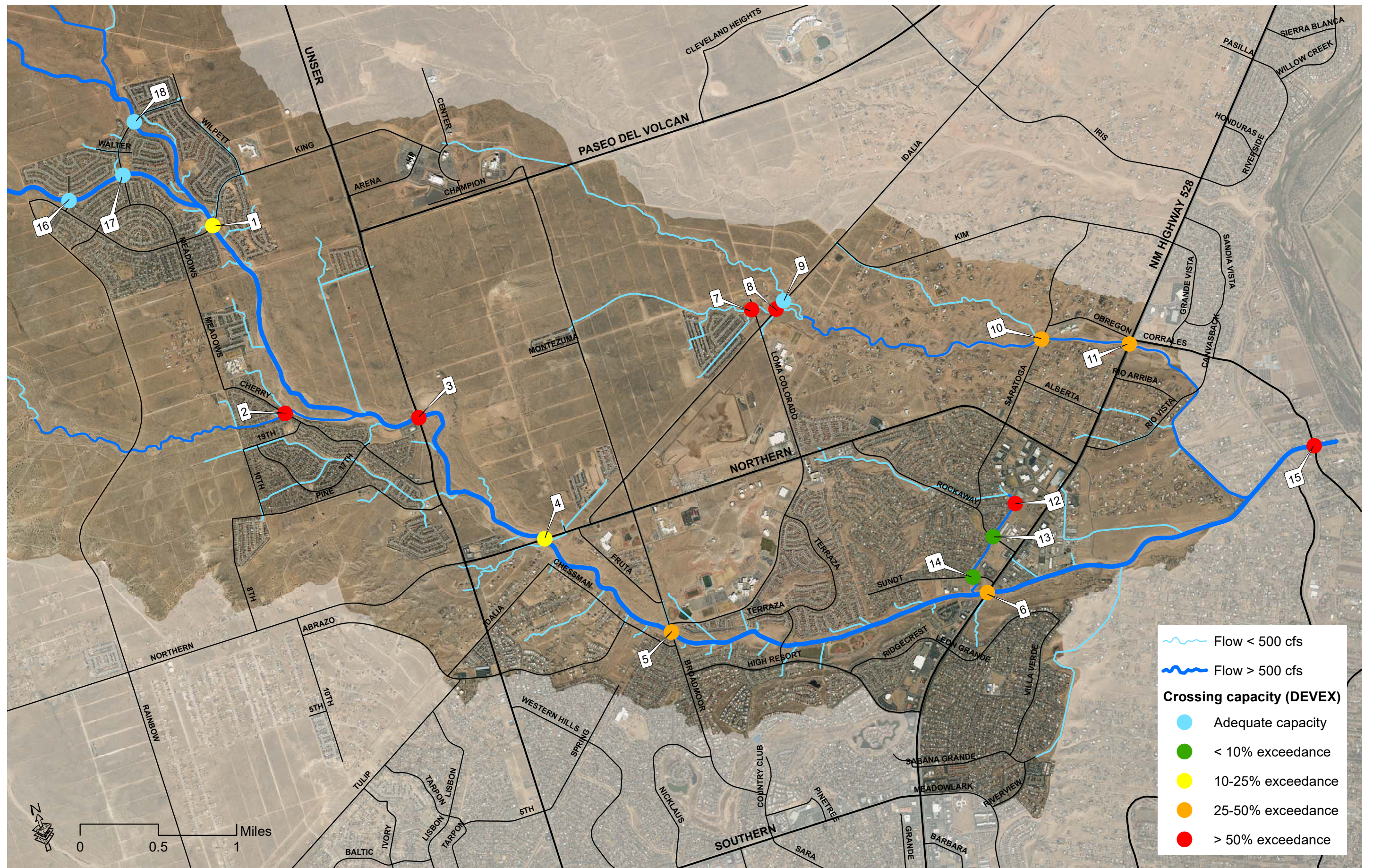


Figure 2.7: Major crossing structures and deficiencies in the Montoyas watershed (DEVEX conditions).

In 2019, a Base Level Engineering (BLE) analysis was completed for SSCAFCA's jurisdictional area by ESP Associates, Inc. in coordination with the University of New Mexico's Earth Data Analysis Center under FEMA's Cooperating Technical Partners program. BLE studies are meant to help communities better predict their flooding risk using estimated base flood elevations.

Hydrologic and hydraulic computations and analyses of the BLE study consisted of determining excess precipitation amounts and calculating Water Surface Elevations (WSELs) for the 10-, 4-, 2-, 1-, and 0.2-percent annual chance flood events. Two-dimensional (2D) hydraulic models were developed for the project area using HEC-RAS version 5.0.7.

Although the 2D analysis can represent impacts from embankments and other features such as storage areas, no structures (i.e. dams, culverts, levees, etc.) were modeled in detail as part of the BLE analysis. SSCAFCA therefore contracted with ESP Associates to update the BLE model for the Montoyas watershed by including culverts with 10% capacity exceedance or more for Existing and/or DEVEX conditions.

Existing conditions inundation maps for culverts 1-11 resulting from the 100-year storm are included below (see Figure 2.8 through Figure 2.15). The full report including inundation maps for DEVEX conditions at each structure is included in Appendix H.

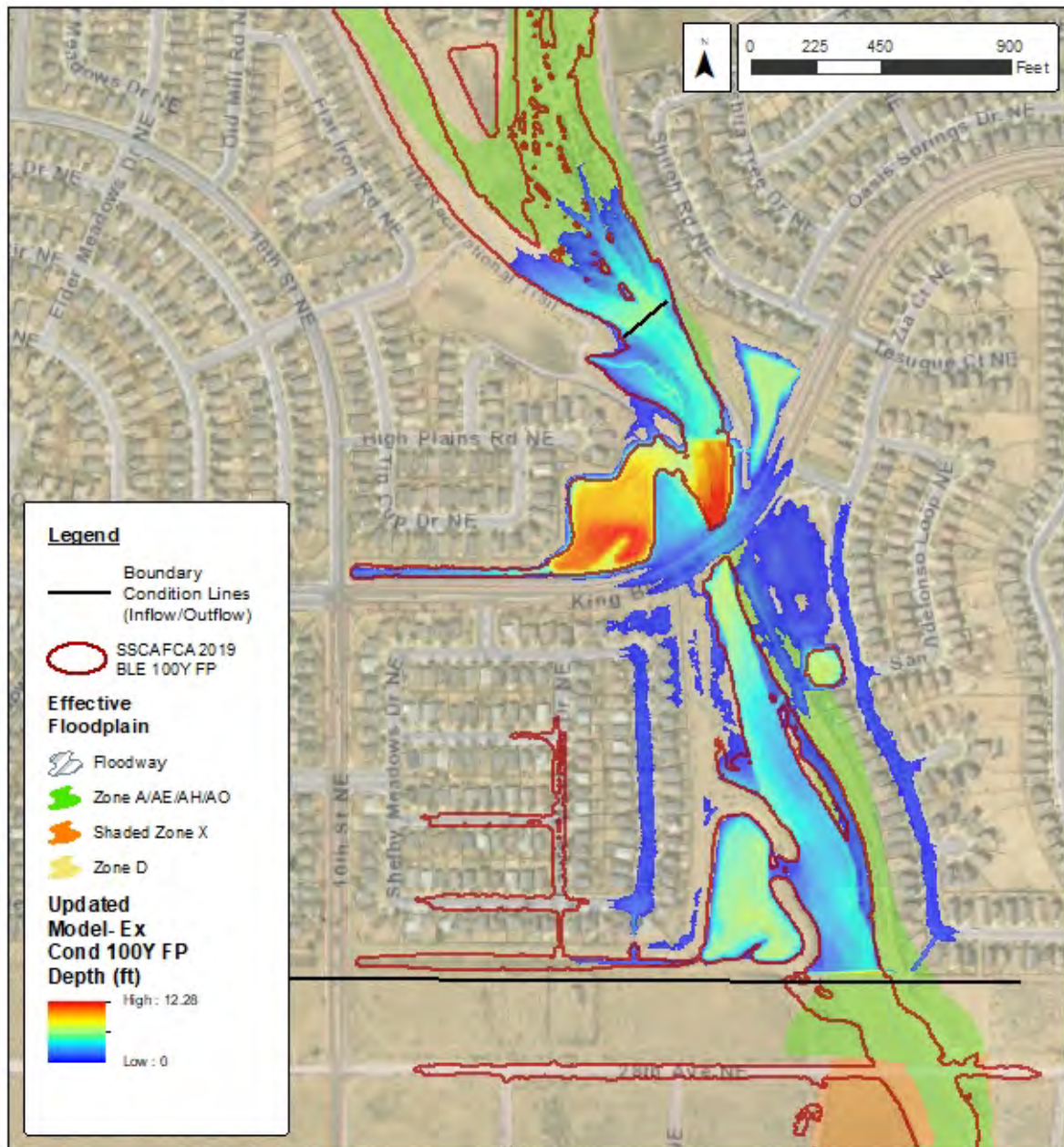


Figure 2.8: Flooding in the vicinity of structure 1 (Montoyas Arroyo and King Blvd.) under Existing conditions.

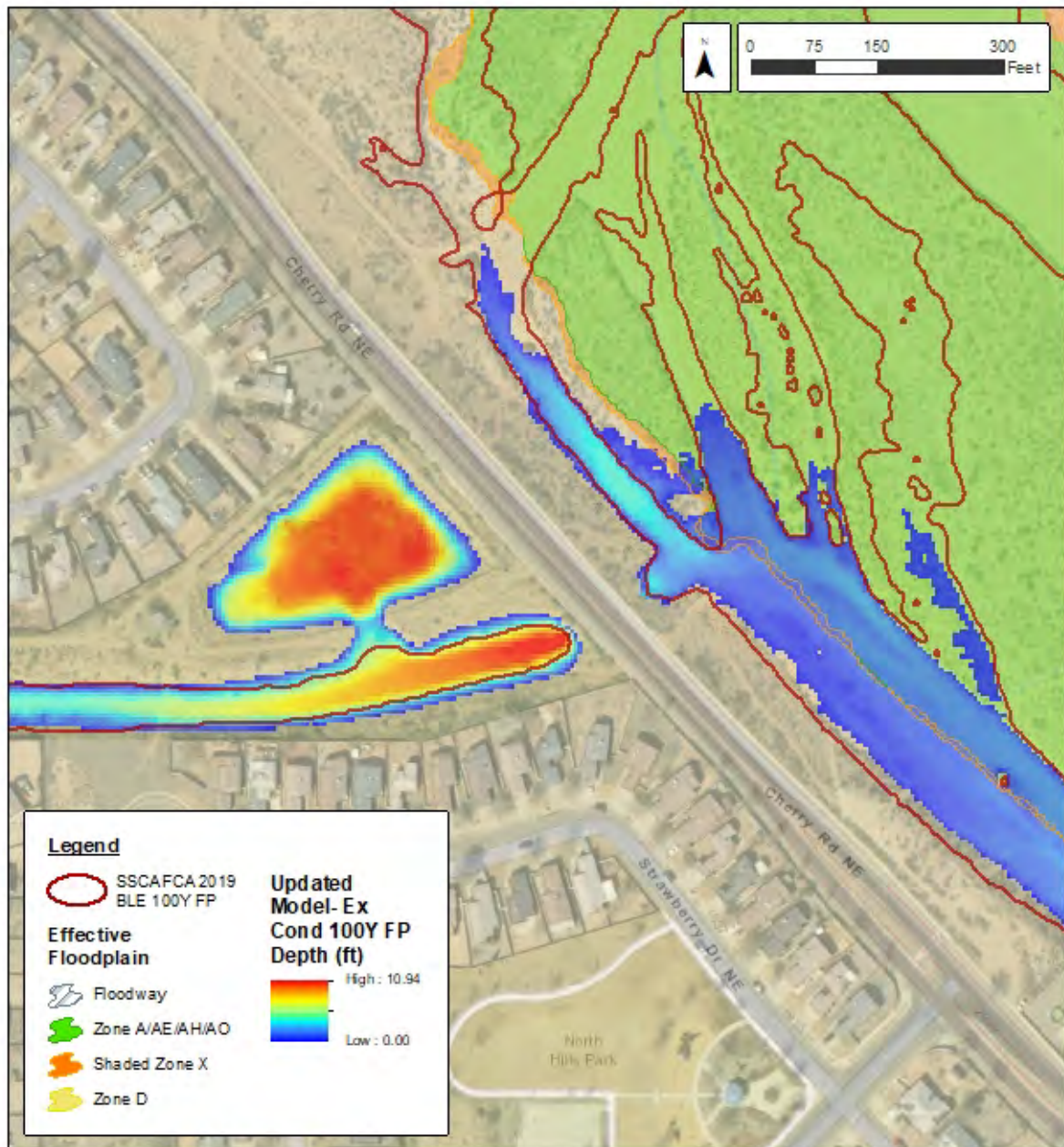


Figure 2.9: Flooding in the vicinity of structure 2 (Tributary G and Cherry Rd.) under Existing conditions.

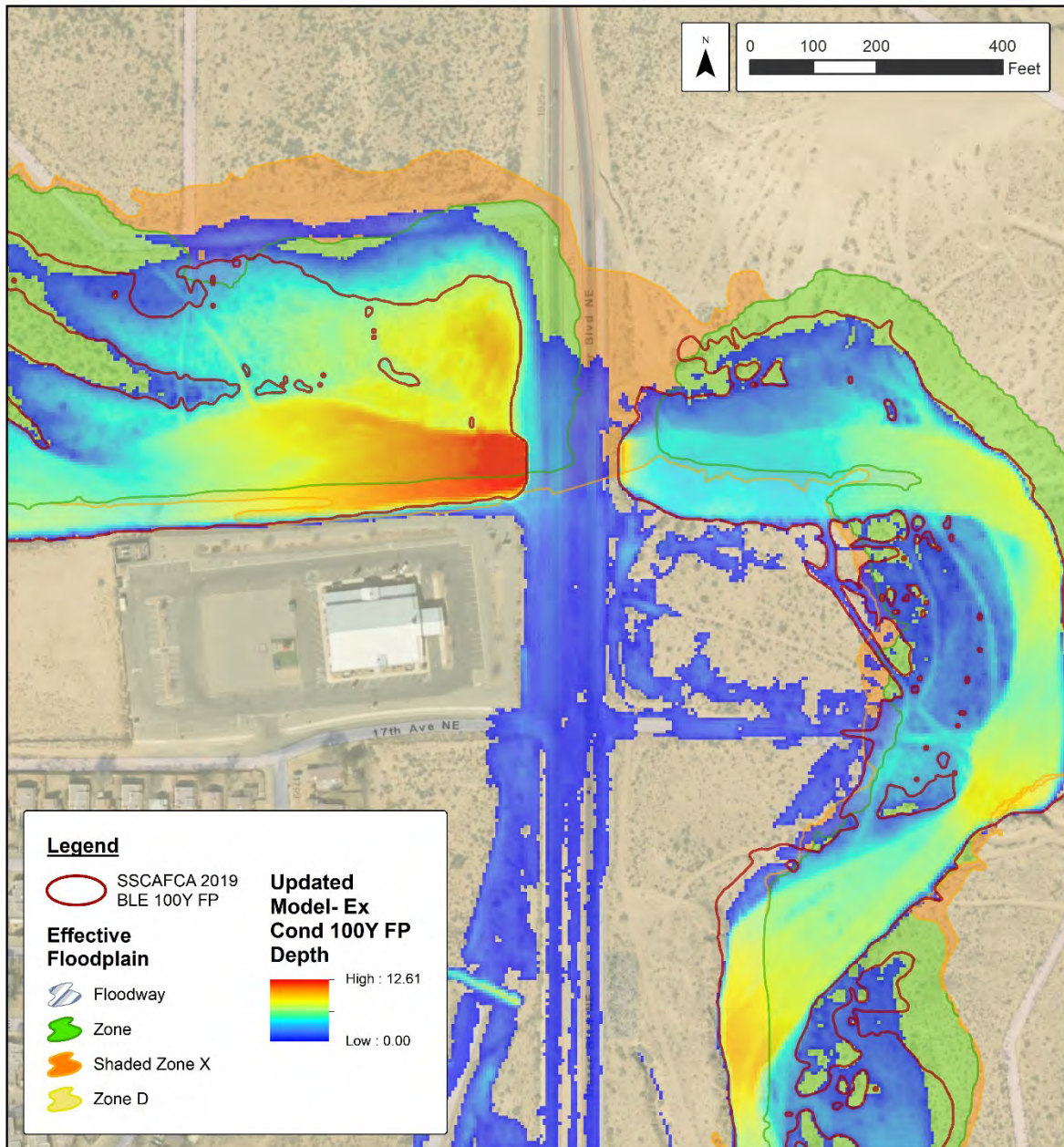


Figure 2.10: Flooding in the vicinity of structure 3 (Montoyas Arroyo and Unser Blvd.) under Existing conditions.

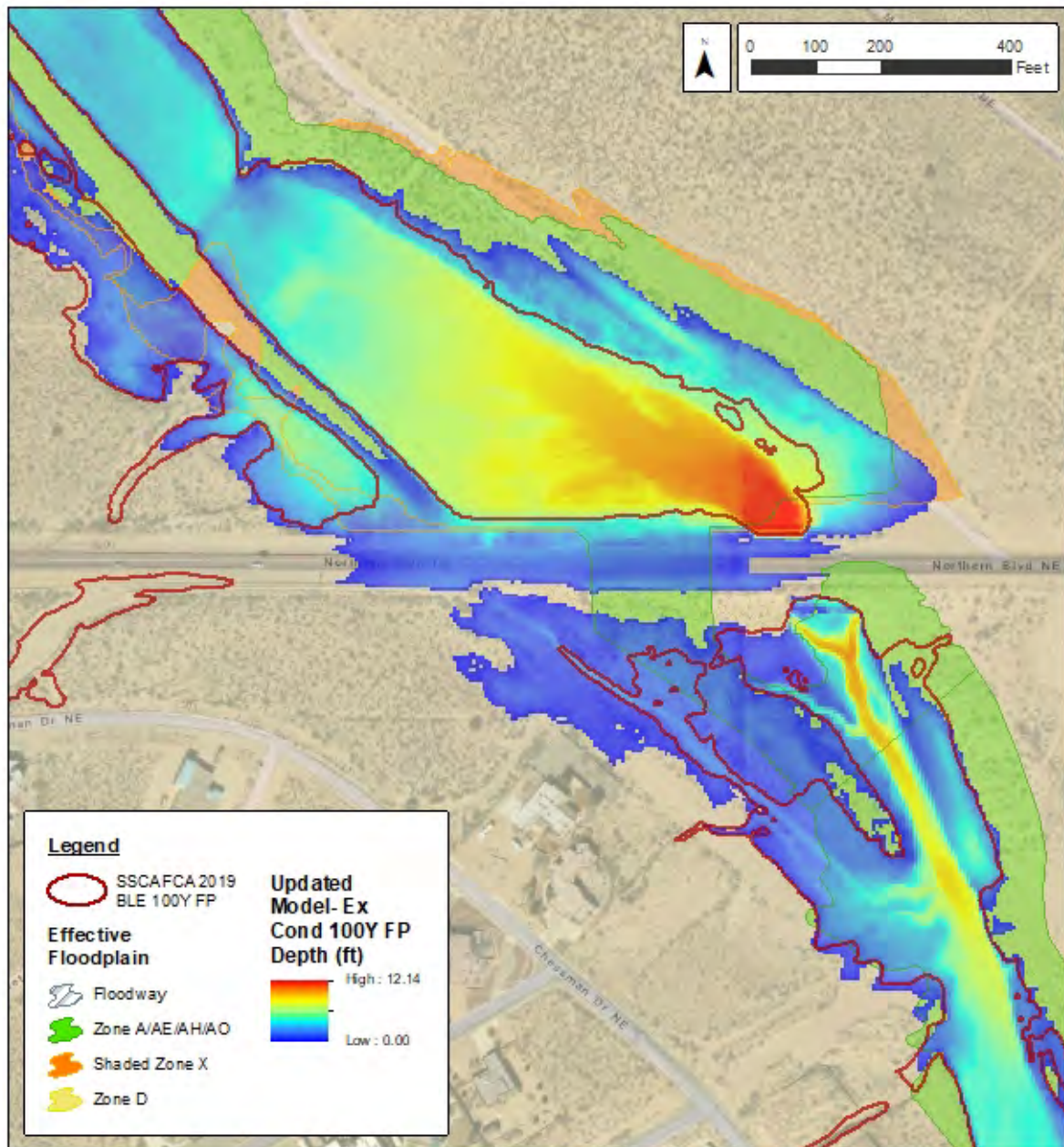


Figure 2.11: Flooding in the vicinity of structure 4 (Montoyas Arroyo and Northern Blvd.) under Existing conditions.

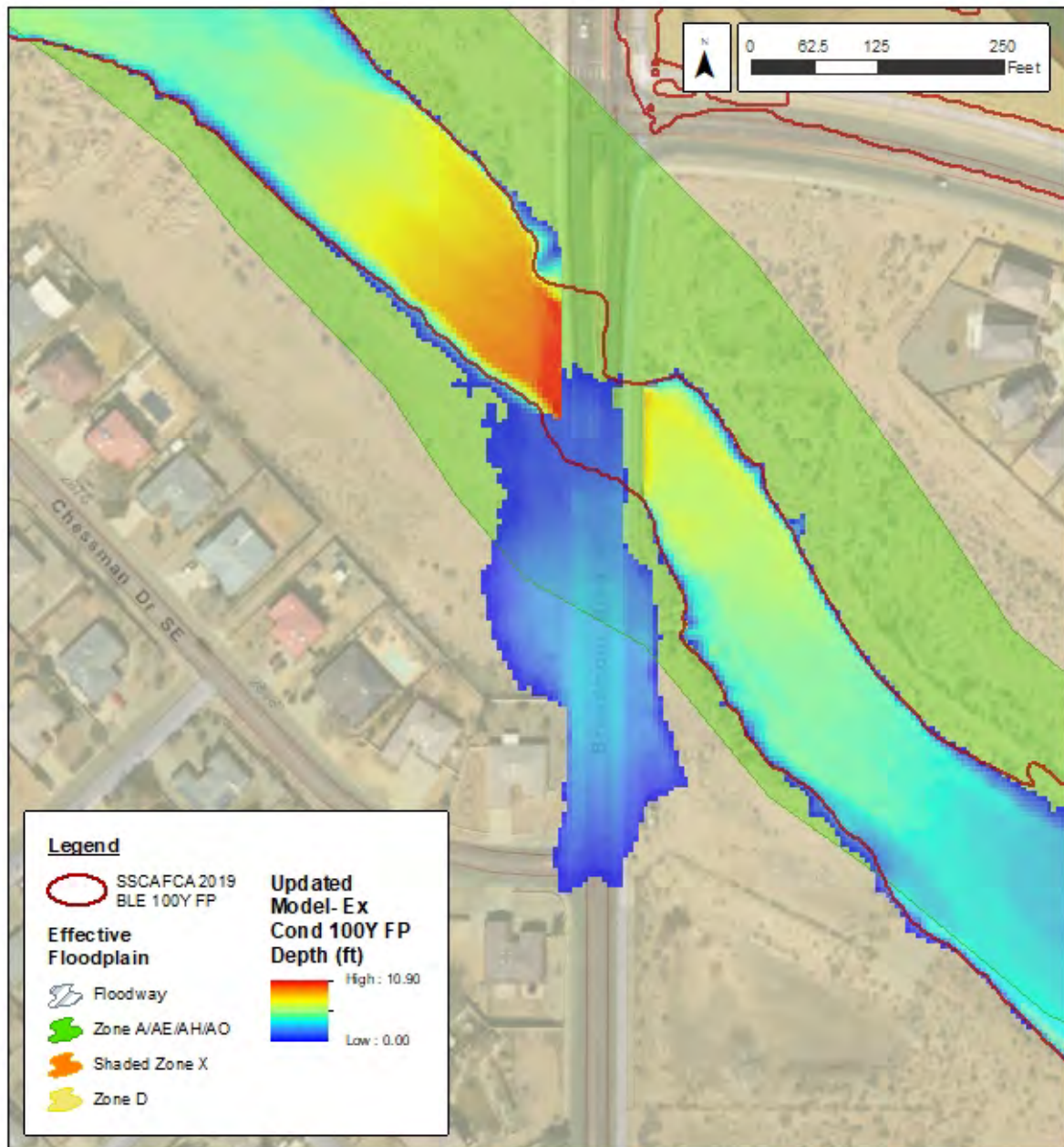


Figure 2.12: Flooding in the vicinity of structure 5 (Montoyas Arroyo and Broadmoor Blvd.) under Existing conditions.

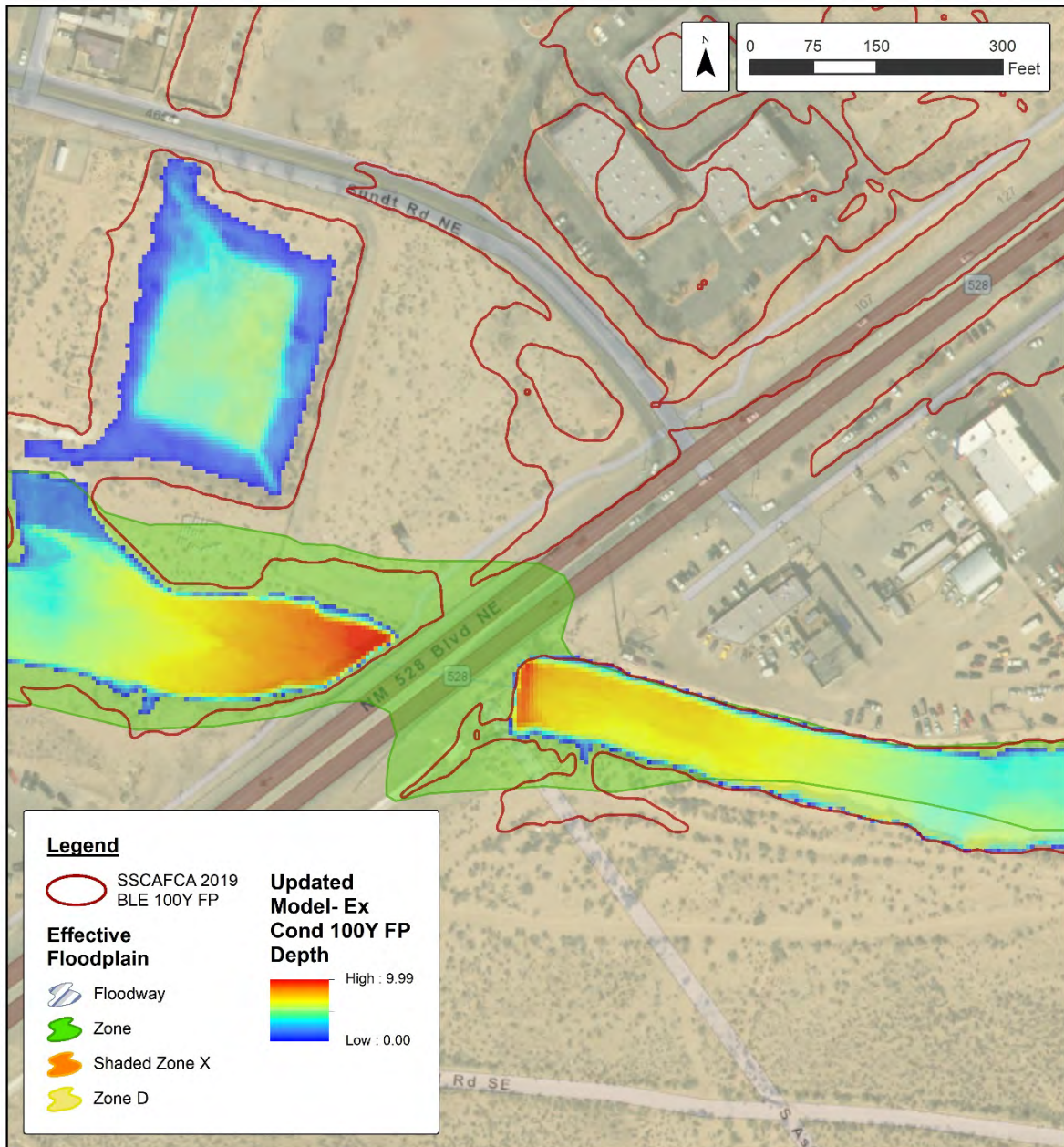


Figure 2.13: Flooding in the vicinity of structure 6 (Montoyas Arroyo and NM 528) under Existing conditions.

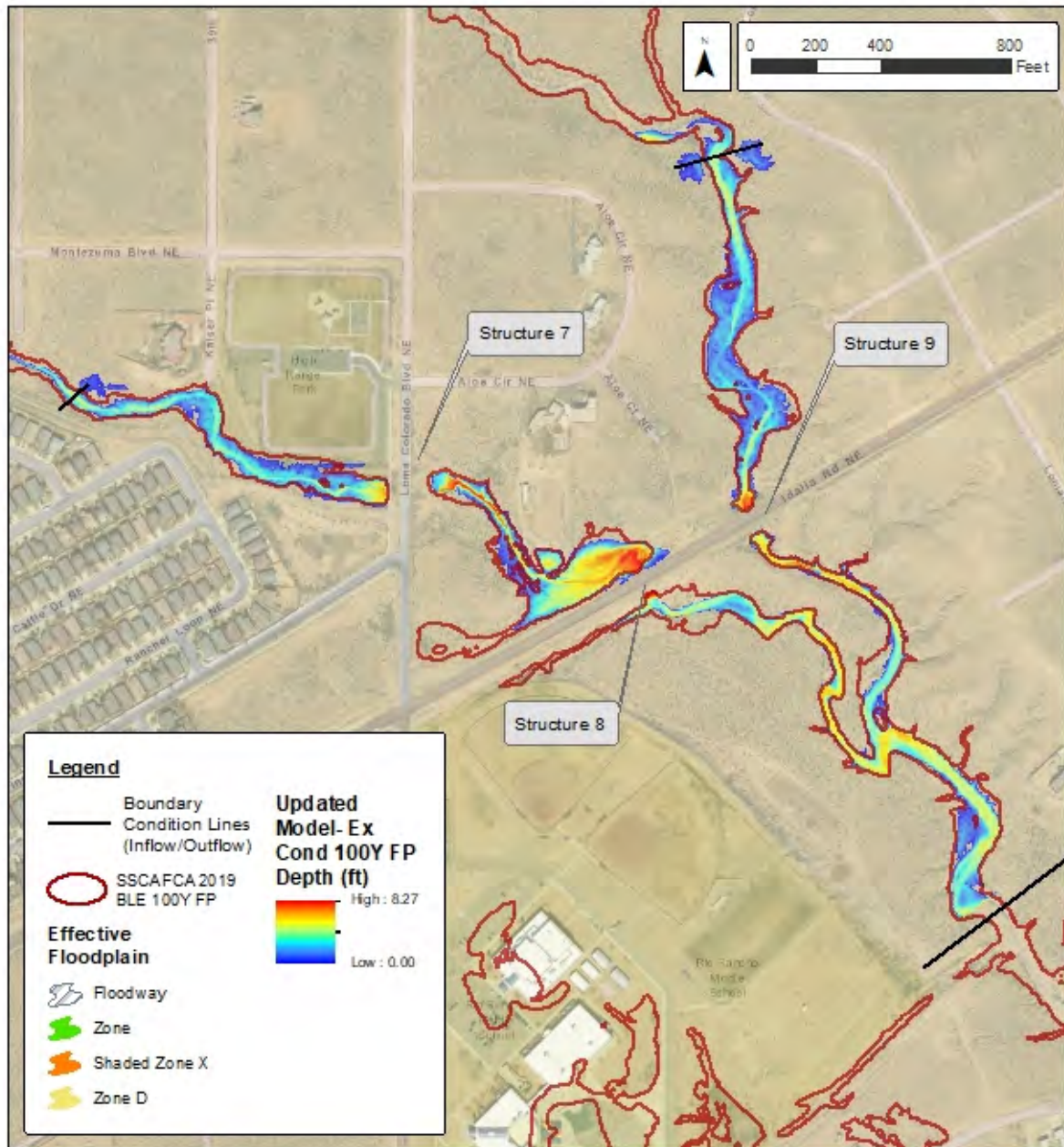


Figure 2.14: Flooding in the vicinity of structures 7-9 under Existing conditions.

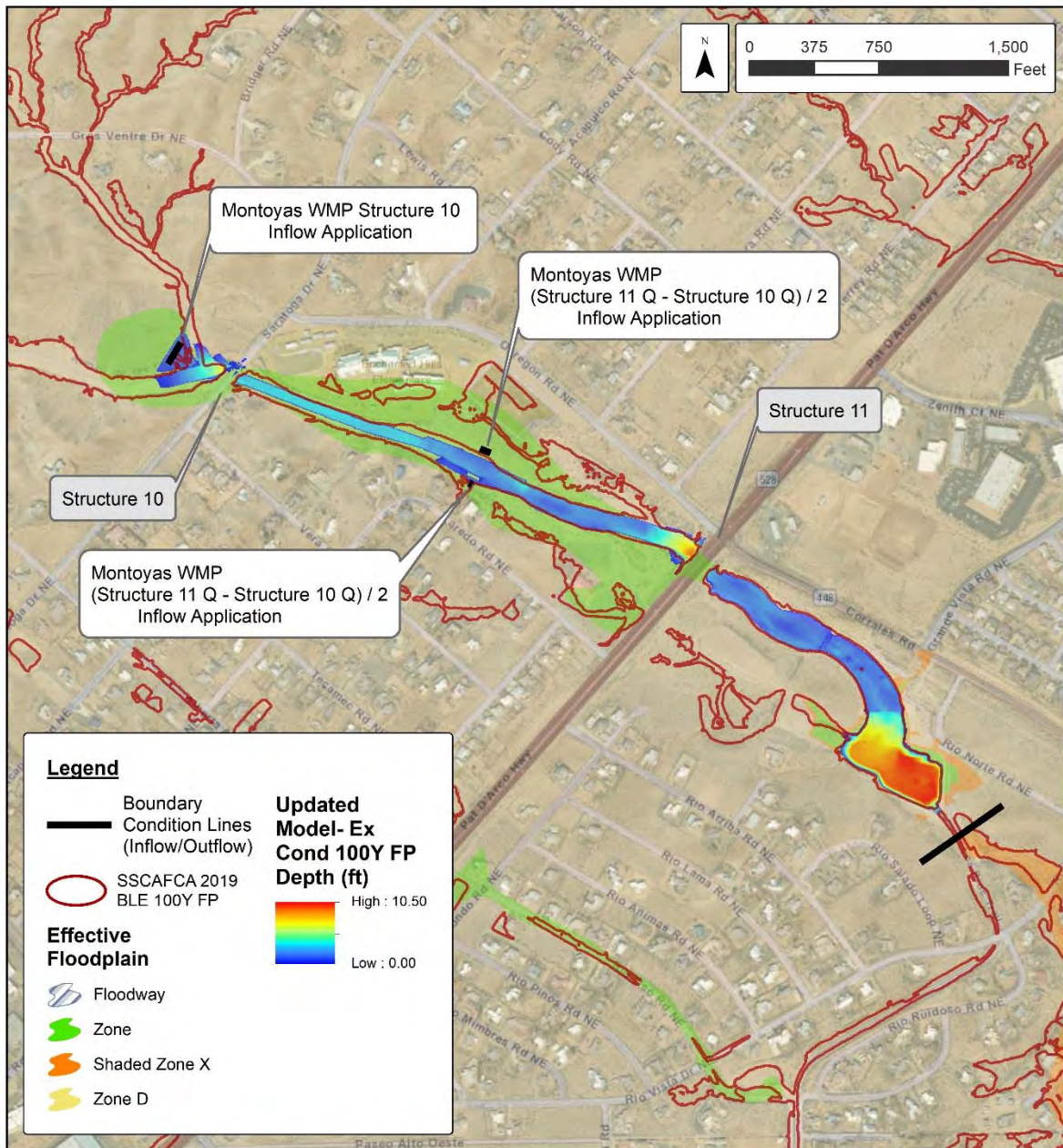


Figure 2.15: Flooding in the vicinity of structures 11 and 12 (Lomitas Negras Arroyo and Saratoga Dr./NM 528) under Existing conditions.

Existing ponds were evaluated for deficiencies based on Existing and DEVEX conditions model runs. Figure 2.8 shows pond locations in the watershed; a larger, more detailed map is included at the end of this document. The following color scheme was adopted to highlight pond deficiencies based on this analysis:

Green (no concern) – ponds have sufficient capacity under Existing and DEVEX conditions.

Yellow (low concern) – activation of the emergency spillway during the 100-year storm under Existing and/or DEVEX conditions, but no expected impact to downstream infrastructure because ponds discharge directly into a publicly owned portion the main stem of the Montoyas Arroyo. Also in this category are ponds with insufficient capacity during DEVEX conditions where plans for upstream improvements are already in place.

Red (high concern) – ponds have insufficient capacity under existing and/or DEVEX conditions and spill onto roadways or private property; also in this category are ponds with limited data where further analysis is recommended.

A detailed table with results for all 60 ponds included in the hydrologic models is contained in Appendix B; the table includes notes for each deficient pond.

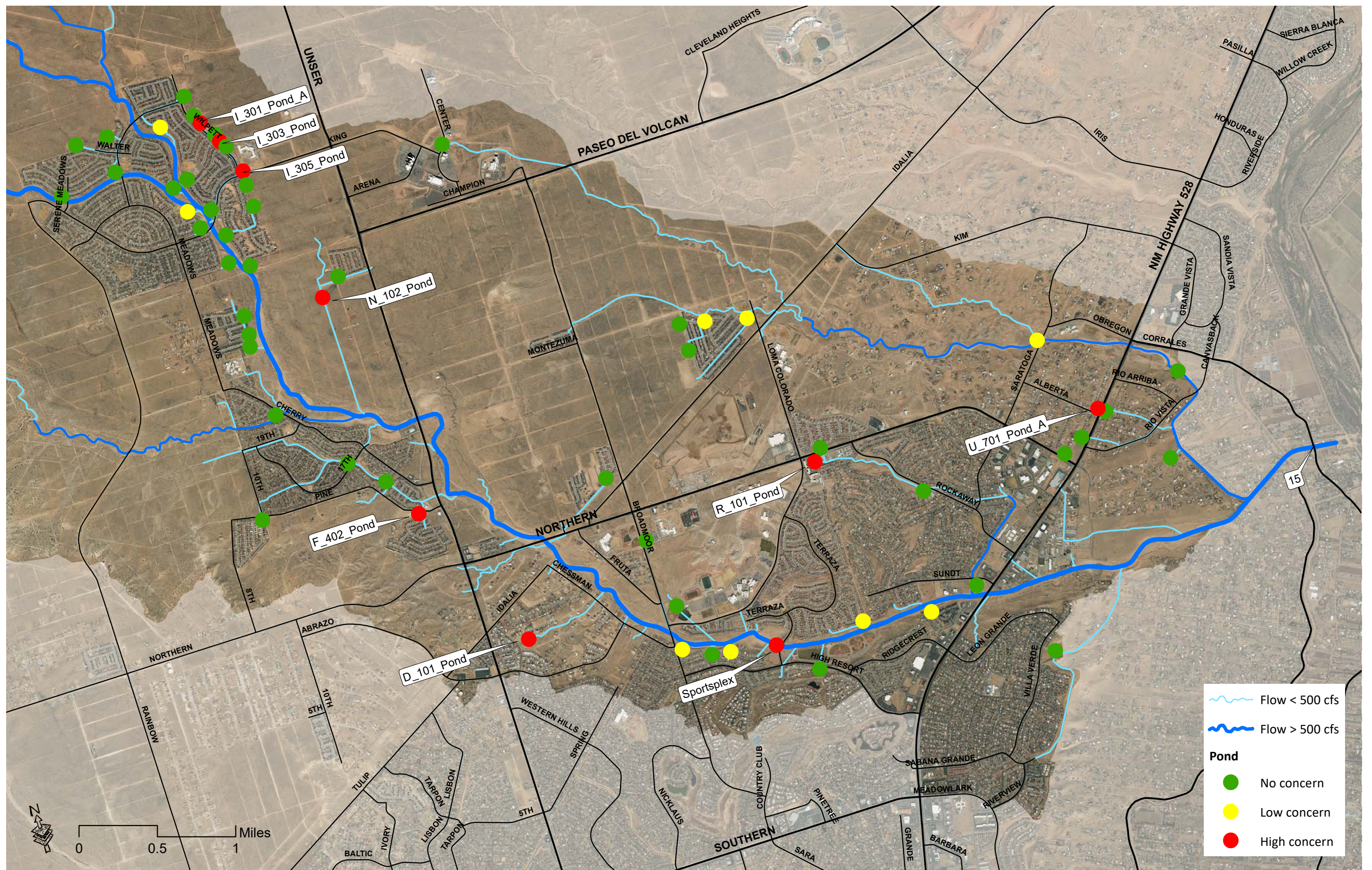


Figure 2.16: Existing stormwater detention ponds in the Montoyas watershed and associated deficiencies.

2.13. Lateral Erosion Envelope

Lateral migration is a natural arroyo process and occurs even in areas that are not impacted by urbanization. In 2008, SSCAFCA published the *Sediment and Erosion Design Guide* (Mussetter, 2008) to provide guidance for evaluating the lateral and vertical stability of arroyos, and for establishing the lateral erosion envelope (LEE). The LEE represents the maximum lateral migration distance of an arroyo that can be expected over the next 30-50 years; it identifies a corridor where properties and infrastructure are potentially at risk from erosion. Figure 2.17 shows mapped lateral erosion envelopes in the Montoyas watershed (see Appendix D for calculations).

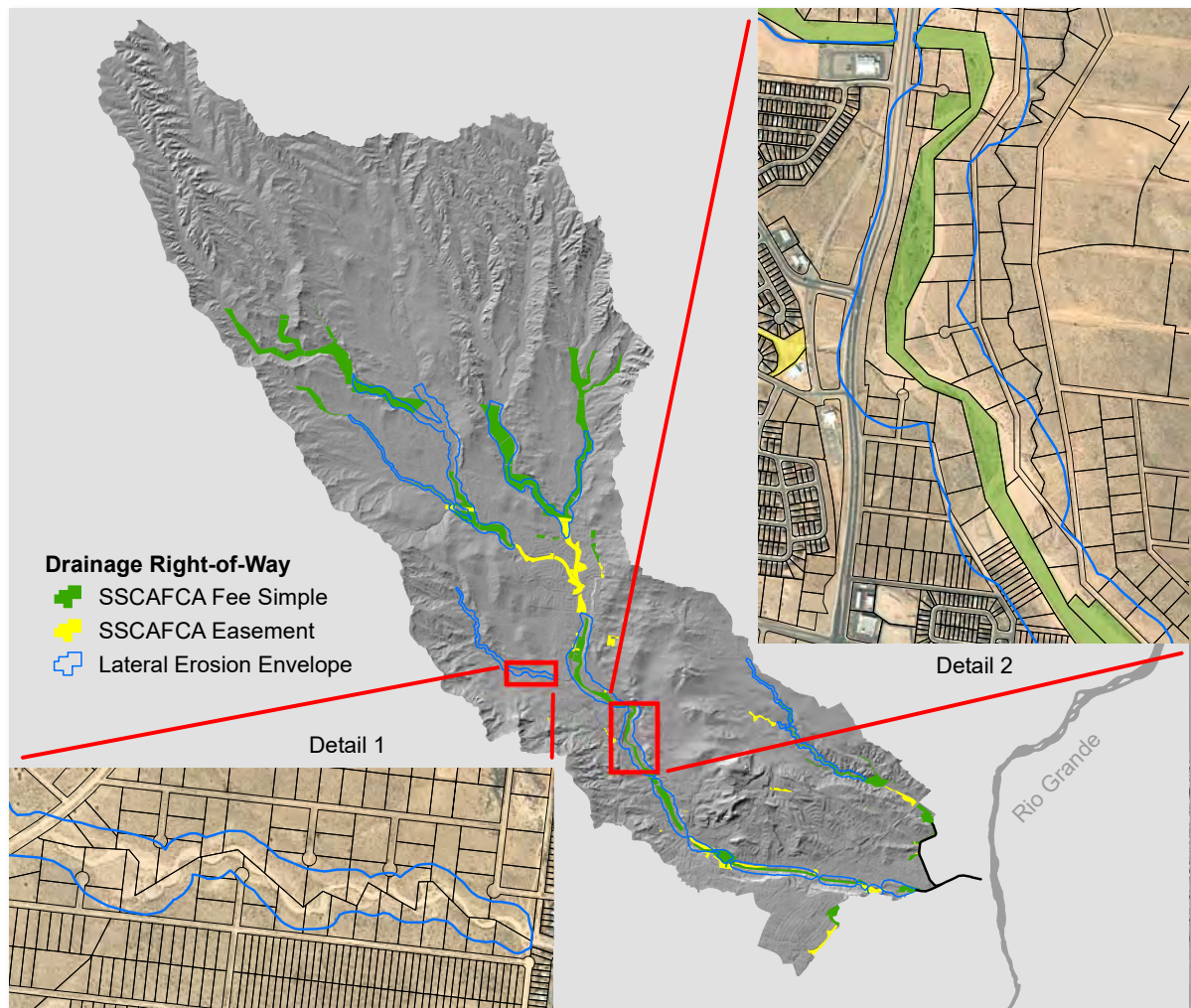


Figure 2.17: Lateral erosion envelopes (blue) in the Montoyas watershed.

2.14. References

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Table 2 – Cultural Resource Summary

Resource No.	Type	NRHP Eligibility Recommendation	Treatment Recommendation
(HCPI 31418) Building 1	Residence	Not eligible	No further treatment
(HCPI 31419) Building 2	Residence	Not eligible	No further treatment
(HCPI 31420) Harvey Jones Channel	Flood water diversion channel	Not eligible	No further treatment
(HCPI 31421) Upper Corrales Riverside Drain	Acequia, drain	Eligible, A	Avoidance
(HCPI 31422) Ditch 3	Irrigation ditch	Eligible, A	Avoidance

Due to modifications that have diminished their integrity and original feel, Building 1 (HCPI 31418) and Building 2 (HCPI 31419) are recommended not eligible for nomination to the NRHP. No further treatment is recommended. The HJC itself (HCPI 31420) is an in-use floodwater diversion channel in good condition. As this channel has been recently constructed and does not serve as an irrigation ditch, HCPI 31420 is recommended not eligible for nomination to the NRHP. No further treatment is recommended. The Upper Corrales Riverside Drain (HCPI 31421) is an in-use, earthen channel. Despite no available information on this particular drain, it is known that the Village of Corrales has been implementing water delivery techniques for rural farming since the 18th century. As a result, HCPI 31421 is recommended eligible for nomination to the NRHP under Criterion A. Therefore avoidance of this structure is recommended during construction. Ditch 3 (HCPI 31422) is an un-named irrigation ditch in fair condition. It too is part of the Village of Corrales historic irrigation system and is recommended eligible for nomination to the NRHP under Criterion A. Similar to the Corrales Riverside drain, avoidance is recommended during construction.

IV. DESIGN CONSIDERATIONS

This section outlines the design concepts intended to meet the project objectives. In addition, this section describes other various elements that must be considered as the project proceeds toward design and construction of any improvements.

A. CONCEPTUAL DESIGN COMPONENTS

The proposed conditions include two main modifications to the HJC to address the sediment accumulation and capacity restraints of the HJC. First, the 3 feet concrete sill will be removed and an earthen sedimentation basin will be constructed downstream of the HJC. This

work will need to be done regardless of which option is selected for increasing channel capacity. Second, an 8 feet span by 6 feet high CBC will be added on the north side of the existing Corrales Bridge. These improvements are described in more detail in the following section.

Following submittal of the Draft DAR, a different alternative was developed. This includes reconfiguring the channel below the Corrales Bridge to facilitate better flow characteristics. The final recommended conceptual design is described in Appendix C.

1. EARTHEN SEDIMENT POND & SILL REMOVAL

The following text describes alternatives considered in fall of 2012. However, due to improved hydraulic analysis and funding constraints the recommended alternative has been modified from the information provided in the remainder of this section. Appendix C provides more current information about the recommended conceptual design.

The current configuration of the HJC and the outfall to the Rio Grande produces a hydraulic jump for most flows in the channel. As a result of this hydraulic jump, the flow velocity is reduced causing sediment to be deposited in the stilling basin. This further reduces the capacity within the channel. To mitigate these effects, the adverse sloping concrete sill will be removed and the existing U-channel will be replaced in this section with a positive sloping U-channel that discharges into the proposed sediment pond. These proposed improvements are shown on Figure 7. The recommended conceptual design (notching a trapezoidal channel into the existing U-channel) is described in Appendix C.

Modeling has shown that the sill modification will push the hydraulic jump downstream, increasing capacity at the bridge and allowing the existing stilling basin to drain.

There are two proposed earthen sediment pond configurations to collect sediment in an area which is much more accessible and have a larger sediment storage volume than the current stilling basin. Therefore, Operation and Maintenance (O&M) costs should be reduced. Furthermore, this sediment basin can act as a water quality facility to remove sediment before it reaches the Rio Grande, both during and following construction. However, primary sediment removal will be accomplished by separate projects upstream of the HJC and Dulcelina Curtis channels.

The bottom of the existing concrete sill is located approximately 670 feet downstream of the existing Corrales Bridge and has an elevation of 5024.3 feet. The top of the sill is located approximately 30 feet downstream of the bottom and has an elevation of 5027.3 and extends to the end of the HJC walls. If the sill is removed by removing the U-channel it will be replaced

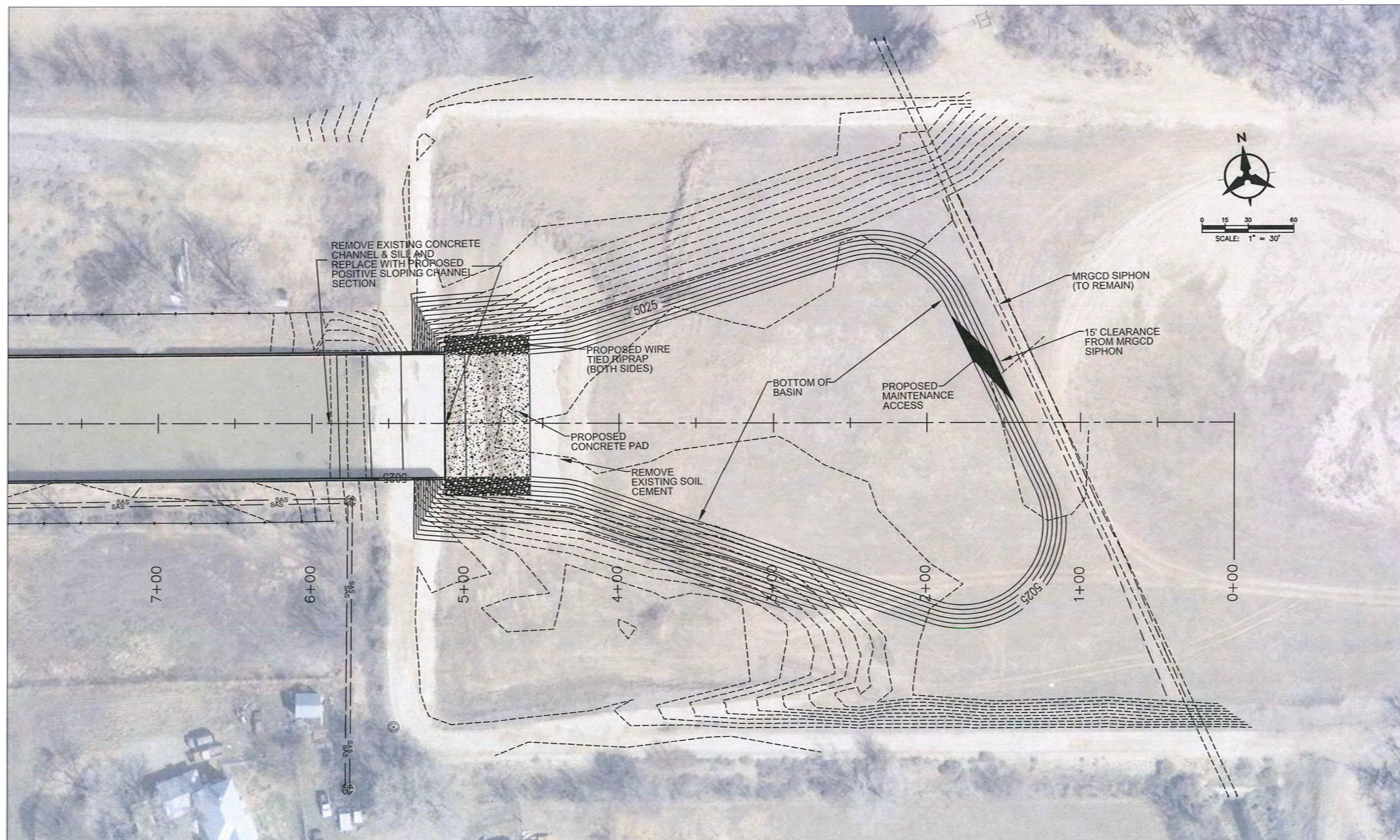


Figure 7
 Harvey Jones Channel Improvements
 Sediment Pond and Sill Removal Conceptual Design

with a channel cross-section that matches the reach immediately upstream. The replaced channel will begin at the upstream end of the existing sill at an elevation of 5024.3 feet and slope down at a 2.4% slope to the end of the existing channel walls where it will outlet to the proposed sediment pond.

The sediment pond, as shown in Figure 7, will be located at the downstream end of the channel walls and will extend to approximately 15 feet west of the existing Middle Rio Grande Conservancy District (MRGCD) siphon. The invert of the pond will be 4 feet below the existing bottom of the concrete sill at an elevation of 5020.3 feet. The bottom of the pond was set at this elevation to be above the groundwater surface elevation determined in the geotechnical investigation. Geo-Test conducted two borings, borings #8 and #9 as shown on Figure 5, within the area of the sediment pond. Based on these results, the groundwater surface elevation was assumed to be at an approximate elevation of 5018.0 feet.

The pond's side slopes will be constructed at 3 horizontal to 1 vertical and will tie to existing ground. Concrete slope paving extending 4 feet up the side slope of the pond will be utilized from the end of the channel wall to the invert of the pond to reduce erosion of the slopes and provide for future maintenance of the channel. The current grading plan requires excavation of approximately 16,000 cy to construct the sediment pond. The actual sediment storage volume from the existing channel invert to the pond bottom is approximately 5.14 ac-ft.

During design of the sediment pond the groundwater level will be confirmed using the piezometer installed by Geo-Test to ensure the bottom of the pond is above the existing groundwater table.

2. ADDING A PARALLEL CBC AT THE CORRALES BRIDGE

The following text describes alternatives considered in fall of 2012. However, due to improved hydraulic analysis and funding constraints the recommended alternative has been modified from the information provided in the remainder of this section. Appendix C provides more current information about the recommended conceptual design.

The existing Corrales Bridge and HJC geometry affects the capacity of the channel when the hydraulic jump hits the bridge. The addition of a CBC at the Corrales Bridge will further increase the capacity of the channel with minimal visual and other impacts.

The proposed structure would be an 8 feet span by 6 feet rise CBC. This additional CBC will be located on the north side of the existing Corrales Bridge as shown on Figure 4. Adding a CBC to the south side of the existing Corrales Bridge was considered as an option, but ultimately eliminated because of an existing SAS that parallels the channel on this side.

The CBC will begin approximately 50 feet upstream of the upstream end of the existing Corrales Bridge and extend approximately 110 feet downstream where the CBC will outlet back into the HJC. The upstream invert of the CBC was set at an elevation of 5024.75 feet and the downstream was set at 5024.21 feet, giving the proposed structure a slope of 0.5%. The CBC inlet and outlet inverts were determined by matching the invert elevations with the existing channel elevations.

With the addition of the CBC, modifications to the existing channel will be needed. The downstream modification would include the removal of approximately 75 feet of the existing channel wall and construction of a new channel wall parallel with the exterior of the proposed CBC.

The modifications upstream of the CBC are more complicated because of an existing road that would need to be returned to its original condition for continued use as a local road. To construct this modification, approximately 20 feet of the channel wall upstream of the proposed CBC would need to be removed. Then a new channel wall and a cantilever overhanging roof will be constructed to connect the CBC and existing channel. Once the upstream channel modifications are complete, the existing road will be replaced to its original condition.

To further decrease the sediment accumulation with the existing CBC, a proposed induced slope in the channel was evaluated. This proposed change of slope within the existing channel would occur along the channel from the limits of the proposed CBC inlet to the proposed CBC outlet and would be constructed as a concrete fillet in the channel. For structural reasons this fillet would require a minimum depth of 5 inches at all connection points in the channel and therefore excavation into the existing channel floor would be needed. To avoid the need to excavate into the floor of the existing two span CBC which serves as the Corrales Bridge, the invert at the outlet of the CBC would need to be raised by 5 inches. With a proposed slope of 0.5% the invert at the inlet of the CBC would be raised by 7.5 inches. This would reduce the existing bridge opening by approximately 8%. Because of the high cost associated the concrete removals, and the reduction in box opening, we are not recommending this improvement.

3. MAINTENANCE CONSIDERATIONS

For all design components consideration has been given to the maintenance requirements and equipment access to the facility.

Currently access to the site is primarily from Romero Road to the south and then following the MRGCD owned levee to the north to the site. SSCAFCA owns a strip of land immediately north of the HJC east of Corrales Road. However, access to this location is limited. It currently requires a 90 degree turn off of the Corrales Road (approximately 160 feet north of the bridge) and another 90 degree turn 15 feet to the east to proceed south toward the bridge. At this point there is another 90 degree turn, which allows access to this strip of land adjacent to the HJC. Due to tight constraints this area is not used for larger equipment access. Furthermore, access to this area would be further reduced due to the proposed configuration of the new CBC and new channel wall downstream of the Corrales Road. However, access by pick-up type vehicles will be maintained, if possible.

The primary improvement proposed to aid in future maintenance will be a vehicle equipment access ramp into the proposed sediment retention pond. Furthermore, the western edge of the pond (which ties into the altered portion of the HJC) will be designed to allow equipment access into the channel itself.

4. FUTURE SEDIMENT REMOVAL PONDS

The Montoyas Arroyo Watershed Management Plan Version 2.0, prepared by Bohannon Huston, Inc. in 2011, proposed two future sediment removal ponds upstream of the Corrales Bridge identified as Pond No.1 and Pond No.2.

Pond No.1 is located at the inlet to the Dulcelina Curtis Channel on the Lomitas Negras Arroyo. This pond was considered a top priority for future watershed management due to the large amounts of sediment that enter the channel at this location. This pond is primarily intended for sediment control, and will store approximately 30 ac-ft of water and sediment.

Pond No. 2 is located at the inlet to the HJC on the Montoyas Arroyo. This pond was considered the second priority in the watershed management plan. This pond will be constructed to reduce the amount of sediment that reaches the HJC.

The future construction of these sediment ponds would greatly reduce the amount of sediment that reaches the Corrales Bridge. Thus, the negative impact of the sediment on the hydraulic capacity of the Corrales Bridge will be reduced.

Currently SSCAFCA intends to pursue these projects, and they are included in their planned funding for the near future.

B. HYDRAULIC ANALYSIS

Utilizing Army Corps of Engineers hydraulic program HEC-RAS, an existing and proposed conditions models were used for both channel configurations in this hydraulic

analysis. Comparison of the models allows an evaluation of the increase in capacity of the facility by adding a CBC and removing the concrete sill at the end of the channel. An analysis was also conducted to determine the effects of sediment deposition in the channel on the CBC capacity. The existing conditions model is based on the HEC-RAS model that was used in BHI's 1999 hydraulic analysis of the HJC with updated elevations and slopes determined from a survey that was completed as part of this DAR. The proposed model expands on the existing model and includes the proposed additional CBC, piers between the CBC's, removal of the 3 feet sill, and the addition of the sediment pond.

In addition to the HEC-RAS models, an oblique wave analysis was conducted to determine the effects of waves on the hydraulic capacity.

1. EXISTING AND PROPOSED HEC-RAS MODELS

The existing conditions HEC-RAS model was updated from the 1999 HEC-RAS model to include 2012 field survey information that was provided for this report. The Manning's N value used for the floor was 0.018 and 0.015 was used for the walls. The N value for the floor was determined from sediment samples taken in the field during the BHI 1999 hydraulic analysis of the Harvey Jones Channel. The upstream boundary condition was based on normal depth. Normal depth was calculated using a 0.016 ft/ft slope based on as-built information upstream of the study area.

Subsequent to the completion of the draft drainage report, a revised hydraulic analysis was performed. The results of the review HEC-RAS models is contained in a memo entitled, "Harvey Jones Improvements – Supplemental Hydraulic Analysis," dated February 5, 2013. The memo is contained in Appendix B.

2. OBLIQUE WAVE ANALYSIS

An oblique wave is caused by the expansion of the cross-sectional width of a channel. Just upstream of the Corrales Bridge, the HJC section expands from 40 feet to 46 feet width. This may cause a hydraulic jump within the channel during supercritical flow which would adversely affect the hydraulic capacity at the Corrales Bridge. Therefore, an oblique wave analysis was conducted at this location to determine the effects of these waves on the bridge capacity.

The oblique wave calculation requires inputs of an alpha angle in degrees (α , deflection angle of vertical wall), Froude number (Fr), and initial flow depth (y_1) at the specified flow rate at the expansion. The calculated outputs from these inputs determine the beta angle in degrees (β , total deflection angle) and the final flow depth (y_2). The angle alpha is defined as

the deflection angle of the vertical channel wall. The calculations for this wave analysis utilized the program WAVE developed by BHI to perform the analysis. The calculations performed by this program are shown below:

- $\sin \beta = \frac{1}{Fr} \sqrt{\frac{1}{2} \frac{y_2}{y_1} \left(\frac{y_2}{y_1} + 1 \right)}$
- $\frac{y_2}{y_1} = \frac{\tan \beta}{\tan(\beta - \alpha)}$

Since the channel is not symmetrical at the point of the expansion, two alpha angles were determined. The south side had an alpha angle equal to 8.65 degrees while the north side had an angle of 6.20 degrees. The alpha angle from the south side of the channel was used because this angle caused the largest change in flow depth. See Appendix C for a discussion of oblique wave.

C. TRAFFIC CONTROL

The information provided in this section is relevant to the option described above of constructing an additional CBC at the Corrales Bridge. However, the recommended concept currently being designed (described in Appendix C) will not require a detour for the traveling public. Therefore the detours discussed below will no longer be required as part of the project.

Middle Rio Grande Council of Governments (MRCOG) data shows that in the vicinity of the project the Average Workday Traffic Count (AWTC) is 4,600 to 4,800. That number increases to 6,000 at the northern terminus of NM 448 at the intersection with Coors Blvd. At the southern terminus (intersection with Alameda Blvd.) the AWTC for NM 448 is 17,000.

BHI has evaluated three options for temporary traffic control during the proposed construction of the HJC improvements. Descriptions of these options including, advantages and disadvantages, are evaluated in the following sections.

1. OPTION 1 – EXTENDED DETOUR (CORRALES ROAD CLOSURE)

Option 1, shown on Figure 10, would close traffic access in both directions across the Corrales Bridge during the construction of the additional CBC. Due to the closure of traffic in both directions, a traffic detour would need to be established. A preliminary detour route has been proposed for this study. The traffic detour from south to north is as follows: At the intersection of Corrales Road and Camino Todos Los Santos travel west on Camino Todos Los Santos to Loma Larga Road. Travel north on Loma Larga Road to Paseo Tomas Montoya. Travel west on Paseo Tomas Montoya to Tierra de Corrales Road. Travel north on Tierra de Corrales Road to Calle Contenta. Travel south and east on Calle Contenta to Paseo Cesar

MEMORANDUM

DATE: February 5, 2013
TO: SSCAFCA
FROM: Bohannon Huston, Inc.
SUBJECT: Harvey Jones Improvements – Supplemental Hydraulic Analysis

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Introduction

This memo is a supplement to the DRAFT Design Analysis Report (DAR) prepared for the Harvey Jones Channel Improvements Project dated November 21, 2012. It summarizes the hydraulic analysis performed at the Harvey Jones Channel from upstream of the existing Corrales Road Bridge to the end of the SSCAFCA property where the Middle Rio Grande Conservancy District's (MRGCD) inverted siphon is located (see Figures 4 & 7). This memo also discusses the assumptions that are included in the hydraulic analysis along with a discussion of the results. The hydraulic analysis contained herein is assuming that there is no sediment deposition in Harvey Jones Channel (HJC). We recognize however, and as we have seen from previous storm events, that any sediment deposition under or downstream of the existing bridge degrades the hydraulic capacity of the structure, resulting in the formation of a hydraulic jump occurring upstream of the bridge, as shown in the following picture.



Existing Conditions

As discussed in Section IV.B.1 Existing Conditions of the DAR, the existing channel upstream of the bridge is 40 feet wide and 10 feet deep, with an upstream slope of about 1.6%. At the bridge, the channel transitions to a 0.0% slope section and the width widens to 46 feet. The bridge is comprised of two CBC's that are 22.5' x 8.3' (w x h). From survey data of the bridge, the low chord is at an elevation of 5032.4. Downstream of the bridge, the channel then gradually transitions to 80 feet wide and remains flat for about 700 feet, ending with a concrete sill that is 3 feet in height.

Proposed Improvements

The proposed improvements contain several elements that have been analyzed in combinations of specific options. The following describes the elements:

- **Trapezoidal channel** incised inside the vertical wall channel inducing a vertical slope that continues to a sediment pond. This Option, labeled as number 1, is an alternative to building the 8'x6' CBC.
- **Parallel 8'x6' CBC** next to the existing CBC's. The additional CBC is contained in all sub-options to Option 2.
- **Concrete Sill Removal** is proposed in all sub-options. However, there are two different ways in which the sill could be removed:
 - **Partial Floor Removal** – this would be accomplished by constructing a trapezoidal channel (40' bottom width, 1:1 side slopes) at the upstream end of the sill. The walls would be left intact and the edge of the trapezoidal channel section would cut through the concrete floor to the end of the concrete channel. The bottom of the trapezoidal channel would be vertically sloped to the sediment pond.
 - **Floor and Wall Removal** – this entails removing the concrete sill in its entirety, which includes the floor and the walls. A vertical wall channel would be reconstructed sloping from the invert upstream of where the sill was removed to the sediment pond.
- **Sediment Pond** – a sediment pond is proposed downstream of the concrete channel to provide additional temporary storage of sediment from summer thunderstorms. The goal of the sediment pond is to reduce the amount of sediment forming under the bridge and in the downstream flat vertical channel. Two different size sediment ponds are proposed:
 - **1.5 Acre Pond** – construct a sediment pond (see Figure 7) maximizing the temporary storage volume, while still providing area for storage of earth to be removed from the pond and stored temporarily on site.
 - **0.7 Acre Pond** – construct a more linear type pond (80' bottom width [bw], 4:1 side slopes [ss]) that provides the minimum amount of temporary storage.

The options analyzed are described as follows:

Option 1 – Trapezoidal Channel

The sub-options presented below for Option 1 have common elements including a trapezoidal channel constructed inside the existing 80' wide "U" channel and a sediment pond built at the

end of the existing channel. The trapezoidal channel would have a 40' bottom width, 2:1 side slopes (horizontal to vertical) and begin just downstream of the existing bridge. The trapezoidal channel would vary in depth from zero near the bridge to about 2' deep at the end of the existing channel, with a bottom slope of about 0.4-0.5%.

- Option 1-A – this option includes the trapezoidal channel, removal of the sill floor and walls, and the 1.5 acre pond.
- Option 1-B – includes the trapezoidal channel, with only the floor of the 3' sill removed (walls preserved) and the 1.5 acre pond.
- Option 1-C – includes the trapezoidal channel, with only the floor of the 3' sill removed (walls preserved) and the 0.75 acre pond.

Option 2 – 8'x6' CBC

The sub-options for Option 2 include the 8'x6' CBC, removal of the concrete sill and construction of a sediment pond.

- Option 2-A – this option includes building the 8'x6' CBC, complete removal of concrete sill floor and walls (replacing with a vertically sloping vertical wall channel) and constructing the 1.5 Acre Pond.
- Option 2-B – this option includes building the 8'x6' CBC, complete removal of the concrete sill and walls (replacing with a vertically sloping vertical wall channel) and the 0.7 acre pond.
- Option 2-C – this option includes building the 8'x6' CBC, partial sill floor removal (replacement with a downward sloping concrete trapezoidal section incised in the concrete sill floor) and the 0.7 acre pond.

Hydraulic Modeling Assumptions

A number of assumptions are required for the HEC-RAS computer model. Discussed here are the assumptions:

- Hydraulic analysis assumed little sediment deposition under the bridge or downstream.
- BHI performed an analysis of the sediment effects on the channel roughness. Sediment samples were obtained and the procedure from the USACE manual EM 11102-1601 was used from previous SSCAFCA study and composite N value of 0.018 was calculated.
- The contraction and expansion coefficients used with this model are based on the HEC-RAS River Analysis System – Hydraulic Reference Manual Version 4.1, January 2010. Where no change in section occurs, which is the case for the existing channel from about 60 feet upstream of Corrales Bridge upstream, and from about 140 feet downstream to the end, the contraction and expansion values are zero. For gradual transitions, such as existing upstream and downstream of the bridge, the contraction and expansion values are 0.01 and 0.03 respectfully. For more severe transitions, such as near the entrance to the proposed 8'x6' CBC the values for contraction and expansion are 0.05 and 0.2, respectfully. For subcritical flow areas with sever transitions, such as the exit of the existing channel, the values are 0.1 and 0.3 respectively.

- The HEC-RAS model was run using the mixed option which can help in defining locations where a hydraulic jump might occur (sudden change in state from supercritical to subcritical).
- The two existing CBC's and Options 1 and 2 were modeled using the HEC-RAS bridge routine. The following describes the assumptions used for the bridge modeling:
 - Bridge piers were modeled using an additional foot of width to reflect any debris that could be caught by piers (wall between CBC's).
 - The momentum equation and the energy equation options were used to analyze low flows when flows were below the low chord of the bridge. The option in the program to use the highest energy answer was turned on. The default (recommended) values for the momentum equation were used.
 - For high flow condition, computations were run separately for both the standard step method (energy equation) and the pressure/weir flow method. Results from both analyses are discussed.
 - The water surface option was used for testing the computations to determine if the bridge hydraulics upstream has the potential of entering into a pressure flow condition (water surface above the low chord).

Oblique Waves

As discussed in the report, oblique waves can form as a result of transitions and channel obstructions such as piers or walls between CBC's when the flow is supercritical. Although a momentum balance is computed by HEC-RAS, the program does not address the formation or propagation of oblique waves. As described in the report, hydraulic affects from wave propagation could cause the flow depths upstream of the bridge to be approximately 1.0 to 1.5 feet higher than the depths predicted in HEC-RAS, upstream of the bridge. The result of this means that any water surface calculation in HEC-RAS, which is within approximately 1.0 of the low chord, has the potential of resulting in a hydraulic jump upstream of the bridge.

We also recognize the formation of oblique wave is possible at the entrance to the proposed 8'x6' CBC. For purposes of this study, we have assumed any flow upstream that is within 1.5 feet of the existing bridge low chord could cause a hydraulic jump.

HEC-RAS Hydraulic Results

Presented here are the results from the hydraulic computations for existing conditions and proposed channel improvement options described previously.

Existing Conditions

Figure B-1 shows the resulting hydraulic profile using the energy equation method for analyzing high flow conditions. Under this condition, it can be seen that a flow of 3,000 cfs can result in a hydraulic jump occurring at the bridge. However, flows of about 4,000 cfs and higher will result in a hydraulic jump that is moved downstream due to the higher specific energy of the upstream supercritical flow. This leads to the conclusion that lower flows have insufficient energy to "push" the hydraulic jump downstream. In addition, the concrete sill creates a flow constriction resulting in a weir flow condition at the sill, causing backwater condition upstream. For flows greater than 3,000 cfs, the backwater depth affect from the sill is higher than the low chord of the bridge.

Figure B-2 shows the hydraulic profile using the pressure/weir flow routine in HEC-RAS for the bridge modeling of high flows. This figure shows the capacity of the bridge once a hydraulic jump has formed upstream of the bridge. In this hydraulic condition, the maximum flow that can go through the bridge opening is 4,000 cfs. This capacity calculation is based on maintaining two feet of freeboard between the water surface and the top of the channel.

Proposed Improvements

This section discusses the results for Options 1 and 2 scenarios. Because the sub-options of Option 1 are not significantly different hydraulically from each other, and likewise the sub-options of Option 2 are not significantly different, only results of Option 1-C and Option 2-C are presented and discussed.

Option 1-C (Trapezoidal Channel, Sill Floor Removed, 0.75 Acre Sediment Pond)

Figure B-3 shows the results of the water surface profile for Option 1-C using the energy equation bridge method in HEC-RAS. With the concrete sill partially removed and the construction of a sloping trapezoidal channel, the downstream water depths are about 5 feet lower than existing conditions. For lower flows, it appears there is a hydraulic rise to the weir flow condition that is present in the location of the MRGCD inverted siphon. For higher flows, the water depth in the concrete trapezoidal/"U" channel is slightly higher than the weir flow depth at the inverted siphon. In either case, it can be seen that the hydraulic jump is still occurring as a result of the inverted siphon. However, the location of the hydraulic jump occurs much farther downstream of the bridge than as compared to the existing condition. The hydraulic jump (depending on the flow) occurs between 350 and 500 feet downstream of the Corrales Bridge. And in addition, for flows up to 6,800 cfs, the water depth is below the low chord of the bridge.

Figure B-4 shows the results of the water surface profile for Option 1-C using the pressure/weir bridge method. In this routine HEC-RAS seems to indicate that pressure flow could occur above 4,000 cfs. However, the HEC-RAS routine that is predicting the potential for pressure flow is not relevant in this case. In addition, we assume that one of the section to section step-method hydraulic balance iterations most likely causes the flag causing the program to assume the pressure flow had occurred. For pressure flow to occur, the water depth would have to have hit the low chord. However, the fact that the downstream subcritical flow depths are well below the low chord of the bridge indicates that hydraulic jump has not occurred upstream of the bridge. In fact, as shown in Table B-1, for the flow of 6,800 cfs the subcritical flow depth downstream is about 1.4 feet below the low chord of the bridge.

Option 2-C (8'x6' CBC, Sill Floor Removed, 0.75 Acre Sediment Pond)

Figure B-5 shows the resulting water surface profile for Option 2-C using the energy equation bridge method in HEC-RAS. With the concrete sill partially removed, water depth for the flows downstream of the bridge are approximately 2.8 feet lower than existing conditions. However, the flow depth in the downstream reach is still controlled by the channel section and flat slope. The flow depth control section (weir flow) at the downstream end of the analysis is located at the MRGCD inverted siphon and is lower than the depth in the channel, so it is not influencing the flow depth in the concrete channel. Based on the energy equation analysis, if supercritical flow is maintained through the bridge then the hydraulic jump will occur downstream of the bridge. In reviewing the analysis, one can see that based on the HEC-RAS momentum balance routine, in lower flows the hydraulic jump occurs closer to the downstream end of the bridge. The

maximum capacity of the structure under this condition is 7,000 cfs with the hydraulic jump potentially occurring about 300 feet downstream of the bridge. However, the flow depth downstream is about 1 foot higher than the low chord elevation; and therefore, any hydraulic instability that might result in the jump moving towards the bridge, would result in a reduction in the capacity at the bridge. A hydraulic instability can relate to "real world" effect, such as sediment or debris in the flow, or the variability in the hydraulic assumptions that makes it difficult to exactly determine the location of a hydraulic jump.

Figure B-6 shows the water surface profiles for Option 2-C using the pressure/weir bridge method. Under this scenario, a flow of 5,400 cfs can pass under the bridge and maintain supercritical flow. In the bridge pressure flow routine, the program is checking the potential of the flow to hit the low chord of the bridge; and for any flows greater than 5,400 cfs, the downstream subcritical flow in the concrete channel is above the low chord.

Conclusions

The construction of Option 2 (8'x6' CBC in conjunction with the partial or complete removal of the concrete sill) improves the hydraulic capacity of the bridge and the downstream channel to about 5,400 cfs. The construction of either pond (1.5 or 0.7 acres) has essentially no effect on the hydraulic capacity, which is to be expected as their purpose is for the temporary storage of sediment from summer thunderstorms.

Option 1 markedly reduces the depth of the subcritical flow (downstream of the occurrence of a hydraulic jump) to the point that up to the flow depth of 6,800 cfs the flow downstream is 1.4 feet below the low chord of the bridge. This indicates that even if hydraulic anomalies caused the hydraulic jump to migrate upstream closer to the bridge, a choke condition at the bridge would not occur at this flow rate.

The results for the HEC-RAS runs are summarized in the following table:

Table B-1

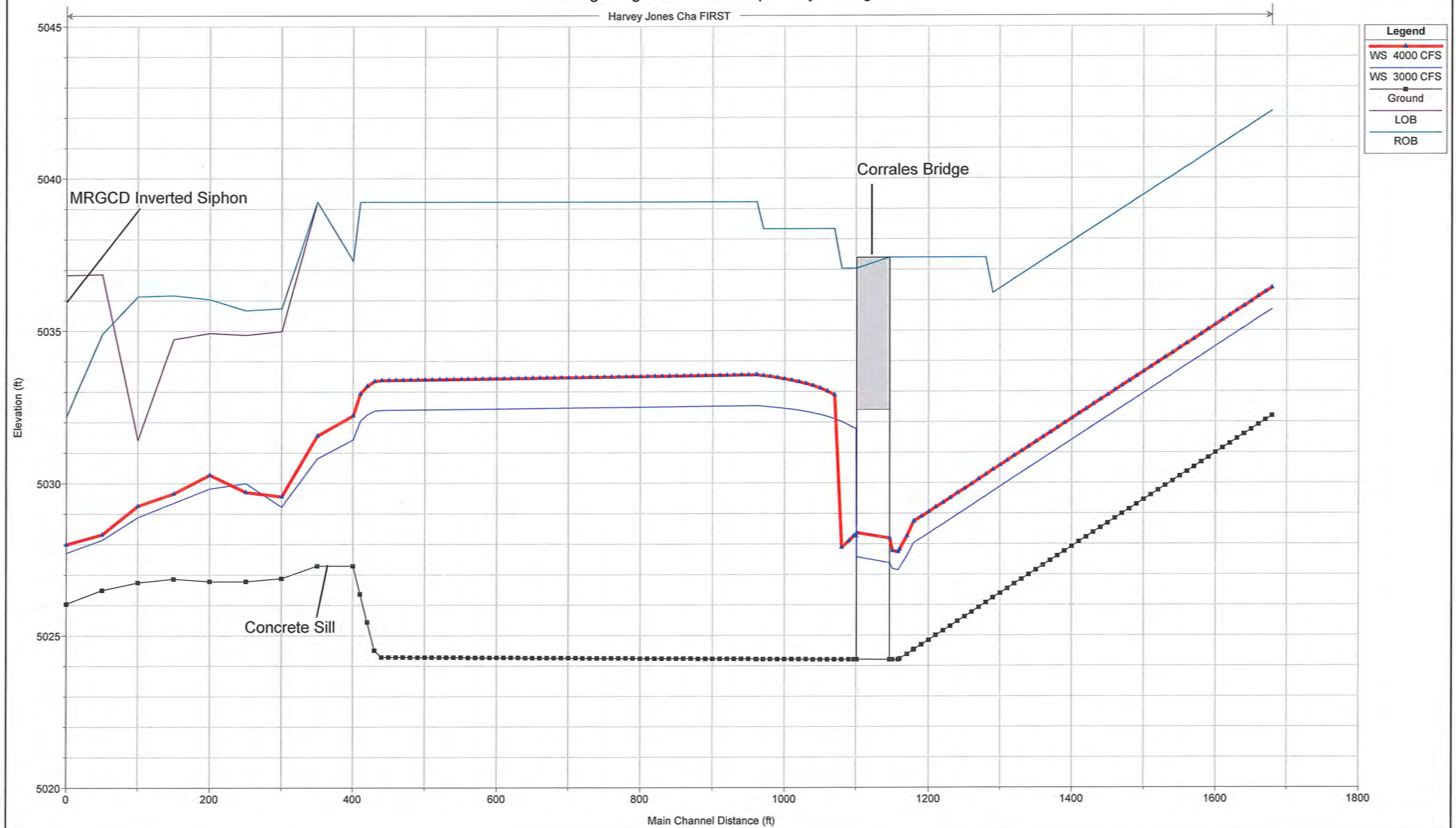
Scenario	Bridge Capacity	Flow Regime Upstream of Bridge	HEC-RAS High Flow Method	Sill Scenario	Pier Debris Width	Delta Low Chord vs. WS Downstream of Bridge	Outlet Scenario
	cfs				ft.	ft.	
Corrales Bridge Low Chord Elev. = 5032.4							
Existing Channel							
	4,000	Backwater	Pressure Flow	-	2	-1.2	Existing
	7,000	Supercritical	Standard Step	-	2	-3.9	Existing
Option 1 - Trapezoidal Channel							
Option 1-C	6,800	Supercritical	Standard Step	Trap thru Sill Floor	2	1.5	0.7 AC Pond
	6,200	Supercritical	Pressure Flow	Trap thru Sill Floor	2	1.8	0.7 AC Pond
	6,800	Supercritical	Pressure Flow	Trap thru Sill Floor	2	1.4	0.7 AC Pond
Option 2 - 8x6 Concrete Box Culvert							
Option 2-C	7,000	Supercritical	Standard Step	Trap thru Sill Floor	2	-1.2	0.7 AC Pond
	5,400	Supercritical	Pressure Flow	Trap thru Sill Floor	2	0.3	0.7 AC Pond

* See explanation of flow capacity for pressure flow condition in Option 1-C results discussion.

FIGURES

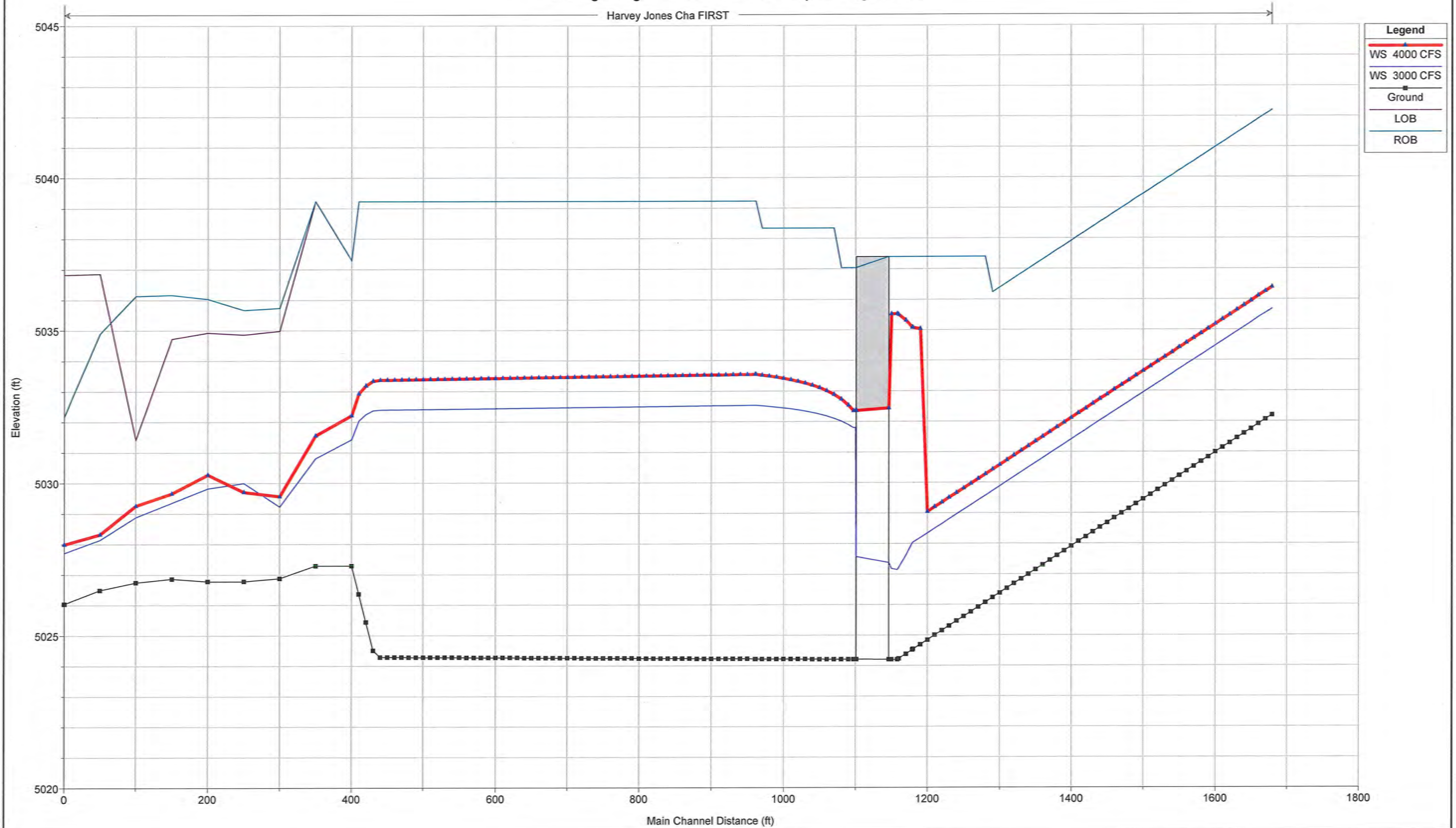
HJCExisitingtoSiphon Plan: Existing-Energy 2/4/2013
Existing Bridge - Standard Step Analysis - Figure B-1

Harvey Jones Cha FIRST

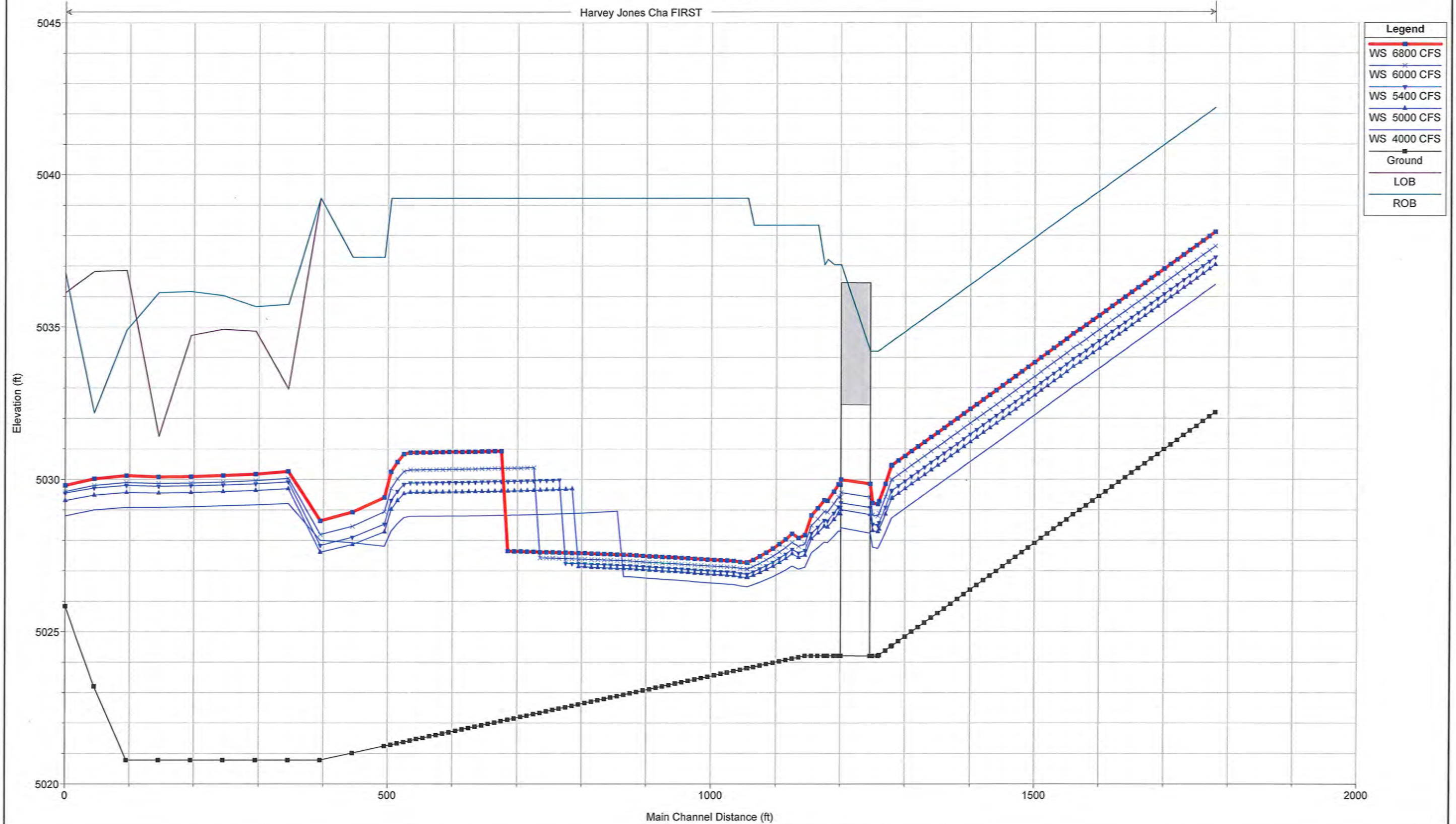


HJCExisitingtoSiphon Plan: Existing-Energy 2/4/2013
Existing Bridge - Pressure Flow Analysis - Figure B-2

Harvey Jones Cha FIRST

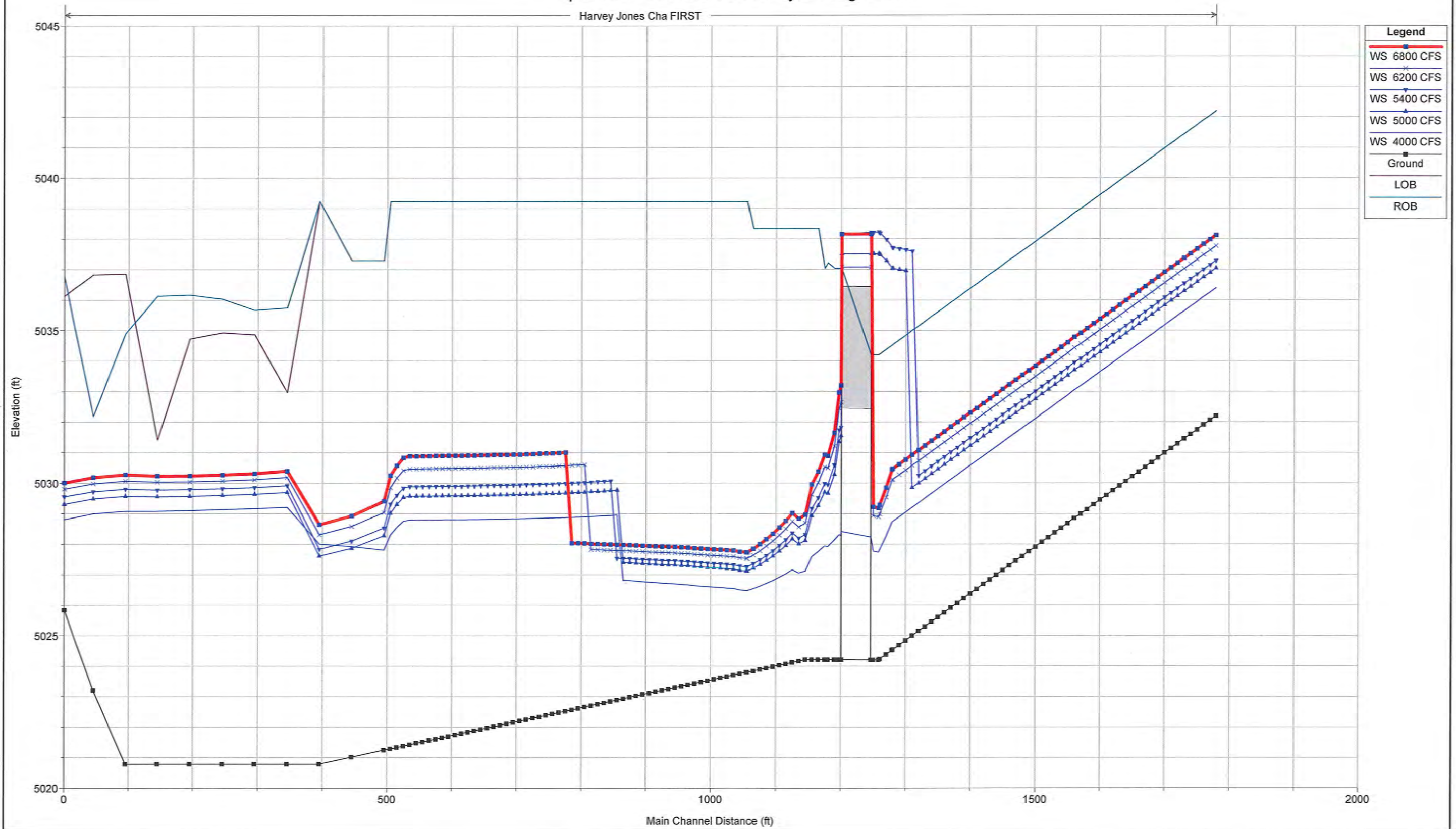


HJCPROPOSEDFIN Plan: Option1-C 2/3/2013
Option 1-C - Standard Step Analysis - Figure B-3



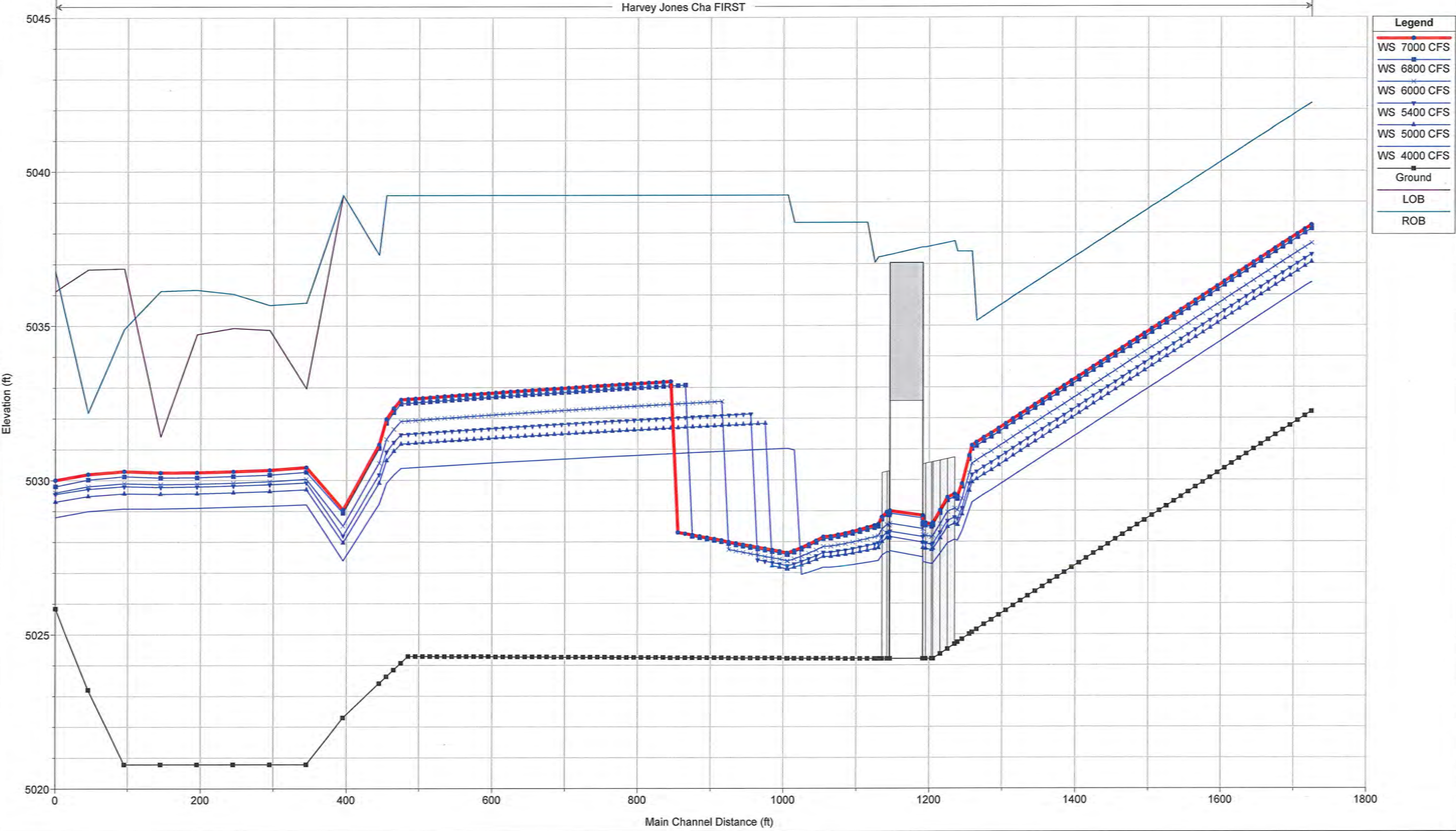
HJCPROPOSEDFIN Plan: Option1-C 2/3/2013
Option 1-C - Pressure Flow Analysis - Figure B-4

Harvey Jones Cha FIRST



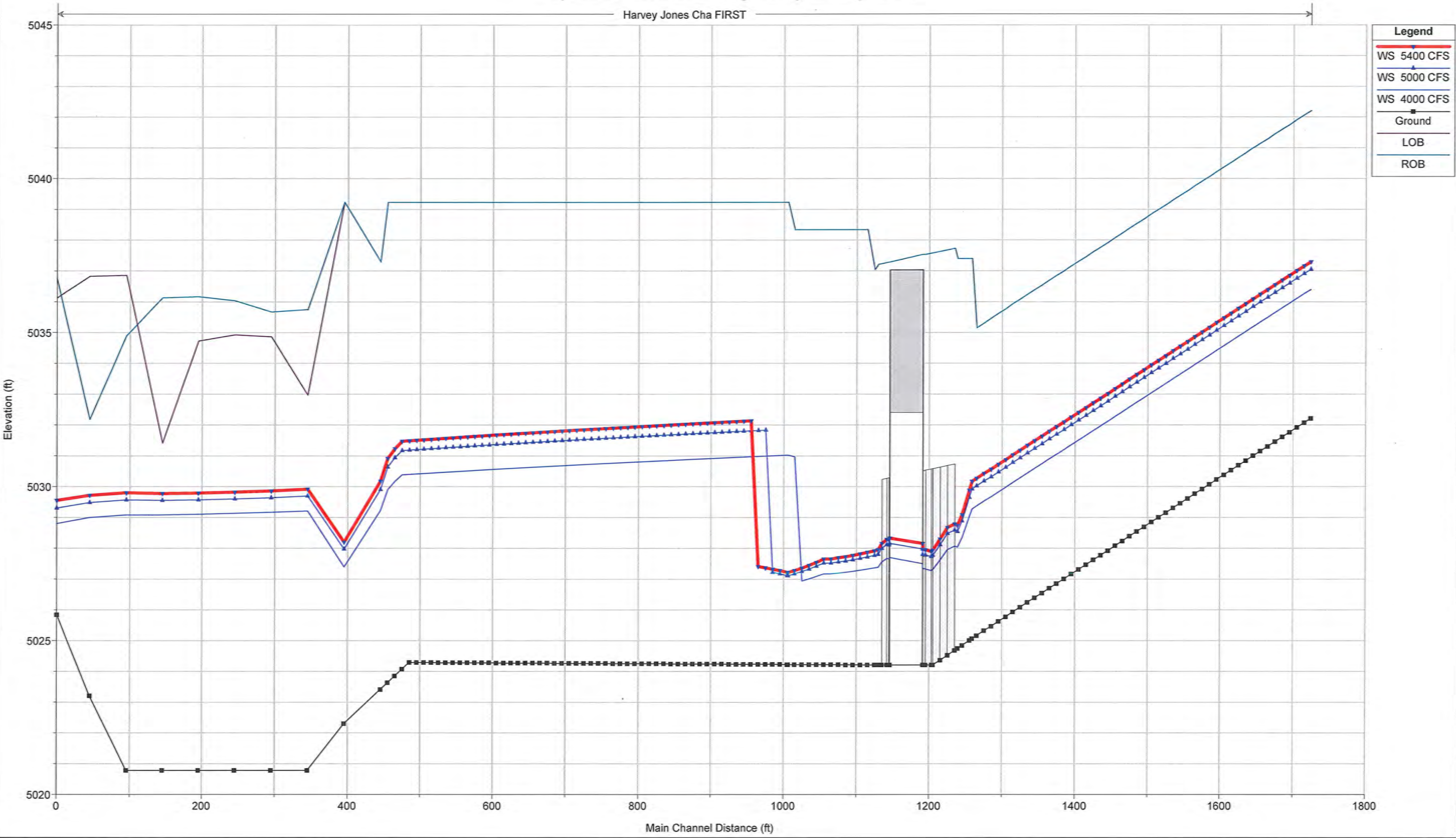
HJCPROPOSEDFIN Plan: Option2-C 2/3/2013
Option 2-C - Standard Step Analysis - Figure B-5

Harvey Jones Cha FIRST

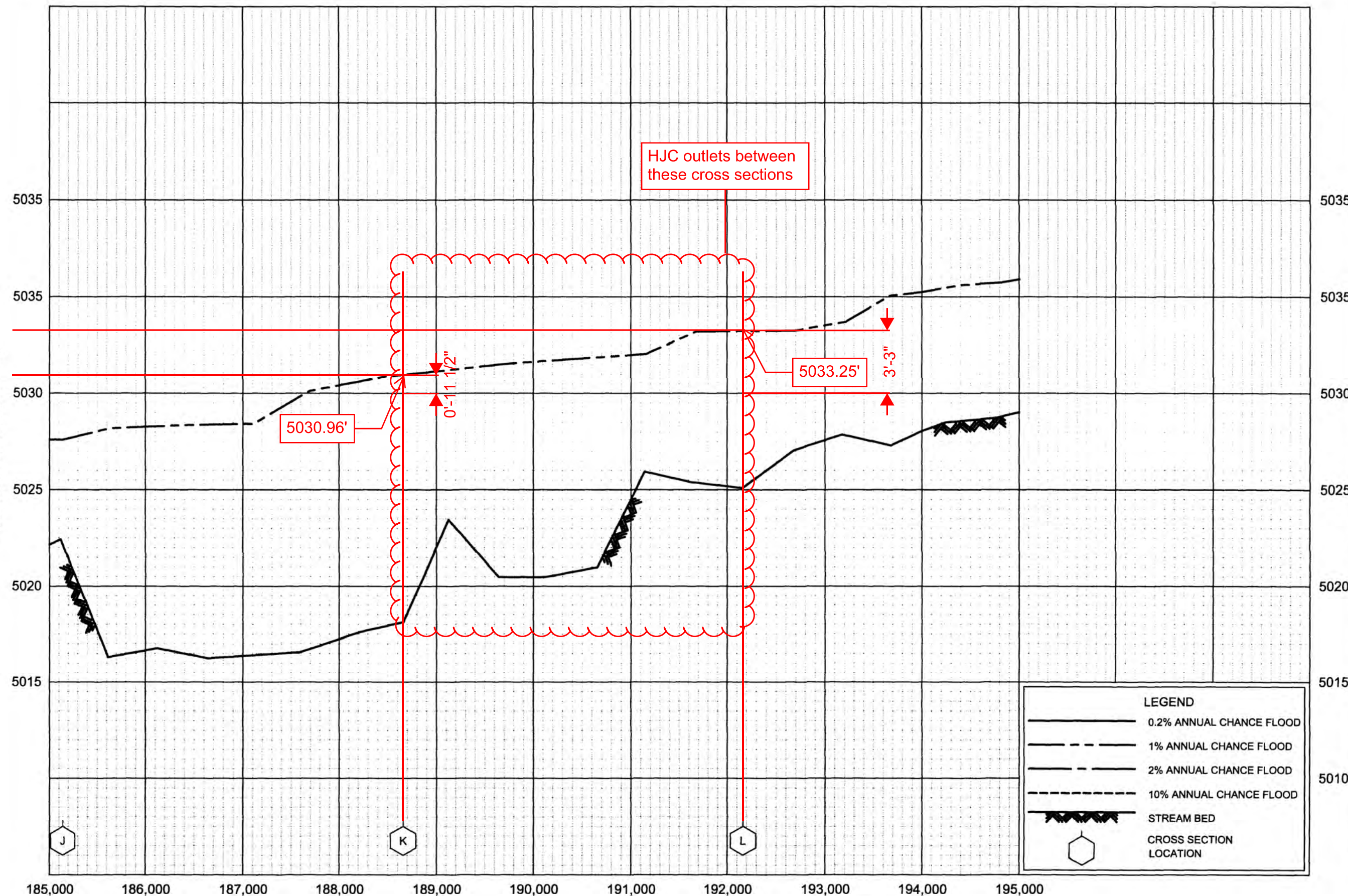


HJCPROPOSEDFIN Plan: Option2-C 2/3/2013
Option 2-C - Pressure Bridge Analysis - Figure B-6

Harvey Jones Cha FIRST



ELEVATION IN FEET (NAVD 88)



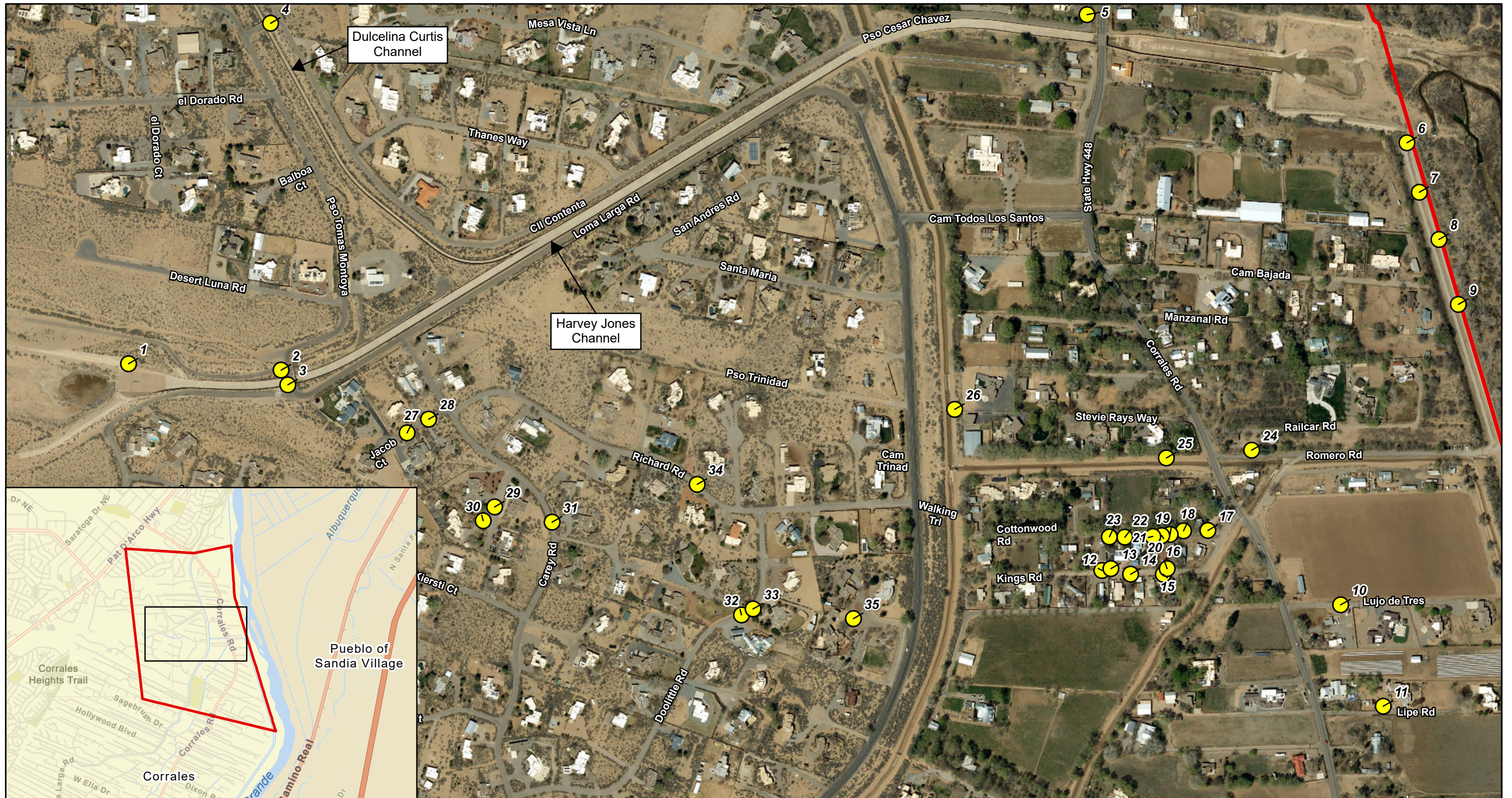
STREAM DISTANCE IN FEET ABOVE STATE HIGHWAY 49 BRIDGE AT LOS LUNAS

FEDERAL EMERGENCY MANAGEMENT AGENCY
SANDOVAL COUNTY, NM
AND INCORPORATED AREAS

FLOOD PROFILES

RIO GRANDE

APPENDIX B – FIELD RECONNAISSANCE



Bohannon & Huston
www.bhinc.com 800.877.5332

- Field Observation Point
- Model Domain



0 200 400 800 Feet
1 in. = 400 ft.

**Montoyas Arroyo
HJC - Hydraulic
Analysis Report**
*Appendix B
Field Reconnaissance*

Field Reconnaissance Observations

Observation Location	Description	Photo(s)
Location 1 Lat: 35° 15' 25.11" N Long: 106° 36' 55.47" W	Name: Photo 1 Feature Category: HJC Structure Description: HJC Inlet and downstream channel as seen from the northern top of inlet, looking east.	
Location 1 Lat: 35° 15' 25.11" N Long: 106° 36' 55.47" W	Name: Photo 2 Feature Category: HJC Structure Description: HJC Inlet and fencing as seen from the northern top of inlet, looking east.	

Location 1 Lat: 35° 15' 25.11" N Long: 106° 36' 55.47" W	Name: Photo 3 Feature Category: HJC Structure Description: HJC Inlet upstream basin as seen from the northern berm at the top of the concrete inlet, looking west.	 A photograph showing a dirt road in the foreground with a blue metal fence. In the background, there is a small body of water (the inlet) surrounded by green trees and shrubs under a clear blue sky.
Location 1 Lat: 35° 15' 25.11" N Long: 106° 36' 55.47" W	Name: Photo 4 Feature Category: HJC Structure Description: HJC Inlet upstream basin as seen from the northern access road to the inlet, looking west.	 A photograph showing a wide, flat, brownish area (likely a dry basin or access road) in the foreground. A blue metal fence runs along the edge. In the background, there is a small body of water (the inlet) surrounded by green trees and shrubs under a clear blue sky.

Location 1 Lat: 35° 15' 25.11" N Long: 106° 36' 55.47" W	Name: Photo 5 Feature Category: HJC Structure Description: HJC Inlet crest and drain from upstream basin as seen from the northern top of slope, looking southeast.	
Location 2 Lat: 35° 15' 24.87" N Long: 106° 36' 47.59" W	Name: Photo 6 Feature Category: HJC Structure Description: Loma Larga bridge upstream face as seen from the top of the northern embankment, looking south.	

Location 2 Lat: 35° 15' 24.87" N Long: 106° 36' 47.59" W	Name: Photo 7 Feature Category: HJC Structure Description: Channel upstream of Loma Larga bridge as seen standing near the north channel wall upstream of the structure, looking west.	
Location 2 Lat: 35° 15' 24.87" N Long: 106° 36' 47.59" W	Name: Photo 8 Feature Category: HJC Structure Description: Inside Loma Larga bridge upstream face as seen from the top of the northern channel wall, looking south.	

Location 3 Lat: 35° 15' 24.25" N Long: 106° 36' 47.24" W	Name: Photo 9 Feature Category: HJC Structure Description: The southern embankment adjacent to Harvey Jones Channel as seen from the southern upstream edge of the Loma Larga bridge, looking south.	
Location 3 Lat: 35° 15' 24.25" N Long: 106° 36' 47.24" W	Name: Photo 10 Feature Category: HJC Structure Description: Harvey Jones Channel upstream of Loma Larga bridge as seen on Loma Larga bridge looking west,	

Location 3 Lat: 35° 15' 24.25" N Long: 106° 36' 47.24" W	Name: Photo 11 Feature Category: HJC Structure Description: Southern Harvey Jones Channel wall upstream of Loma Larga bridge as seen on Loma Larga bridge looking southwest.	
Location 4 Lat: 35° 15' 39.54" N Long: 106° 36' 48.2" W	Name: Photo 12 Feature Category: HJC Structure Description: Dulcelina Curtis Channel walls at the Vasa Wy crossing as seen from the Vasa Wy crossing, looking southwest.	

Location 4 Lat: 35° 15' 39.54" N Long: 106° 36' 48.2" W	Name: Photo 13 Feature Category: HJC Structure Description: Dulcelina Curtis Channel walls and adjacent eastern ground at the Vasa Wy crossing as seen from the Vasa Wy crossing, looking southwest.	
Location 4 Lat: 35° 15' 39.54" N Long: 106° 36' 48.2" W	Name: Photo 14 Feature Category: HJC Structure Description: Dulcelina Curtis Channel at the Vasa Wy crossing as seen from the Vasa Wy crossing, looking southeast.	

Location 5 Lat: 35° 15' 40.03" N Long: 106° 36' 6.08" W	Name: Photo 15 Feature Category: HJC Structure Description: Harvey Jones Channel northern wall, road crossing at Corrales crossing, and adjacent ground as seen from the corner of Calle Contenta adjacent to the crossing, looking southeast.	
Location 5 Lat: 35° 15' 40.03" N Long: 106° 36' 6.08" W	Name: Photo 16 Feature Category: HJC Structure Description: Harvey Jones Channel northern wall at Corrales crossing and adjacent ground as seen from the corner of Calle Contenta adjacent to the crossing, looking southeast.	

Location 5 Lat: 35° 15' 40.03" N Long: 106° 36' 6.08" W	Name: Photo 17 Feature Category: HJC Structure Description: Corrales Rd north of Harvey Jones Channel crossing at and adjacent ground as seen from the corner of Calle Contenta adjacent to the crossing, looking west.	
Location 5 Lat: 35° 15' 40.03" N Long: 106° 36' 6.08" W	Name: Photo 18 Feature Category: HJC Structure Description: Harvey Jones Channel northern wall at Corrales crossing and adjacent ground as seen from the corner of Calle Contenta adjacent to the crossing, looking southwest.	

Location 5 Lat: 35° 15' 40.03" N Long: 106° 36' 6.08" W	Name: Photo 19 Feature Category: Building FFE Adjustment Description: Home in the corner lot adjacent to the Calle Contenta corner and Corrales Rd crossing of Harvey Jones Channel. Address: 111 Autumn Ln Corrales, NM 87048	
Location 5 Lat: 35° 15' 40.03" N Long: 106° 36' 6.08" W	Name: Photo 20 Feature Category: HJC Structure Description: Corrales Rd crossing of Harvey Jones Channel upstream face as seen from northern channel wall, looking southeast.	

Location 5 Lat: 35° 15' 40.03" N Long: 106° 36' 6.08" W	Name: Photo 21 Feature Category: HJC Structure Description: Corrales Rd crossing of Harvey Jones Channel upstream face and upstream channel as seen from northern channel wall, looking southeast.	
Location 6 Lat: 35° 15' 34.66" N Long: 106° 35' 49.55" W	Name: Photo 22 Feature Category: Rio Grande Levee Description: Property adjacent to the Rio Grande levee directly south of the Harvey Jones Channel outlet, as seen from the lower road (nearest to the Upper Corrales Riverside Drain), looking southwest.	

Location 6 Lat: 35° 15' 34.66" N Long: 106° 35' 49.55" W	Name: Photo 23 Feature Category: Rio Grande Levee Description: Property adjacent to the Rio Grande levee directly south of the Harvey Jones Channel outlet, as seen from the lower road (nearest to the Upper Corrales Riverside Drain), looking west.	
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Location 7 Lat: 35° 15' 32.57" N Long: 106° 35' 48.9" W	Name: Photo 24 Feature Category: Building FFE Adjustment Description: Property adjacent to the Rio Grande levee approx. 300 ft south of the Harvey Jones Channel outlet, upper road (nearest to the Rio Grande), looking west. Address: 7200 Corrales Rd Corrales, NM 87048	
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Location 8 Lat: 35° 15' 30.57" N Long: 106° 35' 47.88" W	Name: Photo 25 Feature Category: Building FFE Adjustment Description: Property adjacent to the Rio Grande levee approx. 550 ft south of the Harvey Jones Channel outlet, upper road (nearest to the Rio Grande), looking west. Address: 213 Camino Bajada Corrales, NM 87048	
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Location 8 Lat: 35° 15' 30.57" N Long: 106° 35' 47.88" W	Name: Photo 26 Feature Category: Building FFE Adjustment Description: Property adjacent to the Rio Grande levee approx. 550 ft south of the Harvey Jones Channel outlet, upper road (nearest to the Rio Grande), looking west. Address: 213 Camino Bajada Corrales, NM 87048	
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Location 9 Lat: 35° 15' 27.83" N Long: 106° 35' 46.87" W	Name: Photo 27 Feature Category: Building FFE Adjustment Description: Property adjacent to the Rio Grande levee approx. 900 ft south of the Harvey Jones Channel outlet, upper road (nearest to the Rio Grande), looking west. Address: 250 Camino Bajada Corrales, NM 87048	
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Location 9 Lat: 35° 15' 27.83" N Long: 106° 35' 46.87" W	Name: Photo 28 Feature Category: Building FFE Adjustment Description: Property adjacent to the Rio Grande levee approx. 900 ft south of the Harvey Jones Channel outlet, upper road (nearest to the Rio Grande), looking west. Address: 250 Camino Bajada Corrales, NM 87048	
Location 10 Lat: 35° 15' 15.1" N Long: 106° 35' 52.89" W	Name: Photo 29 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from Lujo De Tres, looking south. Address: 101 Lujo de Tres Corrales, NM 87048	

Location 11 Lat: 35° 15' 10.82" N Long: 106° 35' 50.67" W	Name: Photo 30 Feature Category: Building FFE Adjustment Description: One (1) step FFE adjustment necessary as seen from Lipe Rd, looking north. Address: 6770 Corrales Rd Corrales, NM 87048	
Location 12 Lat: 35° 15' 16.52" N Long: 106° 36' 5.21" W	Name: Photo 31 Feature Category: Building FFE Adjustment Description: Four (4) steps FFE adjustment necessary as seen from Kings Ln, looking north. Address: 90 Cottonwood Ln Corrales, NM 87048	

Location 13 Lat: 35° 15' 16.6" N Long: 106° 36' 4.75" W	Name: Photo 32 Feature Category: Building FFE Adjustment Description: Four (4) steps FFE adjustment necessary as seen from Kings Ln, looking north. Address: 84 Cottonwood Ln Corrales, NM 87048	
Location 14 Lat: 35° 15' 16.37" N Long: 106° 36' 3.73" W	Name: Photo 33 Feature Category: Building FFE Adjustment Description: Four (4) steps FFE adjustment necessary as seen from Kings Ln, looking north. Address: 64 Cottonwood Ln Corrales, NM 87048	

Location 15 Lat: 35° 15' 16.37" N Long: 106° 36' 2.04" W	Name: Photo 34 Feature Category: Building FFE Adjustment Description: Five (5) steps FFE adjustment necessary as seen from Kings Ln, looking south. Address: 86 Kings Ln Corrales, NM 87048	
Location 16 Lat: 35° 15' 16.61" N Long: 106° 36' 1.84" W	Name: Photo 35 Feature Category: Building FFE Adjustment Description: Four (4) steps FFE adjustment necessary as seen from Cottonwood Ln, looking north. Address: 36 Cottonwood Ln Corrales, NM 87048	

Location 16 Lat: 35° 15' 16.61" N Long: 106° 36' 1.84" W	Name: Photo 36 Feature Category: Building FFE Adjustment Description: Ramp FFE adjustment necessary as seen from Cottonwood Ln, looking north. Address: 9 Cottonwood Ln Corrales, NM 87048	
Location 17 Lat: 35° 15' 18.23" N Long: 106° 35' 59.75" W	Name: Photo 37 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from Cottonwood Ln, looking north. Address: 9 Cottonwood Ln Corrales, NM 87048	

Location 18 Lat: 35° 15' 18.21" N Long: 106° 36' 1.01" W	Name: Photo 38 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from Cottonwood Ln, looking north. Address: 35 Cottonwood Ln Corrales, NM 87048	
Location 19 Lat: 35° 15' 18.06" N Long: 106° 36' 1.7" W	Name: Photo 39 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from Cottonwood Ln, looking north. Address: 35 Cottonwood Ln Corrales, NM 87048	

Location 20 Lat: 35° 15' 18.0" N Long: 106° 36' 2.13" W	Name: Photo 40 Feature Category: Building FFE Adjustment Description: Four (4) steps FFE adjustment necessary as seen from Cottonwood Ln, looking south. Address: 36 Cottonwood Ln Corrales, NM 87048	
Location 21 Lat: 35° 15' 17.97" N Long: 106° 36' 2.59" W	Name: Photo 41 Feature Category: Building FFE Adjustment Description: Four (4) steps FFE adjustment necessary as seen from Cottonwood Ln, looking south. Address: 64 Cottonwood Ln Corrales, NM 87048	

Location 22 Lat: 35° 15' 17.93" N Long: 106° 36' 4.04" W	Name: Photo 42 Feature Category: Building FFE Adjustment Description: Four (4) steps FFE adjustment necessary as seen from Cottonwood Ln, looking south. Address: 84 Cottonwood Ln Corrales, NM 87048	
Location 23 Lat: 35° 15' 17.94" N Long: 106° 36' 4.83" W	Name: Photo 43 Feature Category: Building FFE Adjustment Description: Four (4) steps FFE adjustment necessary as seen from Cottonwood Ln, looking south. Address: 90 Cottonwood Ln Corrales, NM 87048	

Location 24 Lat: 35° 15' 21.63" N Long: 106° 35' 57.51" W	Name: Photo 44 Feature Category: Building FFE Adjustment Description: Spoke to the owner. She has been here for 11 years and said part of the house is three steps down and part has a crawl space. She has experienced some heavy rains where her parking area flooded but the home has not flooded. Address: 6964 Corrales Rd Corrales, NM 87048	 A photograph of a single-story adobe house with a blue door, surrounded by desert landscaping and trees. The house is built with reddish-brown adobe walls. A blue door is visible on the right side of the house. The foreground is a gravel driveway. There are several trees and shrubs around the house, including a large tree on the right. The sky is blue with some clouds.
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Location 25 Lat: 35° 15' 21.29" N Long: 106° 36' 1.91" W	Name: Photo 45 Feature Category: Building FFE Adjustment Description: Two (2) steps necessary as seen from northern Sandoval Lateral overbank, looking north. Address: 1 Stevie Ray's Way Corrales, NM 87048	
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Location 26 Lat: 35° 15' 23.31" N Long: 106° 36' 12.84" W	Name: Photo 46 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from eastern Corrales Main Canal overbank, looking northeast. Address: 5 Stevie Ray's Way Corrales, NM 87048	
Location 27 Lat: 35° 15' 22.24" N Long: 106° 36' 41.08" W	Name: Photo 47 Feature Category: Building FFE Adjustment Description: One (1) step FFE adjustment necessary as seen from Jacob Ct, looking southeast. Address: 102 Jacob Ct Corrales, NM 87048	

Location 28 Lat: 35° 15' 22.82" N Long: 106° 36' 39.98" W	Name: Photo 48 Feature Category: Building FFE Adjustment Description: One (1) step FFE adjustment necessary as seen from Jacob Ct, looking southeast. Address: 100 Jacob Ct Corrales, NM 87048	
Location 29 Lat: 35° 15' 19.12" N Long: 106° 36' 36.55" W	Name: Photo 49 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from Carey Rd, looking northwest. Address: 133 Carey Rd Corrales, NM 87048	

Location 30 Lat: 35° 15' 18.53" N Long: 106° 36' 37.14" W	Name: Photo 50 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from Carey Rd, looking east. Address: 131 Carey Rd Corrales, NM 87048	
Location 30 Lat: 35° 15' 18.53" N Long: 106° 36' 37.14" W	Name: Photo 51 Feature Category: Building FFE Adjustment Description: One (1) step FFE adjustment necessary as seen from Carey Rd, looking southeast. Address: 129 Carey Rd Corrales, NM 87048	

Location 31 Lat: 35° 15' 18.48" N Long: 106° 36' 33.58" W	Name: Photo 52 Feature Category: Building FFE Adjustment Description: One (1) step FFE adjustment necessary as seen from Carey Rd, looking east. Address: 136 Carey Rd Corrales, NM 87048	
Location 32 Lat: 35° 15' 14.59" N Long: 106° 36' 23.8" W	Name: Photo 53 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from Doolittle Rd, looking southeast. Address: 120 Doolittle Rd Corrales, NM 87048	

Location 33 Lat: 35° 15' 14.84" N Long: 106° 36' 23.2" W	Name: Photo 54 Feature Category: Building FFE Adjustment Description: Two (2) steps FFE adjustment necessary as seen from Doolittle Rd, looking east. Address: 122 Doolittle Rd Corrales, NM 87048	
Location 34 Lat: 35° 15' 20.1" N Long: 106° 36' 26.11" W	Name: Photo 55 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from Richard Rd, looking south. Address: 120 Richard Rd Corrales, NM 87048	

Location 35 Lat: 35° 15' 14.45" N Long: 106° 36' 18.02" W	Name: Photo 56 Feature Category: Building FFE Adjustment Description: No FFE adjustment necessary as seen from Loma Larga Rd, looking southwest. Address: 8018 Loma Larga Corrales, NM 87048	 A photograph showing a white SUV parked on a gravel driveway. A large, leafy tree is on the left, casting shadows over the car and the driveway. In the background, a single-story house with a light-colored exterior and a dark roof is visible. The house has a garage on the right side. The overall scene is a residential property.
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APPENDIX C – NOAA DATA



NOAA Atlas 14, Volume 1, Version 5
Location name: Rio Rancho, New Mexico, USA*
Latitude: 35.3126°, Longitude: -106.7128°
Elevation: 5639 ft**
* source: ESRI Maps
** source: USGS



POINT PRECIPITATION FREQUENCY ESTIMATES

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

NOAA, National Weather Service, Silver Spring, Maryland

[PF_tabular](#) | [PF_graphical](#) | [Maps_&_aerials](#)

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches) ¹										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.179 (0.154-0.208)	0.232 (0.200-0.269)	0.311 (0.267-0.362)	0.372 (0.318-0.431)	0.456 (0.388-0.528)	0.521 (0.441-0.604)	0.589 (0.495-0.681)	0.660 (0.552-0.764)	0.758 (0.627-0.877)	0.835 (0.686-0.967)
10-min	0.273 (0.234-0.317)	0.353 (0.304-0.410)	0.474 (0.406-0.551)	0.566 (0.484-0.657)	0.694 (0.590-0.804)	0.792 (0.671-0.918)	0.895 (0.753-1.04)	1.00 (0.840-1.16)	1.15 (0.954-1.34)	1.27 (1.04-1.47)
15-min	0.338 (0.290-0.393)	0.438 (0.376-0.508)	0.588 (0.504-0.683)	0.702 (0.600-0.814)	0.860 (0.731-0.997)	0.982 (0.831-1.14)	1.11 (0.934-1.29)	1.25 (1.04-1.44)	1.43 (1.18-1.66)	1.58 (1.30-1.82)
30-min	0.455 (0.391-0.529)	0.589 (0.506-0.684)	0.791 (0.678-0.920)	0.945 (0.808-1.10)	1.16 (0.985-1.34)	1.32 (1.12-1.53)	1.50 (1.26-1.73)	1.68 (1.40-1.94)	1.92 (1.59-2.23)	2.12 (1.74-2.46)
60-min	0.563 (0.484-0.654)	0.729 (0.627-0.847)	0.979 (0.839-1.14)	1.17 (1.00-1.36)	1.43 (1.22-1.66)	1.64 (1.39-1.90)	1.85 (1.56-2.14)	2.08 (1.74-2.40)	2.38 (1.97-2.76)	2.62 (2.16-3.04)
2-hr	0.655 (0.558-0.778)	0.840 (0.714-1.00)	1.11 (0.942-1.32)	1.33 (1.12-1.57)	1.63 (1.37-1.92)	1.87 (1.56-2.20)	2.12 (1.76-2.50)	2.39 (1.97-2.81)	2.77 (2.25-3.25)	3.07 (2.47-3.61)
3-hr	0.704 (0.604-0.832)	0.894 (0.767-1.06)	1.17 (1.00-1.38)	1.39 (1.19-1.64)	1.70 (1.44-1.99)	1.94 (1.64-2.28)	2.20 (1.84-2.58)	2.48 (2.06-2.90)	2.86 (2.35-3.34)	3.17 (2.58-3.72)
6-hr	0.813 (0.705-0.951)	1.03 (0.892-1.20)	1.32 (1.15-1.54)	1.55 (1.34-1.81)	1.87 (1.61-2.18)	2.12 (1.82-2.46)	2.39 (2.03-2.77)	2.66 (2.24-3.08)	3.03 (2.54-3.52)	3.34 (2.77-3.87)
12-hr	0.913 (0.804-1.04)	1.15 (1.01-1.32)	1.46 (1.28-1.67)	1.70 (1.49-1.94)	2.03 (1.77-2.31)	2.29 (1.98-2.60)	2.55 (2.20-2.90)	2.83 (2.42-3.21)	3.19 (2.70-3.63)	3.49 (2.93-3.98)
24-hr	1.07 (0.947-1.21)	1.34 (1.19-1.52)	1.68 (1.49-1.91)	1.95 (1.73-2.21)	2.32 (2.05-2.62)	2.60 (2.29-2.94)	2.90 (2.54-3.27)	3.20 (2.79-3.61)	3.61 (3.12-4.07)	3.93 (3.38-4.44)
2-day	1.14 (1.01-1.28)	1.43 (1.28-1.62)	1.80 (1.60-2.03)	2.09 (1.86-2.35)	2.49 (2.20-2.80)	2.80 (2.46-3.14)	3.12 (2.73-3.50)	3.44 (3.01-3.87)	3.89 (3.37-4.37)	4.23 (3.65-4.77)
3-day	1.27 (1.15-1.41)	1.59 (1.44-1.76)	1.97 (1.79-2.18)	2.28 (2.06-2.52)	2.69 (2.43-2.97)	3.01 (2.70-3.32)	3.34 (2.99-3.68)	3.66 (3.27-4.04)	4.11 (3.64-4.54)	4.45 (3.92-4.93)
4-day	1.40 (1.29-1.53)	1.74 (1.60-1.90)	2.15 (1.97-2.34)	2.46 (2.26-2.68)	2.89 (2.65-3.15)	3.22 (2.94-3.50)	3.55 (3.24-3.86)	3.89 (3.53-4.22)	4.33 (3.91-4.71)	4.66 (4.20-5.08)

7-day	1.62 (1.49-1.76)	2.01 (1.85-2.18)	2.46 (2.27-2.67)	2.81 (2.59-3.04)	3.27 (3.01-3.54)	3.62 (3.33-3.91)	3.97 (3.64-4.29)	4.31 (3.94-4.66)	4.75 (4.33-5.14)	5.08 (4.61-5.51)
10-day	1.78 (1.65-1.93)	2.22 (2.05-2.40)	2.72 (2.52-2.94)	3.12 (2.88-3.37)	3.65 (3.36-3.93)	4.05 (3.71-4.36)	4.45 (4.07-4.78)	4.84 (4.42-5.21)	5.36 (4.87-5.78)	5.75 (5.20-6.20)
20-day	2.28 (2.10-2.47)	2.84 (2.62-3.07)	3.45 (3.19-3.73)	3.92 (3.61-4.23)	4.51 (4.16-4.87)	4.95 (4.55-5.33)	5.37 (4.93-5.78)	5.77 (5.29-6.21)	6.28 (5.74-6.76)	6.64 (6.05-7.17)
30-day	2.76 (2.55-2.97)	3.42 (3.17-3.69)	4.13 (3.82-4.45)	4.66 (4.30-5.01)	5.32 (4.91-5.71)	5.79 (5.34-6.22)	6.24 (5.74-6.70)	6.67 (6.12-7.16)	7.19 (6.58-7.72)	7.56 (6.91-8.12)
45-day	3.39 (3.14-3.65)	4.20 (3.90-4.52)	5.02 (4.65-5.40)	5.61 (5.19-6.03)	6.33 (5.86-6.80)	6.83 (6.32-7.33)	7.29 (6.74-7.82)	7.71 (7.11-8.27)	8.20 (7.56-8.80)	8.53 (7.85-9.15)
60-day	3.90 (3.62-4.20)	4.84 (4.49-5.21)	5.79 (5.37-6.22)	6.46 (6.00-6.94)	7.28 (6.76-7.82)	7.85 (7.28-8.43)	8.38 (7.76-9.00)	8.85 (8.20-9.52)	9.41 (8.71-10.1)	9.78 (9.05-10.5)

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

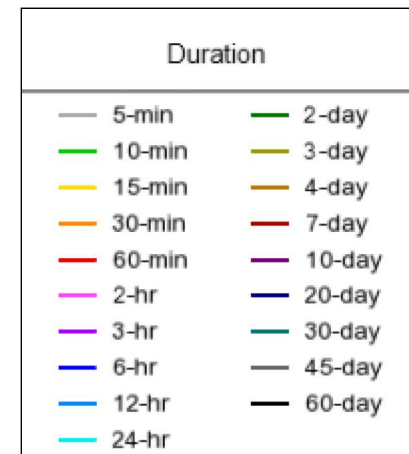
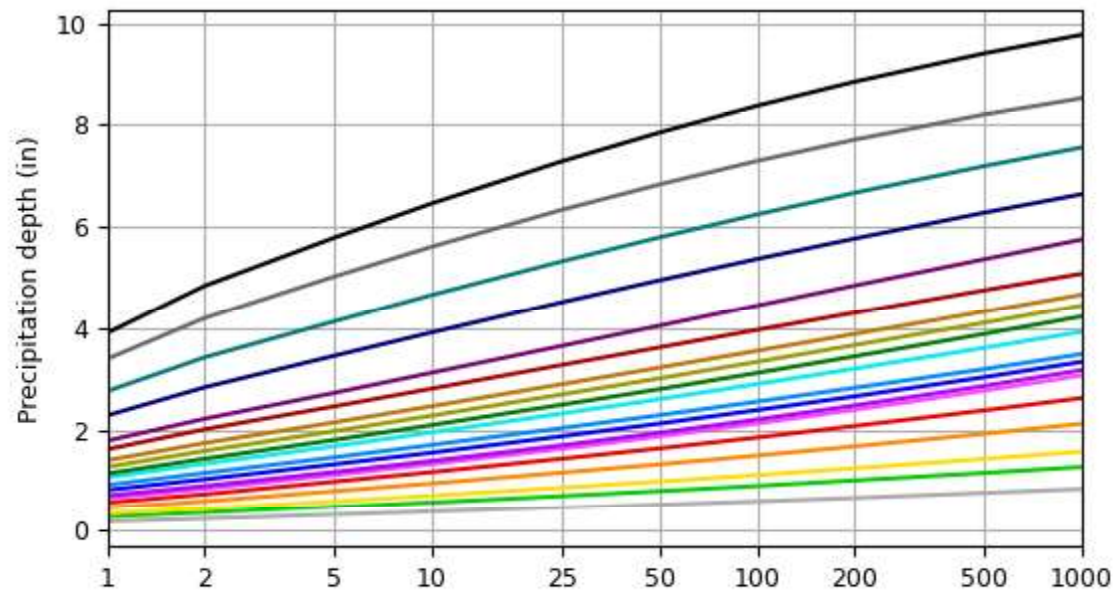
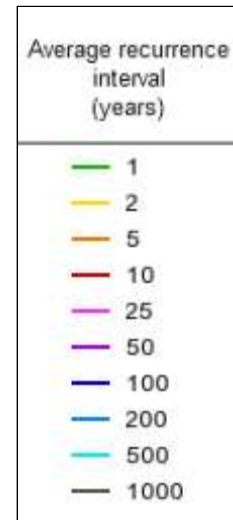
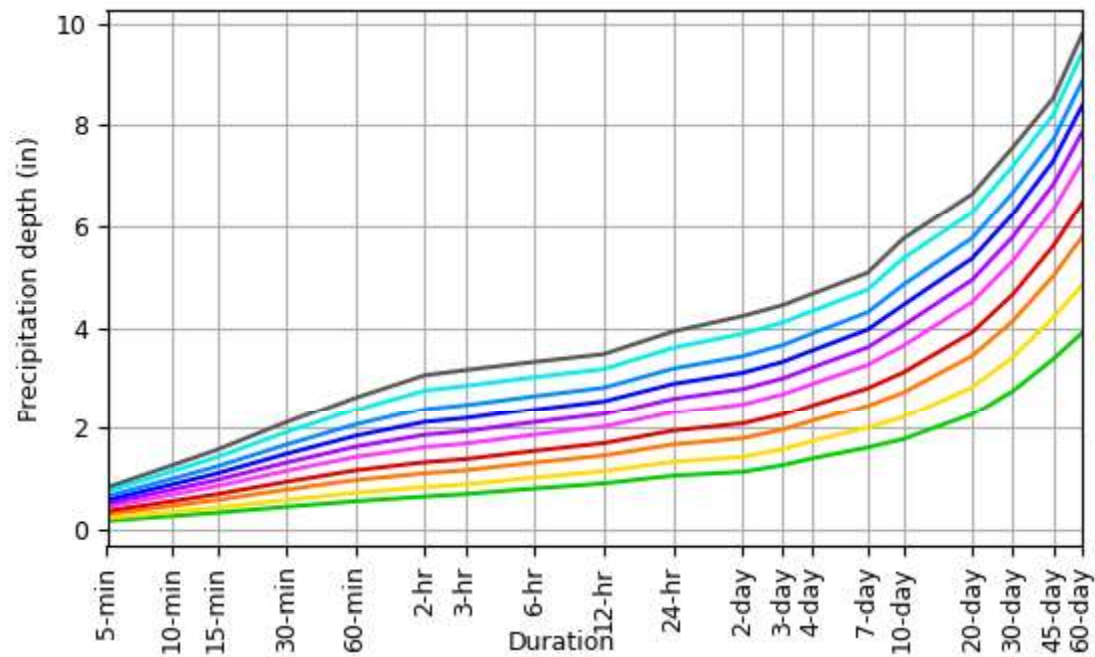
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper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

Please refer to NOAA Atlas 14 document for more information.

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PF graphical

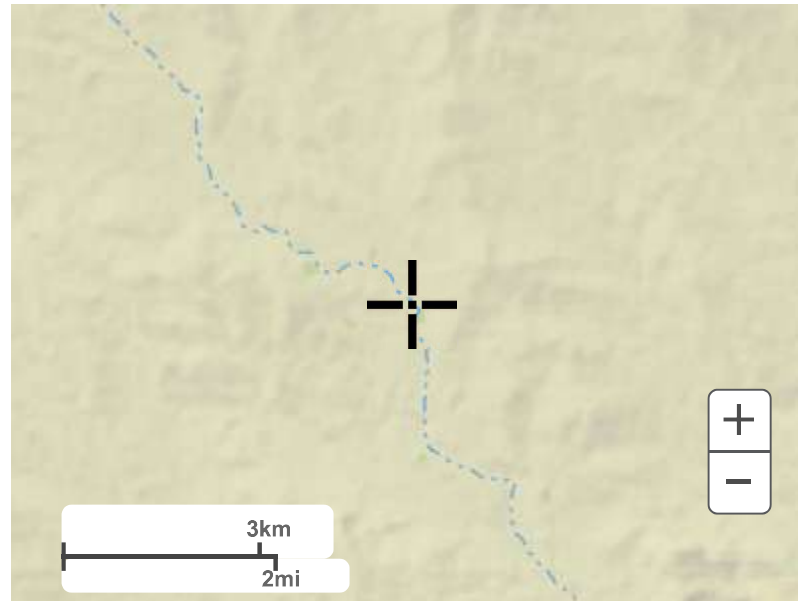
PDS-based depth-duration-frequency (DDF) curves
 Latitude: 35.3126°, Longitude: -106.7128°



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Maps & aerals

Small scale terrain



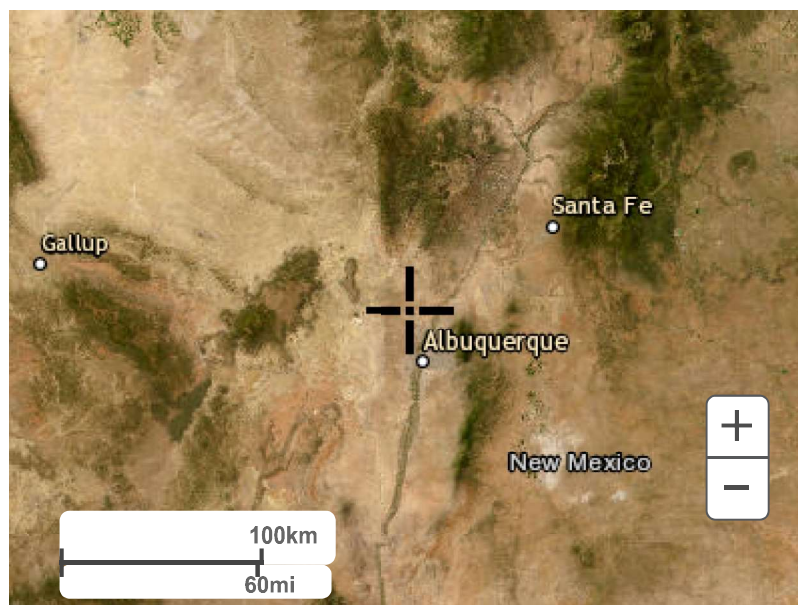
Large scale terrain



Large scale map



Large scale aerial



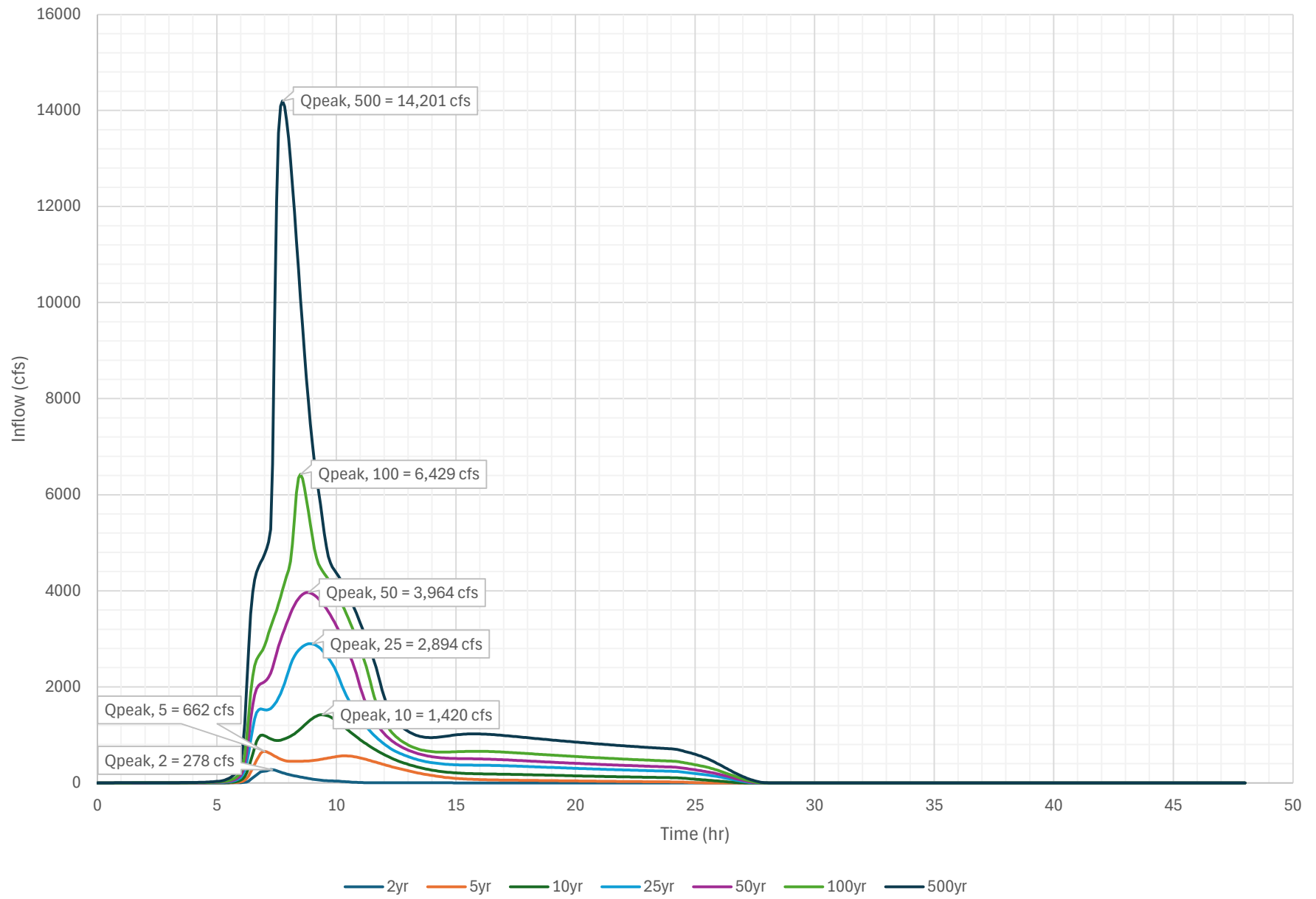
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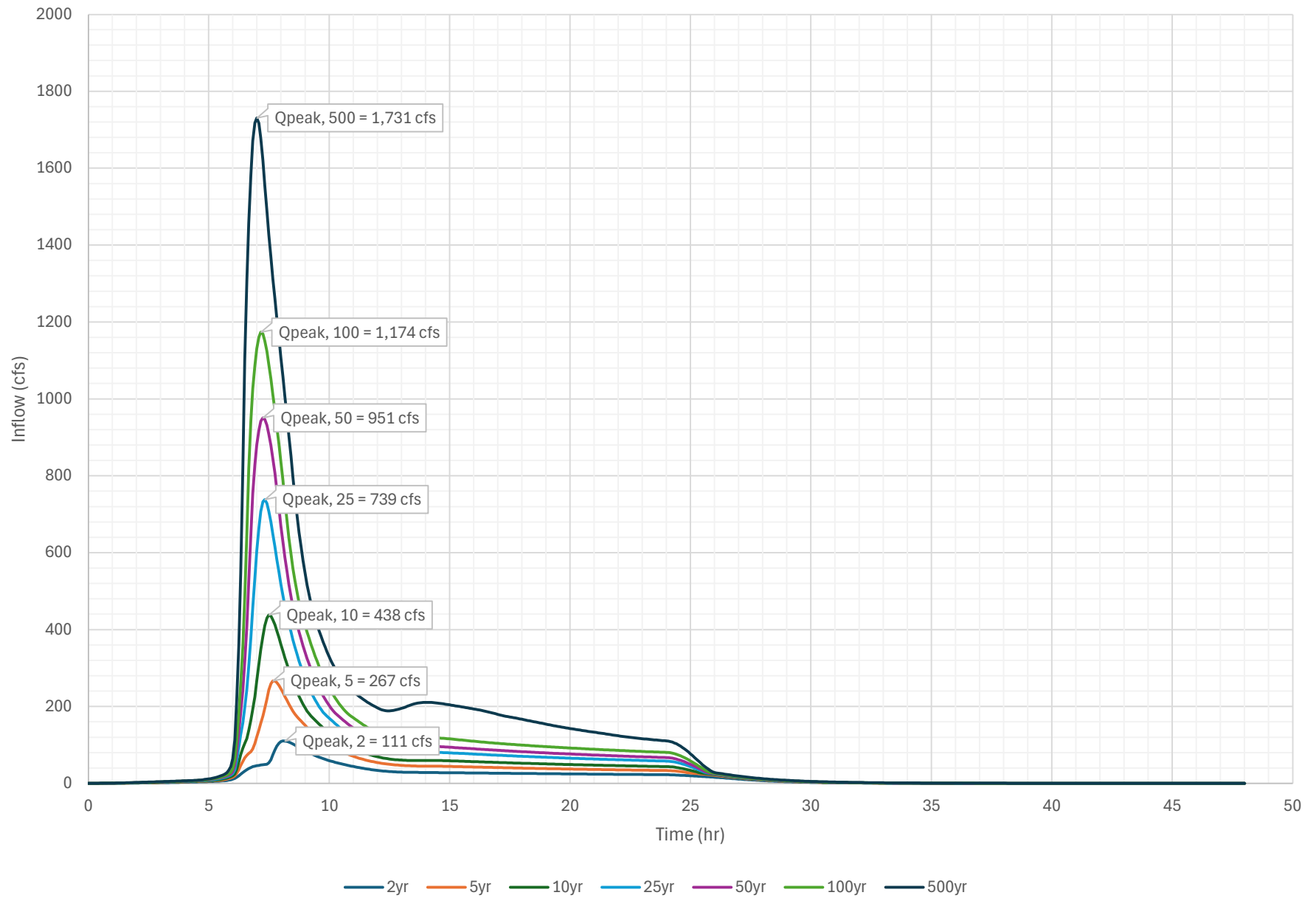
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APPENDIX D – HEC-HMS INFLOW HYDROGRAPHS

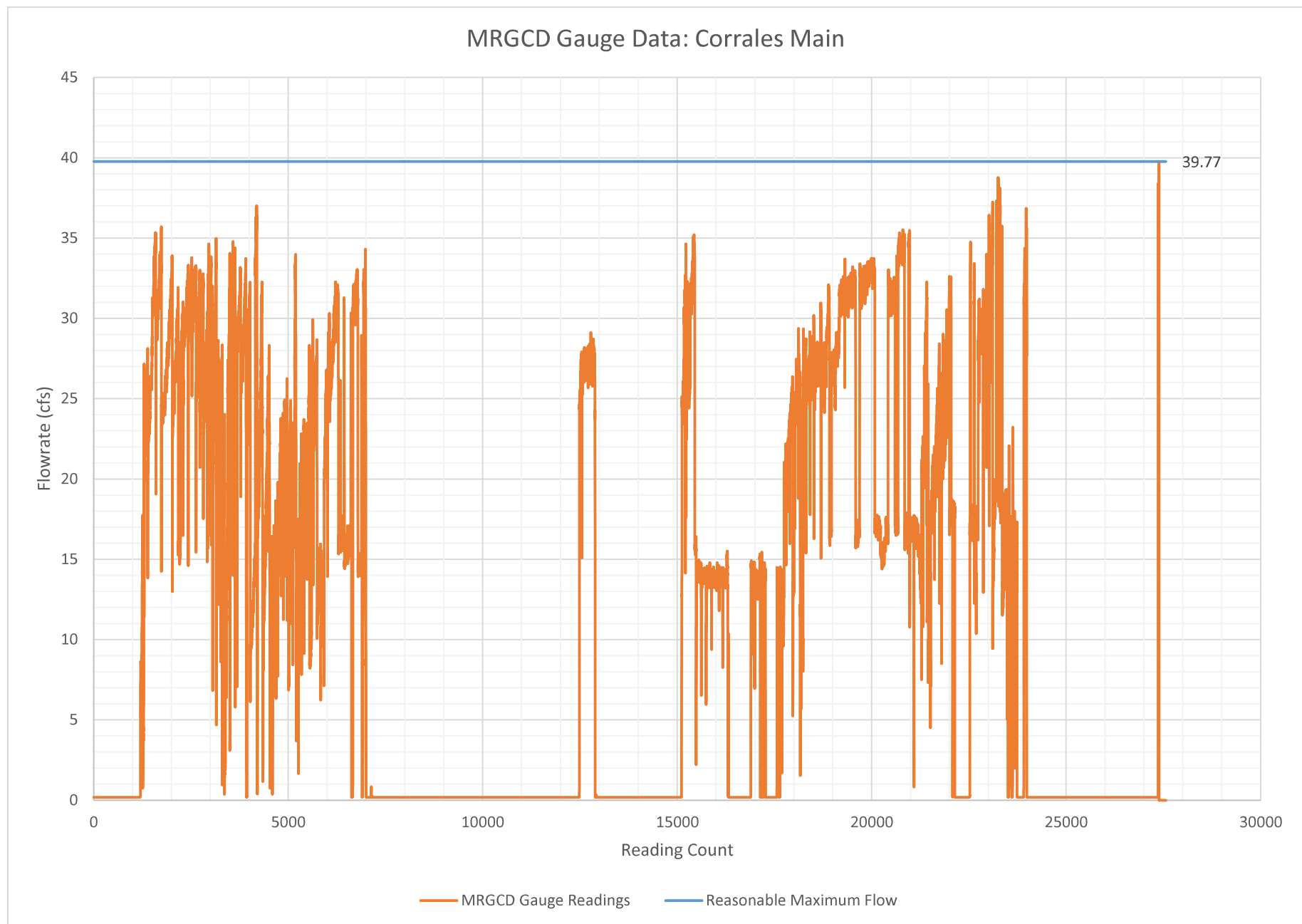
HJC Inflow Hydrograph (HMS "A_111_J4")



Dulcelina Curtis Inflow Hydrograph (HMS "U_901_J")

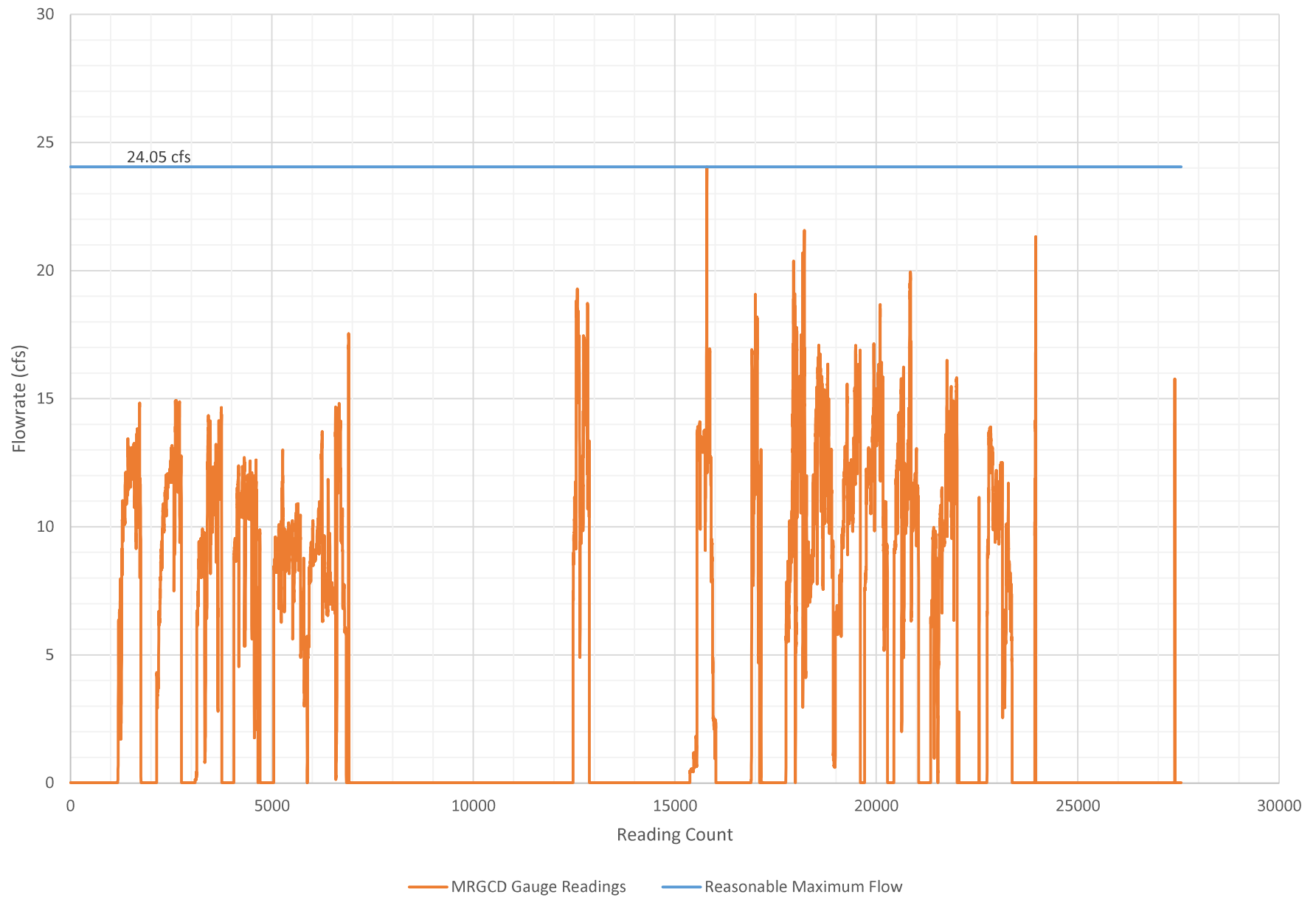


APPENDIX E – MRGCD IRRIGATION GAUGE DATA

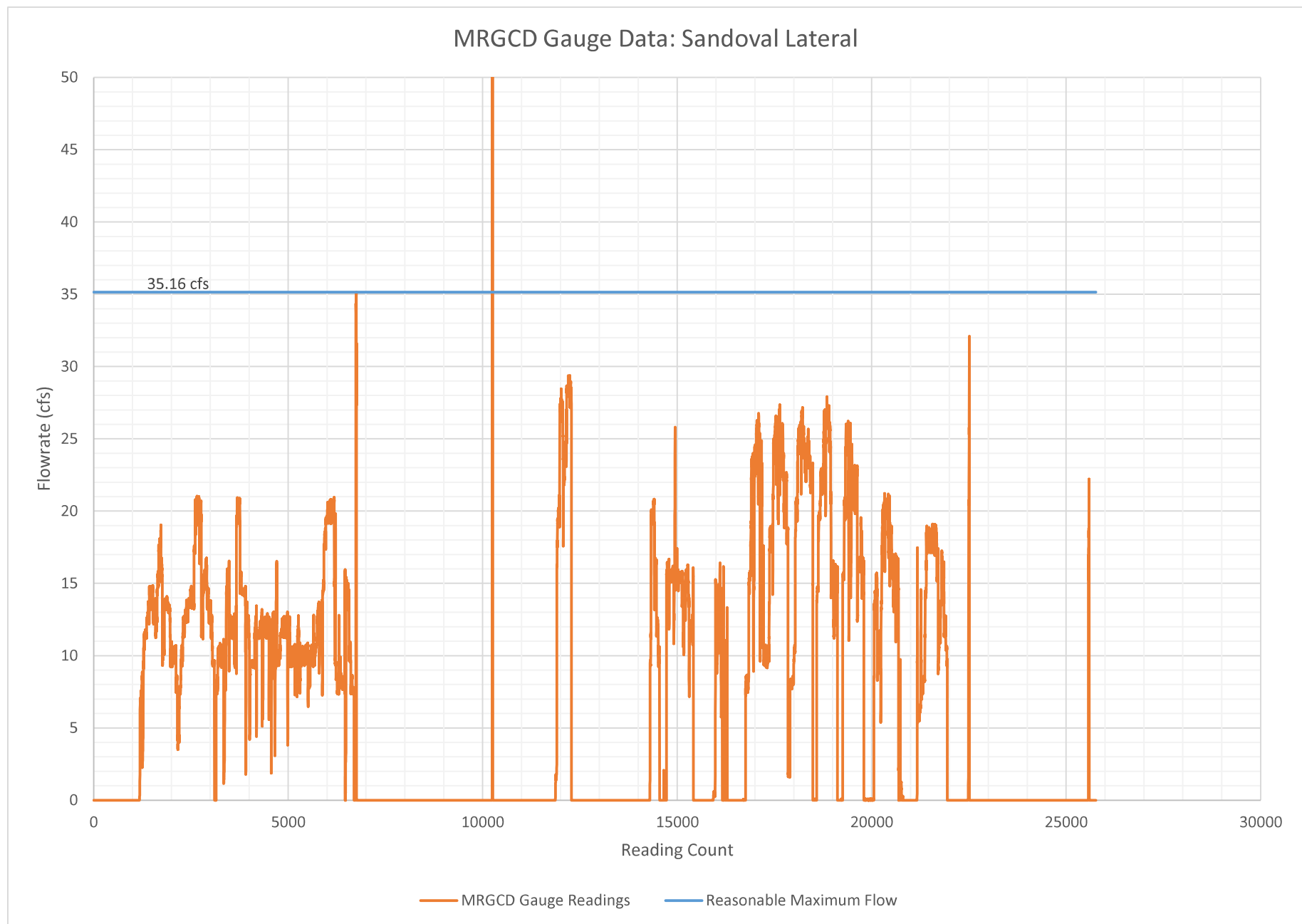


Note: MRGCD gauge time stamps are inconsistent and irrelevant to the "Reasonable Maximum Flow" baseflow input to the HEC-RAS model. Therefore, MRGCD gauge flowrate readings were plotted against reading count rather than time.

MRGCD Gauge Data: Corrales Acequia

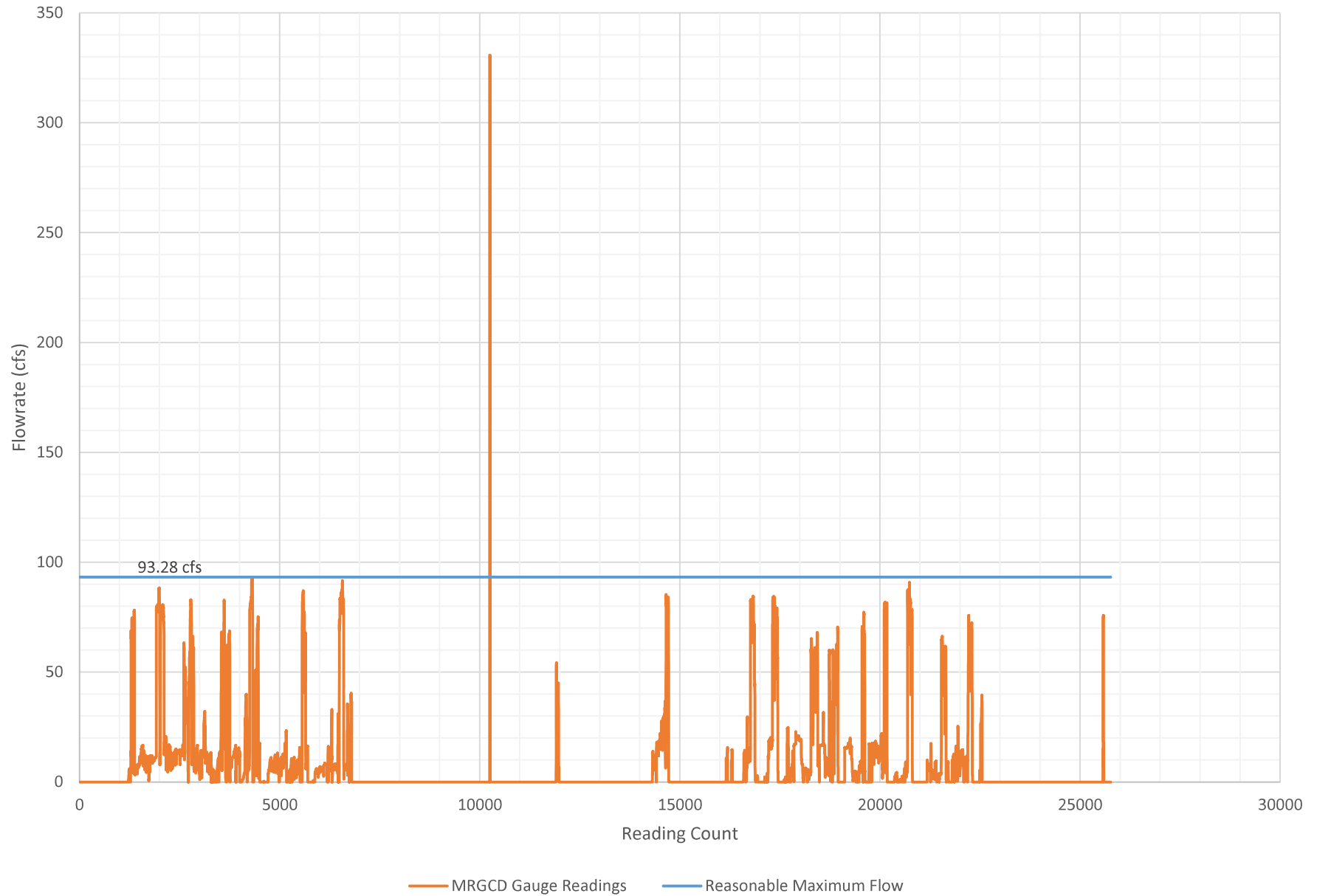


Note: MRGCD gauge time stamps are inconsistent and irrelevant to the "Reasonable Maximum Flow" baseflow input to the HEC-RAS model. Therefore, MRGCD gauge flowrate readings were plotted against reading count rather than time.



Note: MRGCD gauge time stamps are inconsistent and irrelevant to the "Reasonable Maximum Flow" baseflow input to the HEC-RAS model. Therefore, MRGCD gauge flowrate readings were plotted against reading count rather than time.

MRGCD Gauge Data: Corrales Check 06



Note: MRGCD gauge time stamps are inconsistent and irrelevant to the "Reasonable Maximum Flow" baseflow input to the HEC-RAS model. Therefore, MRGCD gauge flowrate readings were plotted against reading count rather than time.

APPENDIX F – HEC-HMS MODEL (DIGITAL)

APPENDIX G – HEC-RAS MODEL (DIGITAL)

APPENDIX H – INUNDATION SHAPEFILES (DIGITAL)