# Hemphill Diversion Rehabilitation

# Hydraulic Model Report

**Draft**
**Revision 0**

January 19, 2022

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## Attachments

# Distribution

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Nevada Irrigation District

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# Revision Log

| Revision No. | Date | Revision Description |
|---|---|---|
| 0 | 01/19/2022 | DRAFT Hydraulic Model Report |
| | | |

# 1.0 Introduction

McMillen Jacobs Associates (McMillen Jacobs) was retained by the Nevada Irrigation District (NID) to provide Engineering services for the Auburn Ravine and Hemphill Diversion Rehabilitation Project, located approximately 2 miles east of Lincoln, in Placer County, California. The site vicinity is presented in Figure 1.1. The rehabilitation project consists of replacing an existing diversion dam with a roughened channel to provide upstream passage for anadromous fish while maintaining essential diversion flows to Hemphill Canal. McMillen Jacobs prepared a hydrologic and hydraulic analysis for the establishment of design criteria for the Project, which includes the development of a two-dimensional hydraulic model of the Auburn Ravine near the Hemphill diversion with HEC-RAS Version 6.1 (USACE, 2016).


**Figure 1-1 Project Vicinity**

## 1.1 Purpose

The purpose of this report is to present the development and detailed results of the hydraulic simulations performed for the Auburn Ravine in the vicinity of Hemphill diversion for existing and proposed conditions. The model results will be used to demonstrate the effect of the proposed rehabilitation on water surface elevations and velocities in the vicinity of the diversion.

## 1.2 Background

The Hemphill diversion structure diverts water from the Auburn Ravine into the Hemphill Canal which provides raw water to NID customers. Presently, the structure impedes the passage of anadromous fish and other local species. The Hemphill Canal Rehabilitation Project proposes to construct a roughened channel fishway, construct a fish screen at the diversion, and re-grade/modify the Hemphill Canal.

Auburn Ravine is one of the many tributaries to the Feather River, and subsequently the Sacramento River, which has been identified by the Central Valley Steelhead Draft Recovery Plan (NMFS, 2009) as a good candidate for habitat restoration. The Auburn Ravine is a unique and valuable system where summertime habitats are created that are not normally found in foothill locations due to the flows that augment natural flow from the Yuba/Bear Watershed and the American River watershed. While winter flows increase with stormwater runoff, summer flows are controlled by diverting water into the Auburn Ravine. By modifying or eliminating the existing Hemphill diversion structure, an upstream passage can be provided for anadromous fish providing additional and essential fish habitat. Chinook Salmon, Steelhead and Pacific lamprey have all been found in Auburn Ravine along with Rainbow Trout, Sacramento Sucker, and the Sacramento pikeminnow. These species will benefit from the rehabilitation of the existing Hemphill diversion structure.

# 2.0 Hydraulic Model Input Data

Data used to develop the hydraulic model include the terrain data, the assumed survey datum, landcover and hydrologic conditions. The following paragraphs present these input data.

## 2.1 Topographic Mapping

Topographic data used for the project consists of Light Detection and Ranging (LiDAR) topography obtained from the United States Geologic Survey (USGS), ground survey conducted by O'Dell Engineering in December 2021 in the vicinity of the Hemphill diversion, and thalweg survey collected by NID in March 2020. The LiDAR topographic data and ground-based surveys were combined to create the terrain for the hydraulic model. The model terrain for existing condition is presented in **Error! Reference source not found.**

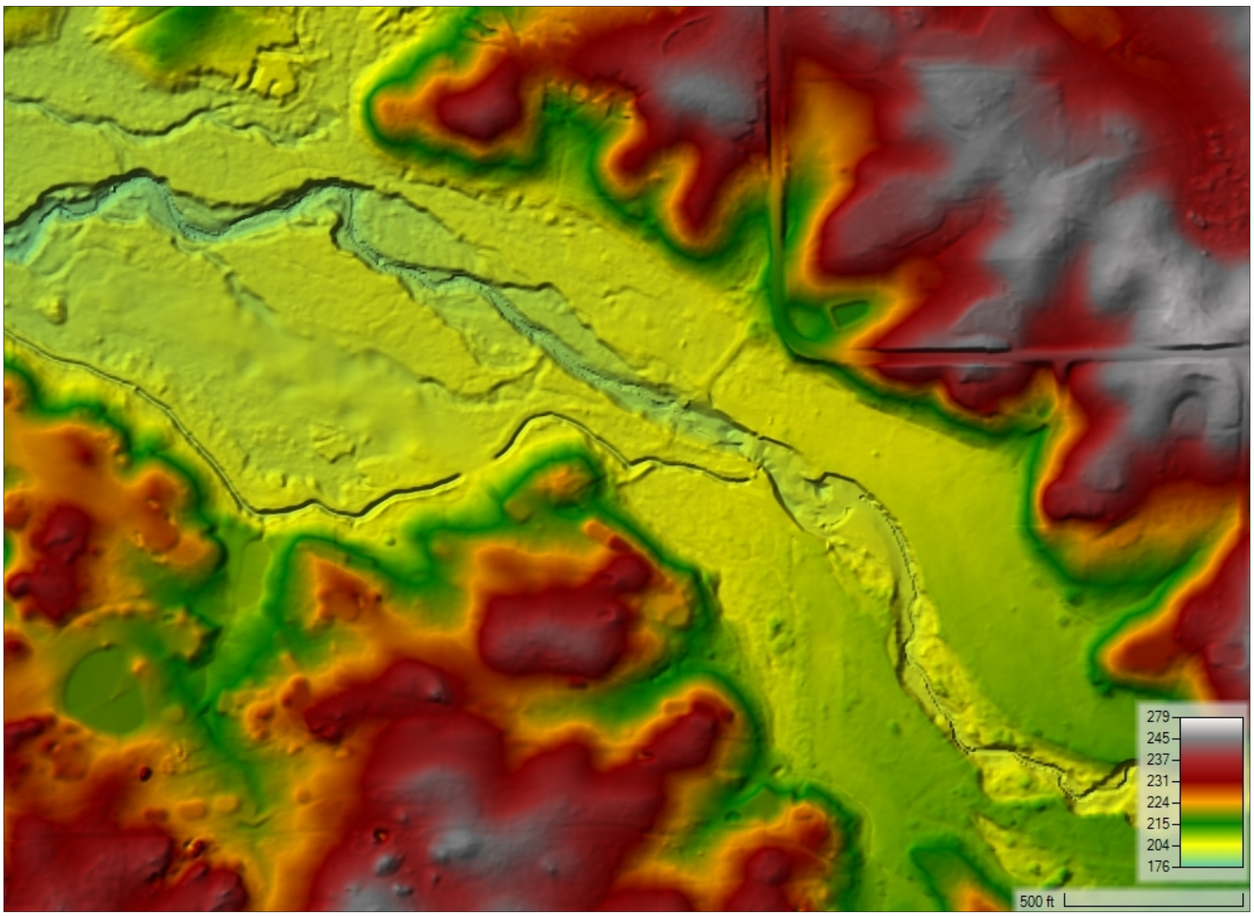


**Figure 2-1 Model Terrain**

## 2.2 Survey Datum and Projection

The Project data provided was in reference to the North American Vertical Datum of 1988 (NAVD88). This is the vertical datum that will be used in all calculations for the Project. The horizontal coordinate system is the State Plane California Zone II, North American Datum of 1983 (NAD83) in feet.

## 2.3 Landcover Data

Landcover in the vicinity of Hemphill diversion was obtained from Google Earth Imagery. Landcover was classified into seven categories for the assignment of Manning's roughness coefficients (*n*-values). The *n*-values were assigned to the land cover types according to guidance from data presented in Open Channel Hydraulics (Chow, 1959) and the Flood Insurance Study (FIS) for Placer County, California (FEMA, 2019). The *n*-values adopted in the FIS for Auburn Ravine vary from 0.015 to 0.071 for the channel and overbank areas. Figure 2-2 presents the land cover. The *n*-values adopted for the hydraulic modeling are presented in Table 2-2.

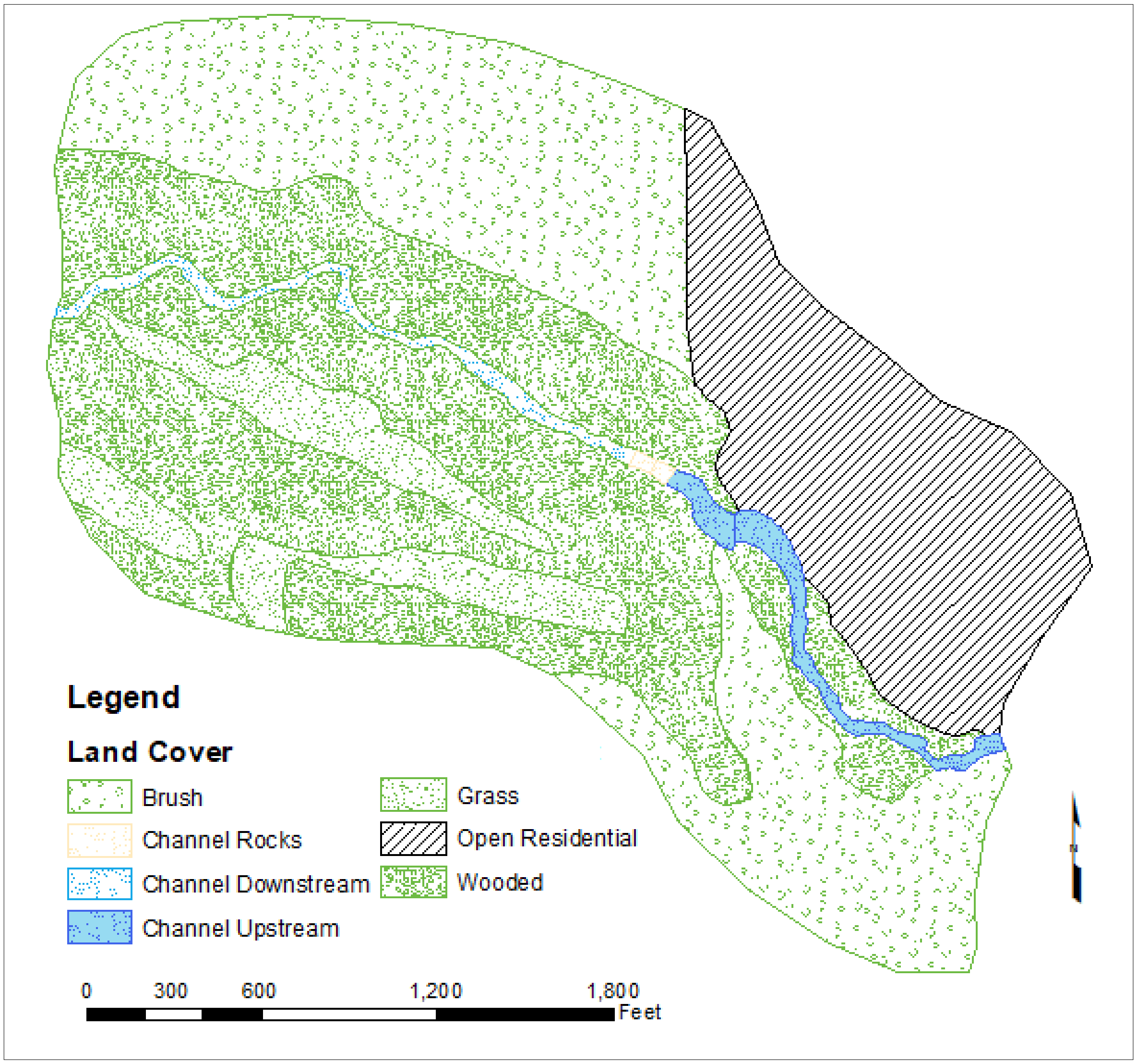

**Table 2-2. Manning's Roughness Coefficients**

| Land Cover | n-value | Description |
|---|---|---|
| Rocky Channel | 0.05 | Main Channels (clean, stones) |
| Channel Downstream | 0.027 | Main Channels (clean, straight) |
| Channel Upstream | 0.045 | Main Channels (pools, shoals, stones) |
| Brush | 0.05 | Floodplains (light brush) |
| Grass | 0.03 | Floodplains (short grass) |
| Open Residential | 0.05 | Floodplains (light brush and trees) |
| Wooded | 0.06 | Floodplains (trees) |

## 2.4 Hydrologic Condition

According to the StreamStats report for the contributing watershed of Hemphill Diversion, the drainage area is approximately 25.9 square miles, with mean elevation of 844 feet and 7.2 percent of impervious areas. The mean annual precipitation is 31.1 inches. The StreamStats one percent annual exceedance probability (AEP) flow, which is equivalent to the 100-year flow, at the Hemphill diversion is 4,660 cfs. The StreamStats report is presented in Attachment A.

According to the Flood Insurance Study (FIS) for Placer County (FEMA, 2019) the 1% AEP flow of Auburn Ravine near the Hemphill diversion is 15,643 cfs. The FIS flow for Auburn Ravine is derived with deterministic hydrologic modeling, whereas the StreamStats flow is calculated using regression equations. The use of different methodologies explain the large difference in flows. The FIS flow is effectively used for regulatory purposes to determine the flood plain extent and the base flood elevation. Therefore, the FEMA 1% AEP flow was used as the hydrologic condition for the hydraulic analysis.

# 3.0 Hydraulic Model Development

The following section outlines the development of the baseline hydraulic model (existing conditions for 100-year flow) for the section of the Auburn Ravine near the Hemphill Diversion. Simulation of the proposed condition will follow a similar methodology for the baseline model development and will be based on similar parameters and boundary conditions presented here. This section describes how the 2D HEC-RAS model was setup, including a discussion of the model geometry and boundary conditions.

## 3.1 HEC-RAS Software

The hydraulic analysis of the Auburn Ravine near the Hemphill Diversion was conducted using the HEC-RAS Version 6.1 (USACE, 2016). HEC-RAS software is capable of one-dimensional (1D) and two-dimensional (2D) unsteady hydrodynamic routing using the Saint-Venant equations or the Diffusion Wave equations. The following paragraphs present the data used to develop the two-dimensional hydraulic model. HEC-RAS was chosen because it is a well-vetted, industry-standard software program.

## 3.2 Model Geometries

The model geometry consists of the 2D flow area. The 2D flow area is the computational mesh that combines elevation, roughness, boundary conditions and other information used in the flow calculations. The 2D flow area defines the model extent, which includes approximately 0.70 mile of the Auburn Ravine, starting approximately 0.3 mile upstream of the diversion structure. A key consideration in the development of the computational grid is defining appropriate cell sizes for the terrain and flow conditions while maintaining appropriate simulation run times.

The cells making up the computational mesh were defined with a variable size according to location. The cells along the main channel of the Auburn Ravine are on average 25 square feet (5 feet by 5 feet) and are aligned with the flow direction. The average cell size within the model domain is approximately 100 square feet (10 feet by 10 feet). The computational grid is further refined by aligning computational cells along breaklines. Breaklines are used to align cell edges with slope breaks following the top of banks, thalweg, or structural edges. **Error! Reference source not found.** shows the extent of the 2D flow area. Figure 3-2 presents the mesh configuration near the Hemphill Diversion for the baseline simulation.

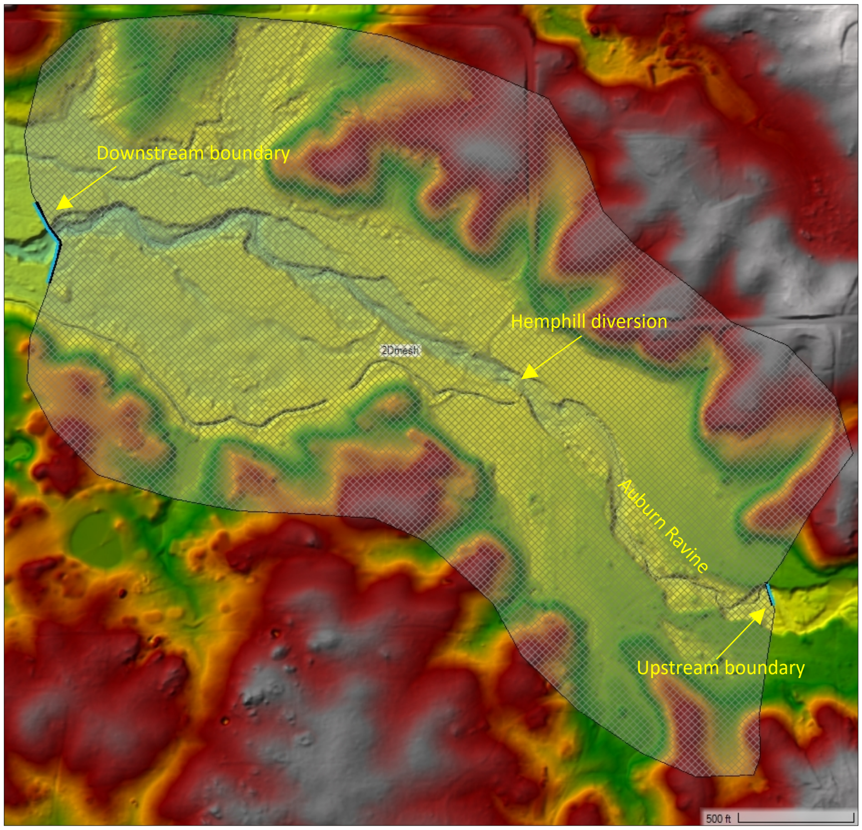


**Figure 3-1 Model 2D Mesh Extent**

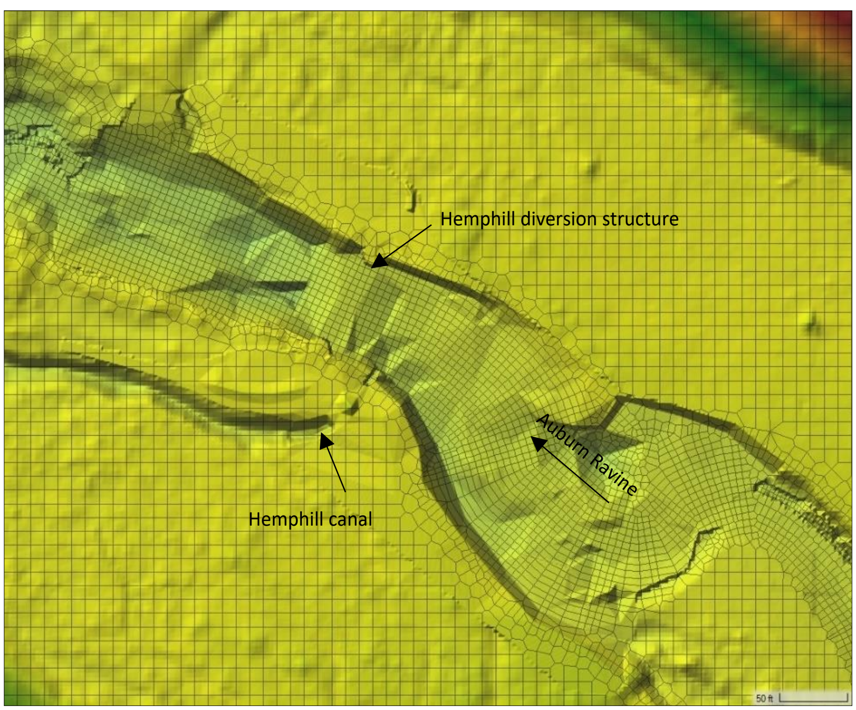


**Figure 3-2 Mesh Configuration Near Hemphill Diversion**

## 3.3 Boundary and Initial Conditions

Boundary conditions define how water enters and exits the model. Initial conditions are used to set water surface elevation (WSE) throughout the model at the start of each model run. A flow hydrograph boundary condition is included in the upstream limit of the numerical mesh. A normal depth boundary condition is included in the downstream limit of the numerical mesh. Figure 3-1, above, presents the location of the boundary conditions. The upstream boundary condition was assigned a flow hydrograph describing the inflow to the model. The inflow represents the 100-year flow for the baseline model described in Section 2.4. The flow is assumed constant at the upstream boundary. The downstream boundary describes the outflow conditions. A normal depth boundary condition with a friction slope of 0.0035, was considered for the baseline simulation. The friction slope was determined based on the channel slope. The boundary conditions for the baseline simulation are summarized in **Error! Reference source not found.**.

**Table 3-1 Boundary Conditions**

| Simulation | Auburn Ravine Upstream Boundary Flow (cfs) | Downstream Boundary Friction Slope |
|---|---|---|
| Baseline | 15,643 | 0.0035 |

## 3.4 Simulation Parameters

The simulation was performed in unsteady mode using the diffusion wave equation and a 4-second time-step. The total simulation time was three hours. The simulation time was sufficient to obtain quasi steady state flows in the vicinity of the diversion structure.

# 4.0 Baseline Simulation Results

The results of the baseline simulation, model calibration and sensitivity analysis are discussed in this section. Figure 4-1 presents the simulated depth for the 100-year flow. Figure 4-2 presents the water surface elevation profile near the Hemphill diversion. The simulated water surface elevation at the structure diversion is 205.5 feet.

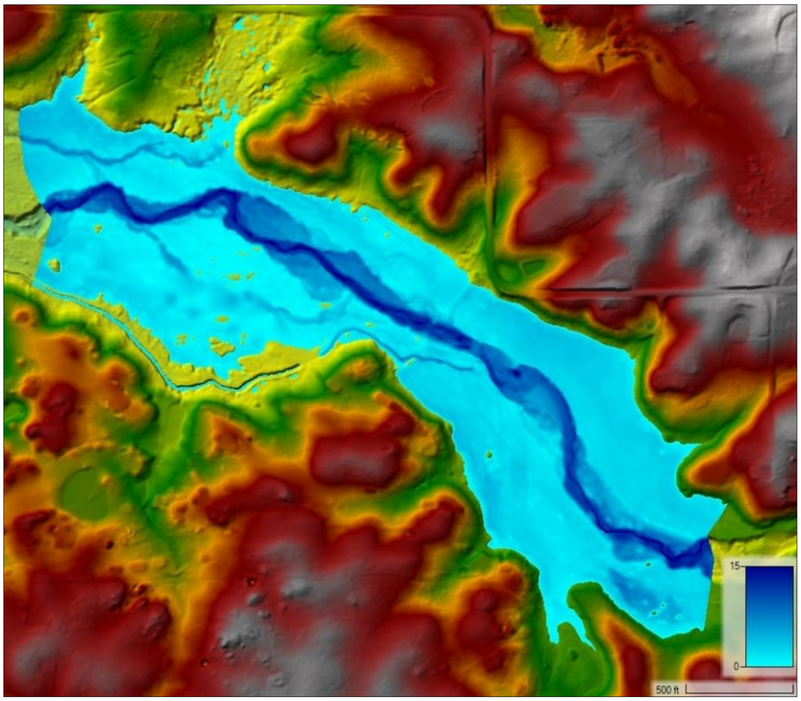


**Figure 4-1 Simulated Depth 100-year Flow (Depth in feet)**

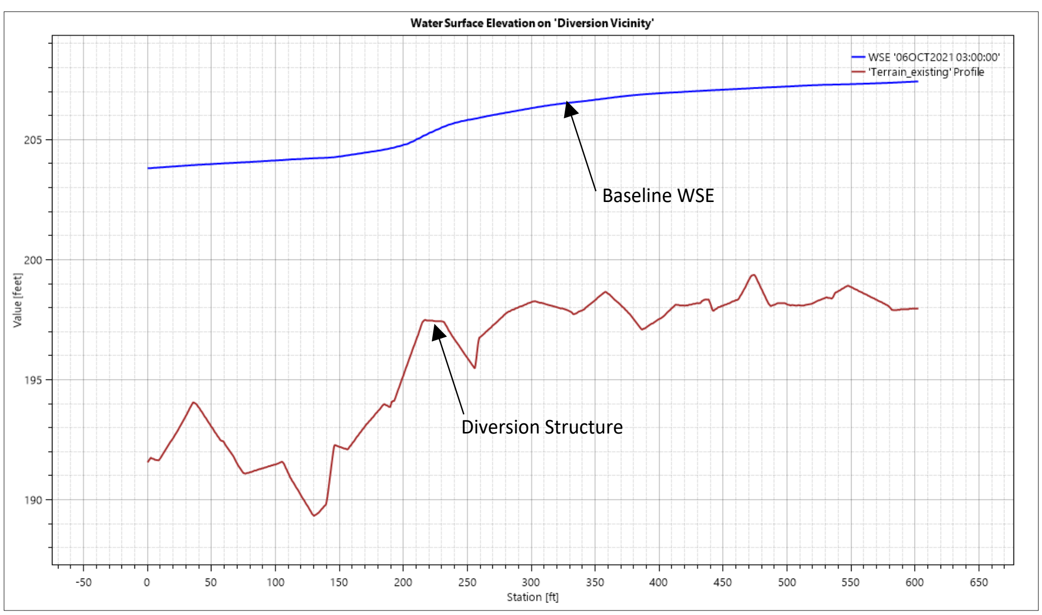


**Figure 4-2 Water Surface Elevation Profile 100-year Flow**

Figure 4-3, below, presents the flow hydrograph at the Hemphill diversion structure.

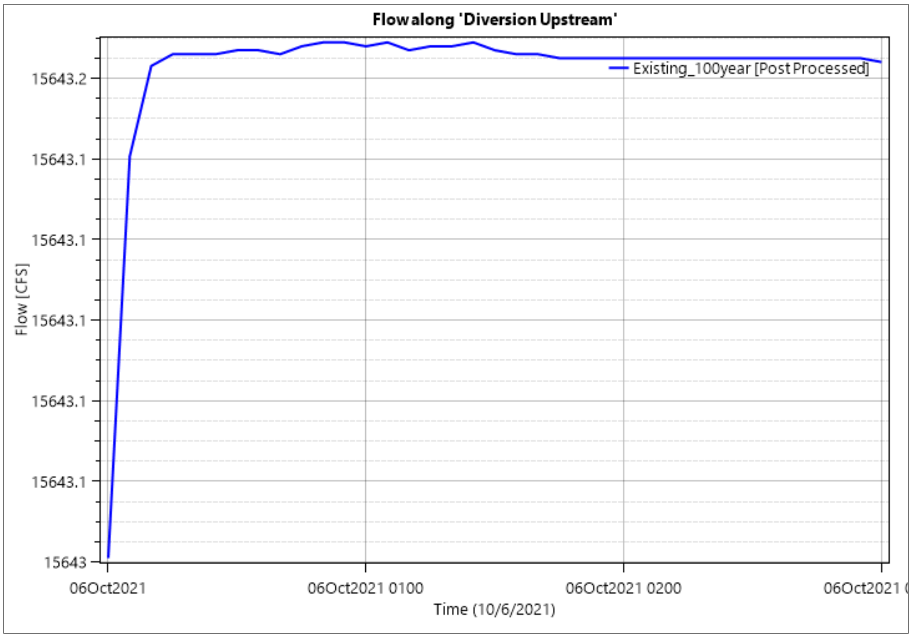


**Figure 4-3 Flow Hydrograph Near the Diversion Structure**

The simulated velocities for the baseline scenario in the vicinity of the Hemphill diversion are presented in Figure 4-4. The simulated average velocity in the vicinity of the diversion structure is approximately 20 feet per second.

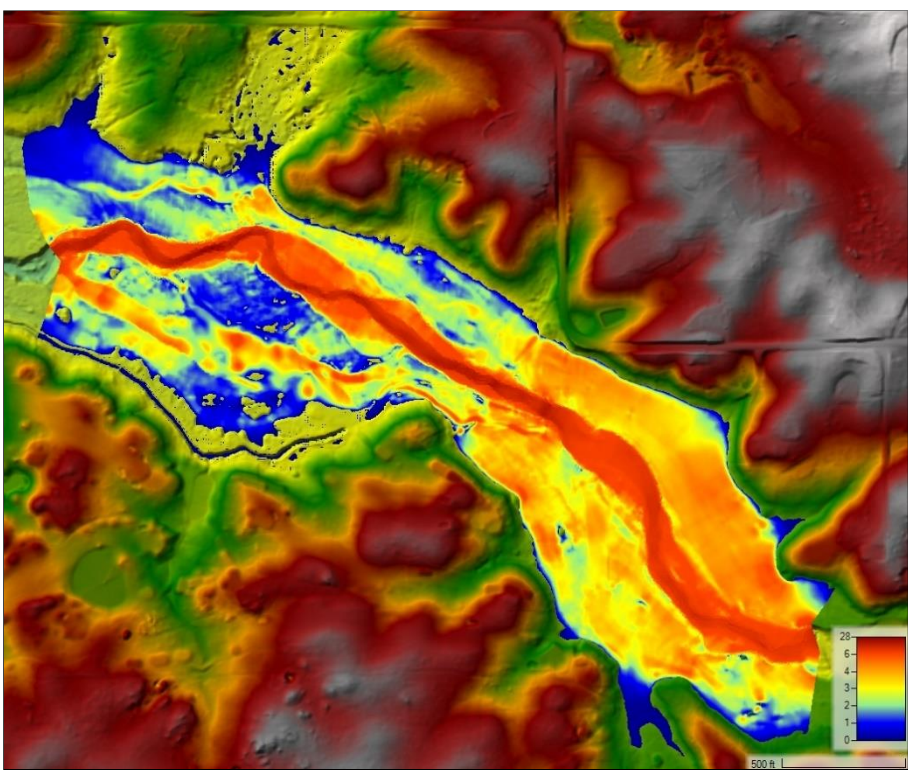

**Figure 4-4 Velocity 100-year Flow (feet/second)**

## 4.1 Model Calibration

Model calibration was performed based on the FIRM for the city of Lincoln, Placer County, California. (FEMA, 2018). Figure 4.5 shows an excerpt of the FIRM panel 719 for the city of Lincoln near the Hemphill Diversion. The complete panel 719 is presented in Attachment B. According to the FIRM, The FEMA base flood elevation in the vicinity of the diversion is 206.8 feet. The model calibration consisted of adjusting *n*-values to simulate water surface elevations as close as possible to FEMA's base flood elevations at the respective cross-sections. The calibration was performed for five FEMA cross-sections included within the 2D hydraulic model domain. The focus of the calibration was cross-section AM, which is the closest to the Hemphill diversion. Table 4.1 presents the FEMA base flood elevations and the predicted water surface elevations obtained with the 2D hydraulic model for the selected cross-sections. The FEMA base flood elevations were obtained with a 1D modeling approach and using an older version of the terrain. Therefore, the results obtained with the 2D hydraulic model are different for some cross-sections. The average difference between the 2D model water surface elevations and FEMA base flood elevations for the five cross-sections evaluated is 0.12 feet.

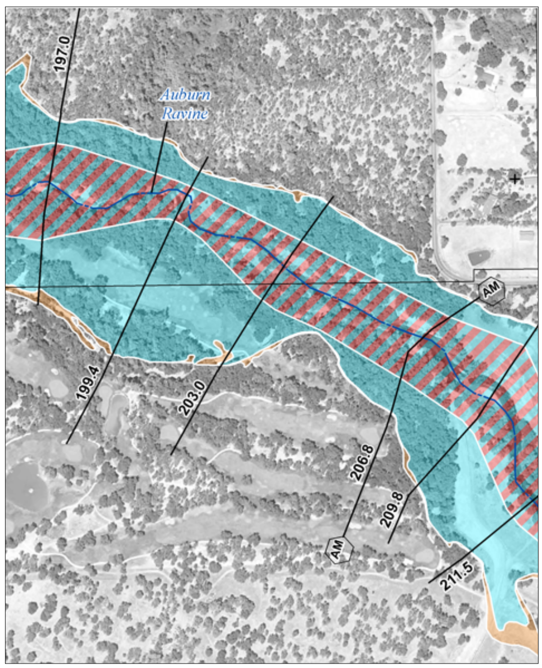


**Figure 4-5 FIRM Panel 719 Excerpt**

**Table 4-1 FEMA Base Flood Elevations and 2D Model Water Surface elevations**

| FEMA Base Flood Elevation (feet) | Model Water Surface Elevation 100-year Flow (feet) | Difference (feet) |
|---|---|---|
| 197.0 | 197.0 | 0.0 |
| 199.4 | 200.0 | 0.6 |
| 203.0 | 201.9 | -1.1 |
| 206.8 | 206.8 | 0.0 |
| 209.8 | 210.9 | 1.1 |

## 4.2 Sensitivity Analysis

A sensitivity analysis was performed on *n*-values. The *n*-values were adjusted up and down to their maximum and minimum extents identified for each land cover type included in the model and are shown in **Error! Reference source not found.**.

**Table 4-1. Range of Manning's Roughness Coefficients Evaluated for Sensitivity Analysis**

| Land Cover Type | Model's n-value | Minimum n-value | Maximum n-value | Explanation |
|---|---|---|---|---|
| Rocky Channel | 0.05 | 0.045 | 0.06 | Main Channels (clean, stones) |
| Channel Downstream | 0.027 | 0.015 | 0.033 | Main Channels (clean, straight) |
| Channel Upstream | 0.045 | 0.033 | 0.05 | Main Channels (pools, shoals, stones) |
| Brush | 0.05 | 0.035 | 0.06 | Floodplains (light brush) |
| Grass | 0.03 | 0.025 | 0.035 | Floodplains (short grass) |
| Open Residential | 0.05 | 0.04 | 0.08 | Floodplains (light brush and trees) |
| Wooded | 0.06 | 0.05 | 0.08 | Floodplains (trees) |

The results of the sensitivity analysis of Manning's roughness coefficients on water surface elevations are presented in **Error! Reference source not found.**. The results were evaluated at the FEMA cross-sections. By decreasing the *n*-values to their lowest recommended values there is a maximum decrease of 0.9% in water surface elevation. By increasing the *n*-values to their highest recommended values there is a maximum increase of 0.46% in water surface elevation. The results of the sensitivity analysis indicate that the expected range of variation of water surface elevations is approximately one foot above or below the predicted baseline value.

**Table 4-2 Sensitivity Analysis Results**

| FEMA Cross-section | Model n-value (Baseline) WSE (feet) | Minimum n-value WSE (feet) | Maximum n-value WSE (feet) | Minimum n-value % Difference | Maximum n-value % Difference |
|---|---|---|---|---|---|
| 197.0 | 197.0 | 196.1 | 197.9 | -0.46% | 0.46% |
| 199.4 | 200.0 | 198.2 | 200.7 | -0.90% | 0.35% |
| 203.0 | 201.9 | 200.1 | 202.7 | -0.89% | 0.40% |
| 206.8 | 206.8 | 206.0 | 207.4 | -0.39% | 0.29% |
| 209.8 | 210.9 | 210.3 | 211.6 | -0.28% | 0.33% |

# 5.0 Proposed Hemphill Diversion Rehabilitation Simulation

## 5.1 Model Geometries

The proposed design consists of replacing the existing diversion structure with a roughened channel and weir. The proposed roughened channel includes a triangular low-flow channel to maintain minimum depths during low flows for fish passage. The model geometry for the proposed condition model consists of the same 2D flow area extents that were used for the baseline model. The overall size of the cells making up the computational mesh and distribution were maintained for the proposed condition model. However, the cells in the vicinity of the diversion were rearranged to capture the proposed changes in channel geometry. A 2D Connection was included in the model geometry to represent the crest. Figure 5-1 presents the mesh configuration of the proposed condition model and terrain near the Hemphill diversion. Figure 5-2 presents the geometry of the proposed weir in HEC-RAS 2D.

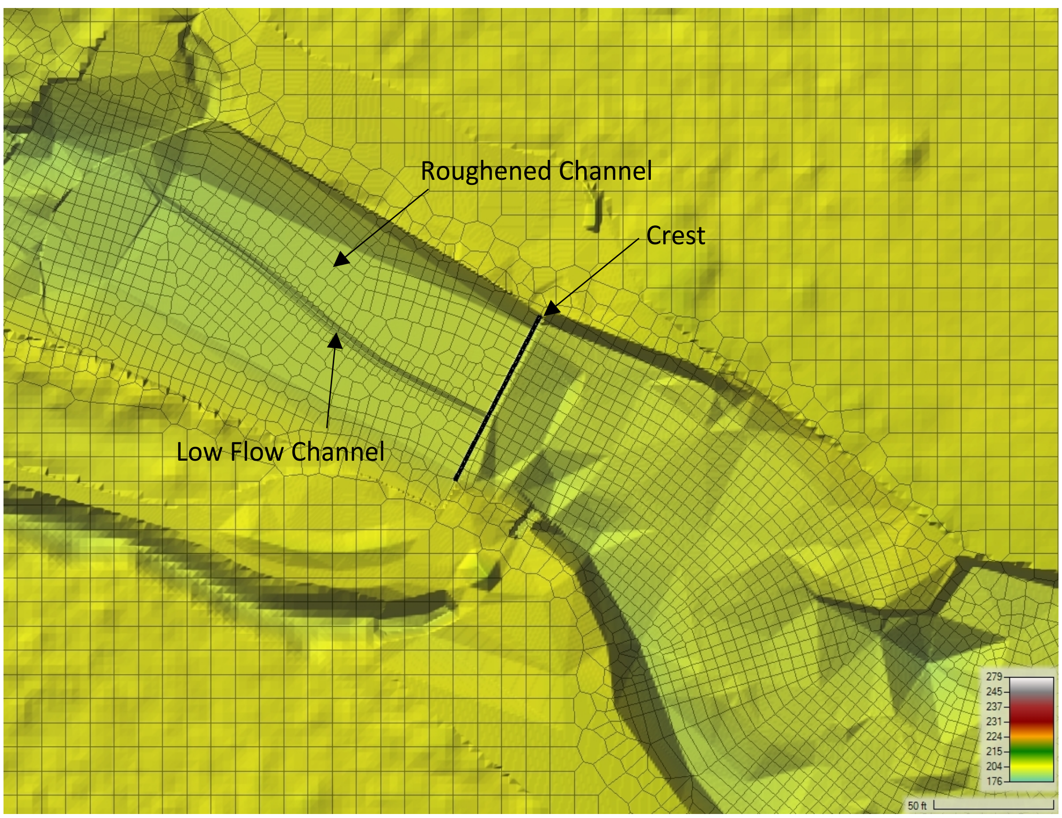


**Figure 5-1 Proposed Geometry Mesh**

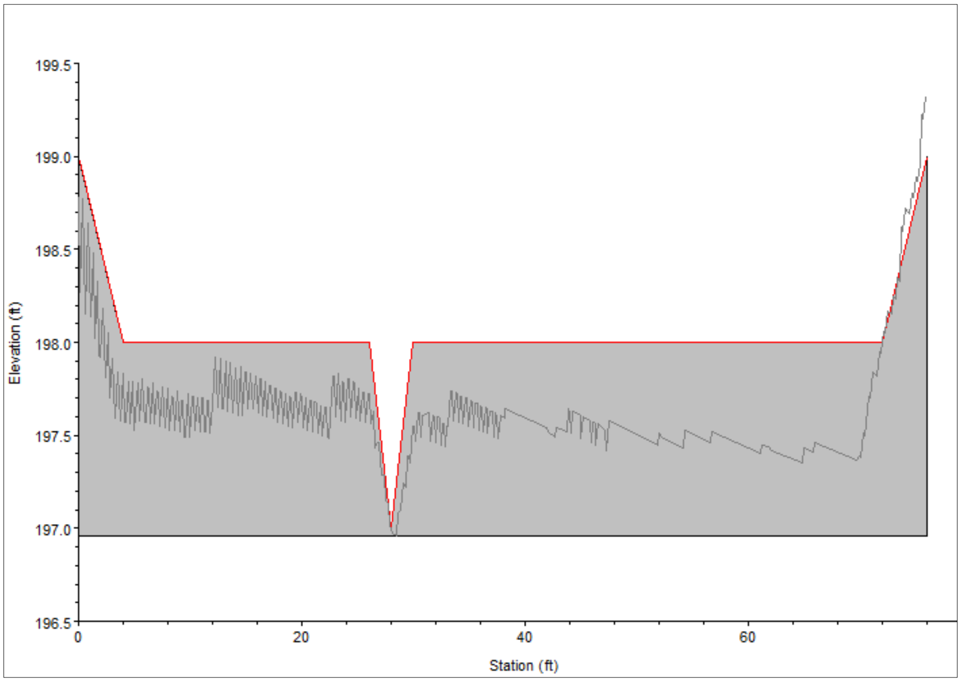


**Figure 5-2 Proposed Crest Geometry**

## 5.2 Boundary Conditions and Simulation Parameters

The hydrologic condition for the proposed condition simulation is the 100-year flow. The flow is assumed constant at the upstream boundary. The Manning's roughness coefficients used for the proposed condition are identical to the baseline model. However, an *n*-value of 0.05 (Rocky Channel) was applied for the extent of the proposed roughened channel. The total simulation time was three hours. The simulation time was sufficient to obtain quasi-steady state flows in the vicinity of Hemphill diversion.

## 5.3 Results of the Proposed Diversion Rehabilitation

The results of the proposed condition simulation are discussed in this section. Figure 5-3 presents the simulated depth. Figure 5-4 presents the profile of the simulated water surface elevation near the Hemphill diversion for proposed and existing condition. Figure 5-5 presents the flow hydrograph near the diversion structure. Figure 5-6 presents the water surface elevation profile at a cross-section just downstream of the proposed crest for proposed and baseline conditions.

The simulated water surface elevation at the proposed crest for the 100-year flow is 205.8 feet, which is approximately 4 inches above baseline condition (205.5 feet). The cross-section profile indicates that the greater rise in WSE occurs near the proposed crest, which indicates a localized impact for the 100-year event.

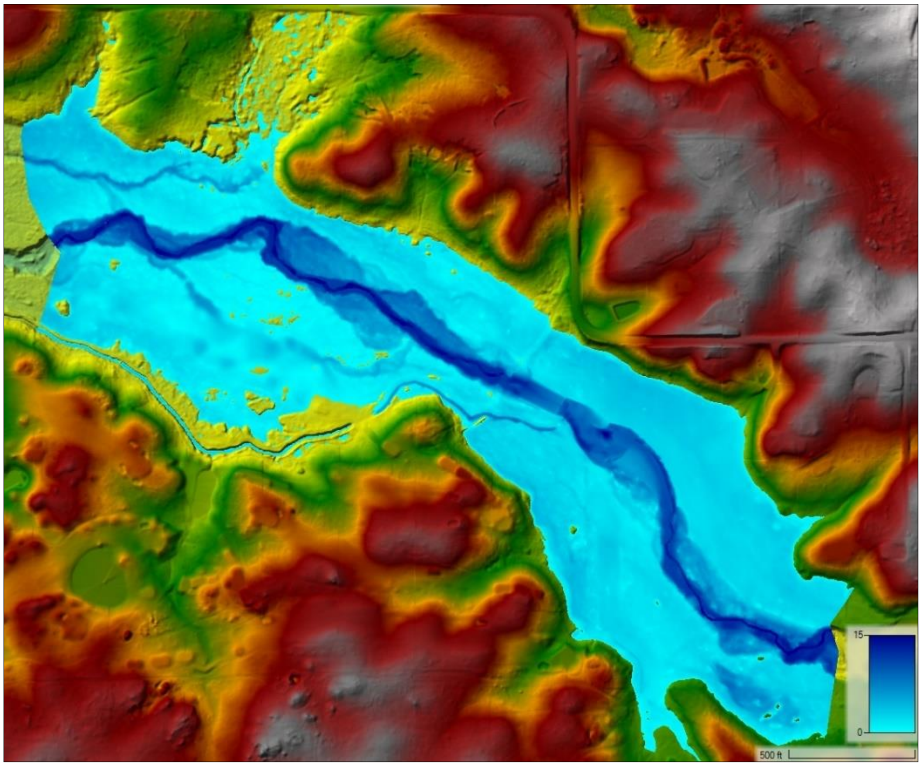


**Figure 5-3 Simulated Depth Proposed Condition (feet)**

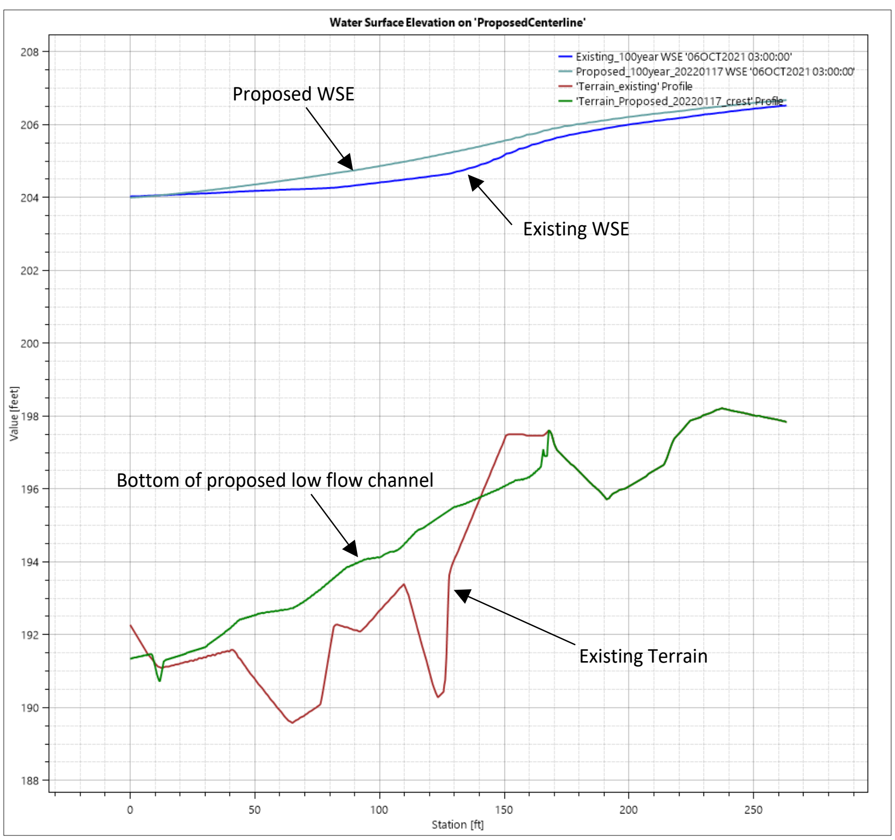


**Figure 5-4 Water Surface Elevation Profile**

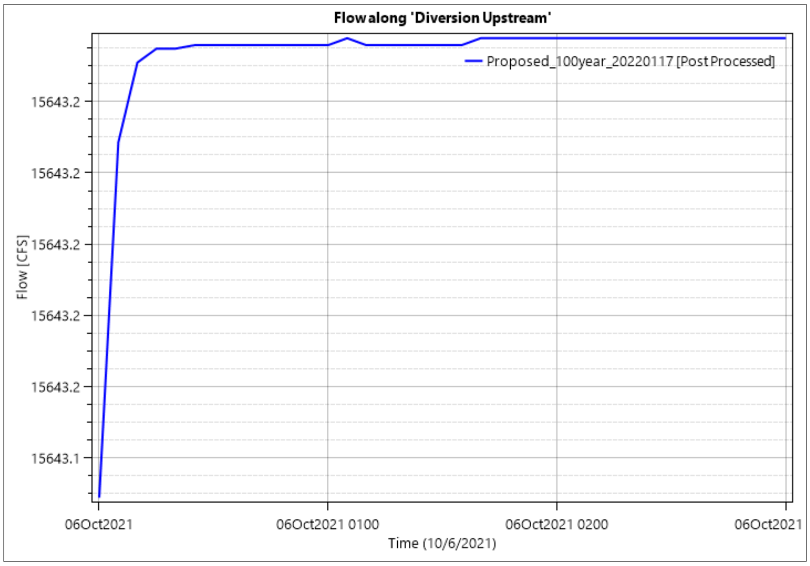


**Figure 5-5 Flow Hydrograph Near the Diversion Structure - Proposed Condition**

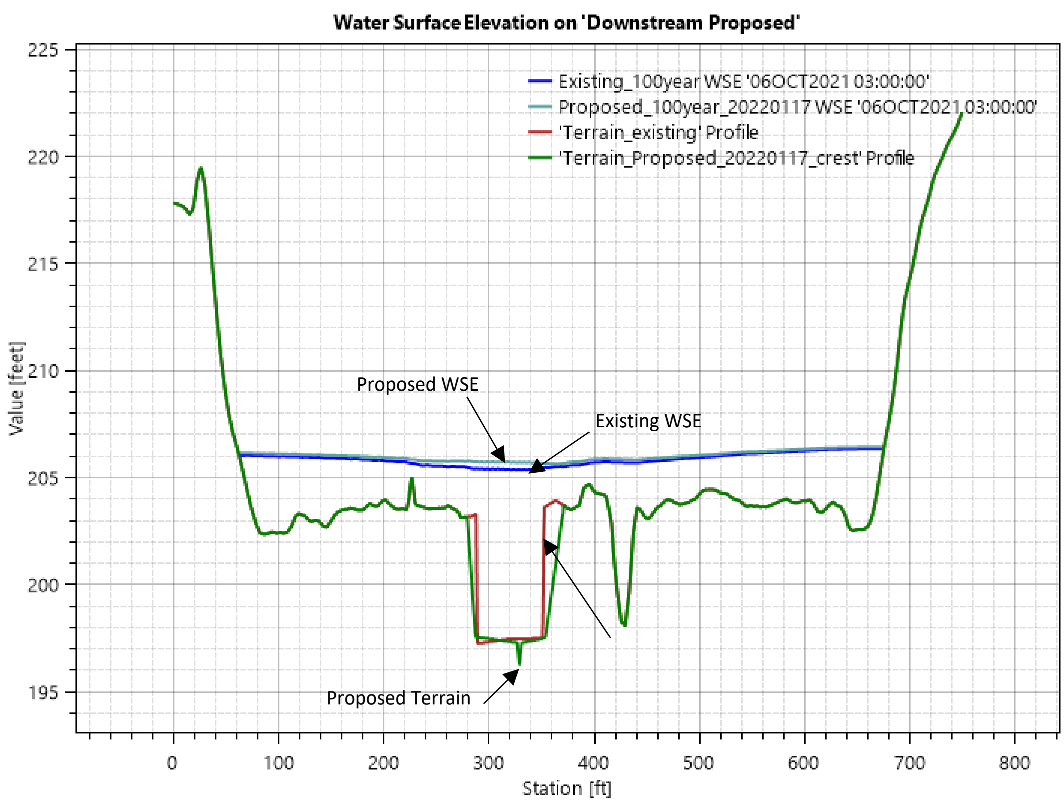


**Figure 5-6 Water Surface Elevation Profile Downstream the Weir**

# 6.0 Ravine Flows

Nevada Irrigation District has a stream gage downstream of the Hemphill Canal approximately 3 miles downstream near the City of Lincoln, CA. This gage has recorded hourly data from 1995 to the present. The intended purpose of the gage is to measure flow in Auburn Ravine to provide data that NID needs for improved water management. As such, the gage is focused on flows between 0 cfs and 200 cfs. Once the flow in Auburn Ravine increases past 200 cfs, the gage does not record reliable data.

Simulations were developed to evaluate the water surface elevation at the point of diversion for proposed conditions and flows varying from 5 cfs to 5,000 cfs. Table 6.1 presents the predicted water surface elevations for the range of flows evaluated. Figure 6-1 presents the rating curve at the point of diversion.

**Table 6-1 Predicted WSE at Diversion Point**

| Flow (cfs) | WSE (feet) |
|---|---|
| 2.0 | 198.0 |
| 13.3 | 198.1 |
| 20 | 198.1 |
| 50 | 198.2 |
| 100 | 198.3 |
| 200 | 198.6 |
| 500 | 199.1 |
| 1000 | 199.8 |
| 2500 | 201.2 |
| 5000 | 202.9 |

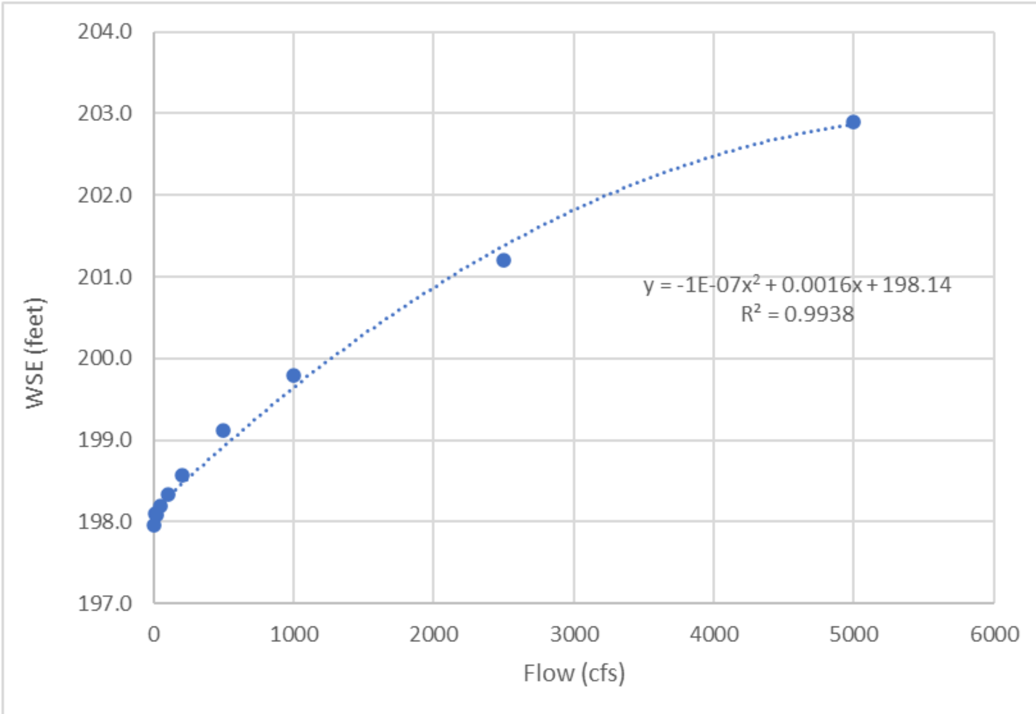


**Figure 6-1 Rating Curve at Diversion Point**

# 7.0 Sediment Transport

Previous reports that have addressed the removal or the Hemphill Diversion Structure (Balance, NHC) have suggested the need for further analysis of sediment transport and bank stability in Auburn Ravine if dam removal is selected as the preferred alternative. These reports were considering removing the existing structure and lowering the bed at the diversion structure by two (2) to five (5) feet. If the bed were lowered to that extent, further investigations into the resulting hydraulic characteristics and affects to sediment would be recommended. The currently proposed design would remove the existing structure and replace it with a roughened rock channel maintaining a crest at a similar elevation to the existing structure. This proposal would not have a significant effect on the upstream conditions of Auburn Ravine.

The proposed modifications to the point of diversion will allow the District to divert water without the use of stop logs. Removing the need for stop logs in Auburn Ravine will avoid the current condition where the stream banks are being saturated by a high water surface elevation each summer and then experiencing a rapid drawdown when the stop logs are removed in the fall. By returning the ravine to a more natural cycle of water surface elevation change, vegetation will take hold on the stream banks and a more stable channel will develop.

# 8.0 Hemphill Canal Hydraulics

NID maintains a flow gage on Hemphill Canal near Auburn Ravine that measure the water that is diverted into the canal. This gage has been recording hourly flow since 1995 during the irrigation season. From 1995 to 2000, the flow averaged approximately 12 cfs, from 2000 to 2012, the average flow was approximately 8 cfs, and from 2012 to the present the average flow has been closer to 6 cfs. The current NID Water Master Plan indicates that demand on Hemphill Canal could be as high as 18 cfs.

The flow conditions in the canal were analyzed for a minimum flow of 3 cfs, a normal flow of 6 cfs and a maximum flow of 18cfs. A water surface profile was developed for the canal between Auburn Ravine and the first culvert approximately 790 feet down canal. This culvert was analyzed and presents a hydraulic control point. Normal depth was calculated between the culvert and the outlet from the proposed fish screen, and a standard step backwater curve was developed to estimate the length of canal required to reach normal depth. At 6 cfs, 2,765 feet of canal would be required. Since the culvert is only 790 feet from the outlet, the water surface elevation at the outlet will be controlled by the backwater from the culvert and was found to be 197.57 feet.

Based on current design of the cone screen alcove, cone screen, pipe, and headwall, the headloss between the water surface elevation in Auburn Ravine (198.1 feet) at 95% flow (13.3 cfs) and the canal was calculated for flow rates of 3 cfs, 6 cfs, and 18cfs. Total head losses were found to be 0.21 ft, 0.31 ft, and 0.80 ft respectfully. The available head (198.1 – 197.57) of 0.53 feet is greater than the required head of 0.31 feet indicating that during the 95% flow in Auburn Ravine, NID will be able to successfully divert 6 cfs while meeting fish passage criteria in Auburn Ravine.

# 9.0 Roughened Channel

The existing concrete check board diversion dam is an impediment to upstream fish passage, creates an annual cycle of raising and lowering water which negatively affects upstream bank stability and required NID staff to annually install and remove check boards. NID proposes to remove this structure and construct a roughened channel extending downstream of the existing structure to maintain water surface elevation sufficient to deliver water in the existing Hemphill canal while also providing upstream fish passage.

A roughened channel is an engineered “nature like” fishway constructed with rocks and streambed material sized and placed to mimic the configuration of a natural stream bed. The roughened channel proposed for this project will have a low flow channel designed to maintain minimum flow depth of 1 foot at 7 cfs (13.3 cfs minus the normal flow of 6 cfs in Hemphill Canal). The low flow channel will be a V-shaped channel with 2H:1V side slopes. The leading edge of the roughened channel will be defined with a sheet pile crest which will keep water from flowing through the interstitial voids within the rocks. The roughened channel will be formed with streambed material meeting CDFW sizing and gradation criteria. Initial calculations point to a $D_{50}$ rock size of approximately 22 inches. Additionally, larger rocks will be distributed throughout the roughened channel to provide flow diversity and refuge locations.

# 10.0 Conclusions

McMillen Jacobs Associates prepared a two-dimensional hydraulic model of the Auburn Ravine near the Hemphill diversion with HEC-RAS Version 6.1 to evaluate the effect of the proposed diversion rehabilitation on the water surface elevations. The model was calibrated based on FEMA base flood elevations. The water surface elevations obtained with the Baseline simulation are in agreement with the FEMA 100-year flood elevations. The proposed design for the diversion rehabilitation will cause a maximum localized rise of water surface elevation of approximately four inches near the proposed crest. The overall rise in the base flood elevation is estimated to be approximately 0.16 feet near the diversion. Simulations of water surface elevations for the proposed design indicate the expected fluctuation of water surface elevations at the diversion point is five feet, between 197.9 to 202.9 feet for a 5 to 5,000 cfs flow range.

# 11.0 References

National Marine Fisheries Service. (2009). Public Draft Recovery Plan. NOAA. Sacramento.

Chow. (1959). Open-channel Hydraulics. New York: McGraw-Hill.

FEMA. (2019). *Flood Insurance Study, Placer County, California.*

FEMA. (2018). *Flood Insurance Rate Map, Placer County, California. Panel 719. Effective November 2, 2018.*

USACE. (2016). *HEC-RAS River Analysis System User's Manual.* Washington, DC: United States Army Corps of Engineers.

# Attachment A

# StreamStats Report

# Hemphill Diversion Project

**Region ID:** CA
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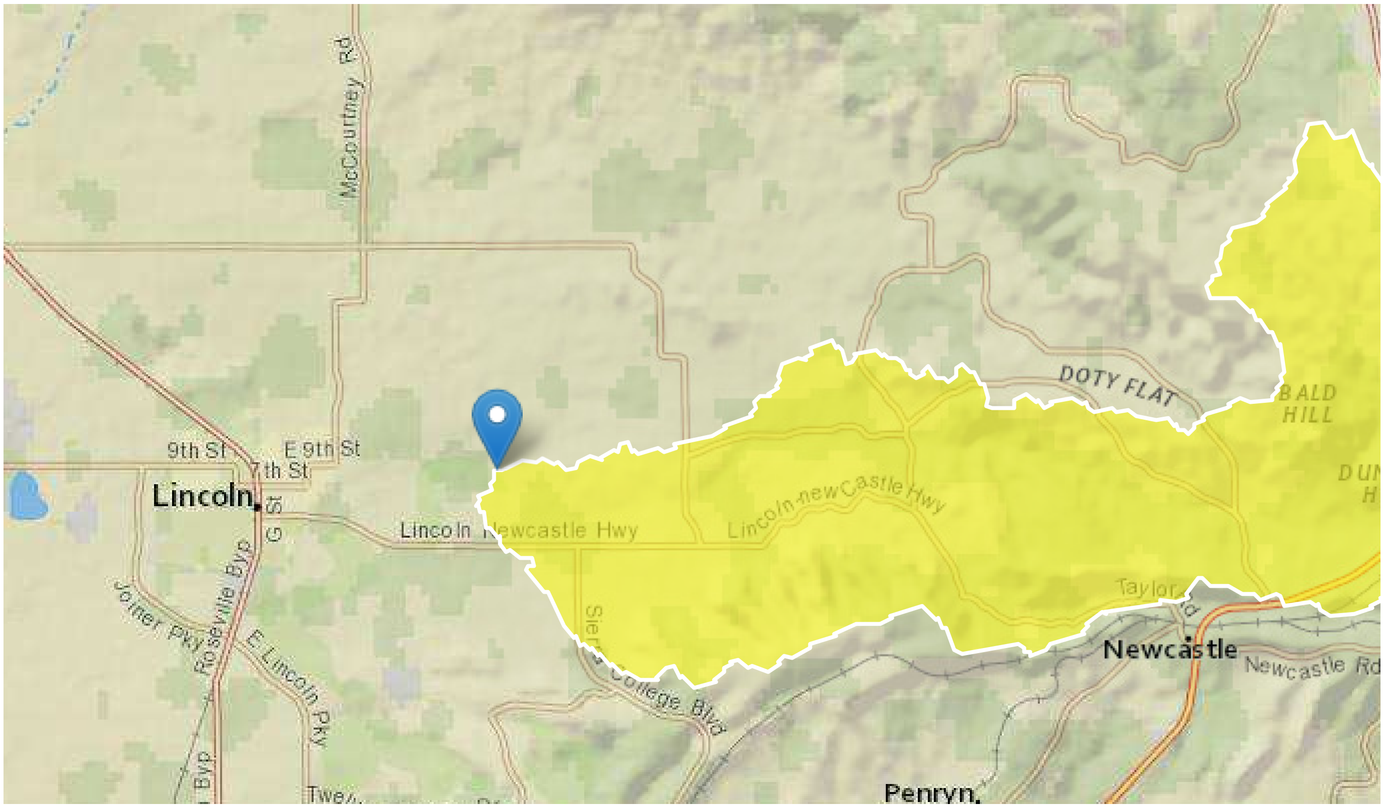
**Clicked Point (Latitude, Longitude):** 38.89659, -121.25205
**Time:** 2022-01-19 14:36:59 -0800




Basin Characteristics

| Parameter Code | Parameter Description | Value | Unit |
|---|---|---|---|
| DRNAREA | Area that drains to a point on a stream | 25.9 | square miles |
| ELEV | Mean Basin Elevation | 844 | feet |
| PRECIP | Mean Annual Precipitation | 31.1 | inches |

Peak-Flow Statistics Parameters [2012 5113 Region 3 Sierra Nevada]

| Parameter Code | Parameter Name | Value | Units | Min Limit | Max Limit |
|---|---|---|---|---|---|

| Parameter Code | Parameter Name | Value | Units | Min Limit | Max Limit |
|---|---|---|---|---|---|
| DRNAREA | Drainage Area | 25.9 | square miles | 0.07 | 2000 |
| ELEV | Mean Basin Elevation | 844 | feet | 90 | 11000 |
| PRECIP | Mean Annual Precipitation | 31.1 | inches | 15 | 100 |

Peak-Flow Statistics Flow Report [2012 5113 Region 3 Sierra Nevada]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, ASEp: Average Standard Error of Prediction, SE: Standard Error (other -- see report)

| Statistic | Value | Unit | PIl | PIu | ASEp |
|---|---|---|---|---|---|
| 50-percent AEP flood | 752 | ft^3/s | 252 | 2250 | 74.4 |
| 20-percent AEP flood | 1690 | ft^3/s | 727 | 3930 | 54.4 |
| 10-percent AEP flood | 2390 | ft^3/s | 1070 | 5320 | 51.5 |
| 4-percent AEP flood | 3250 | ft^3/s | 1460 | 7260 | 52.3 |
| 2-percent AEP flood | 3960 | ft^3/s | 1710 | 9150 | 54.6 |
| 1-percent AEP flood | 4660 | ft^3/s | 1930 | 11200 | 58 |
| 0.5-percent AEP flood | 5350 | ft^3/s | 2120 | 13500 | 61.5 |
| 0.2-percent AEP flood | 6280 | ft^3/s | 2310 | 17100 | 67.3 |

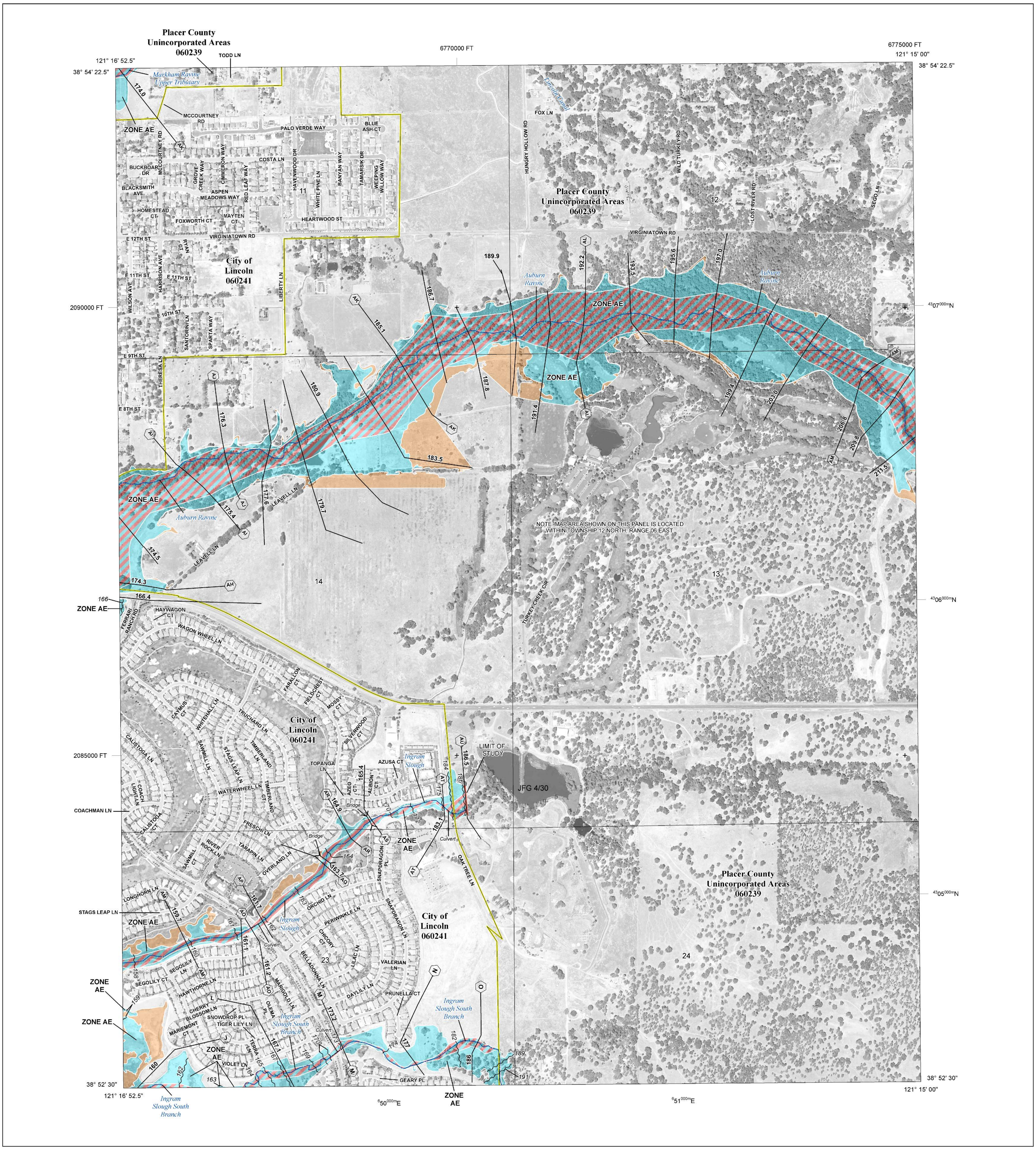
*Peak-Flow Statistics Citations*

**Gotvald, A.J., Barth, N.A., Veilleux, A.G., and Parrett, Charles,2012, Methods for determining magnitude and frequency of floods in California, based on data through water year 2006: U.S. Geological Survey Scientific Investigations Report 2012–5113, 38 p., 1 pl. (http://pubs.usgs.gov/sir/2012/5113/)**

Application Version: 4.6.2
StreamStats Services Version: 1.2.22
NSS Services Version: 2.1.2

# Attachment B

# FIRM




## FLOOD HAZARD INFORMATION

**SEE FIS REPORT FOR ZONE DESCRIPTIONS AND INDEX MAP**

**THE INFORMATION DEPICTED ON THIS MAP AND SUPPORTING DOCUMENTATION ARE ALSO AVAILABLE IN DIGITAL FORMAT AT HTTP://MSC.FEMA.GOV**

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

## NOTES TO USERS

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

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

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

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

Base map information shown on this FIRM was provided in digital format by the USDA National Agriculture Imagery Program (NAIP). This information was photogrammetrically compiled at a scale of 1:24,000 from aerial photography dated 2014.

## SCALE

Map Projection:
NAD 1983 StatePlane California II FIPS 0402 Feet; Western Hemisphere; Vertical Datum: NAVD 88

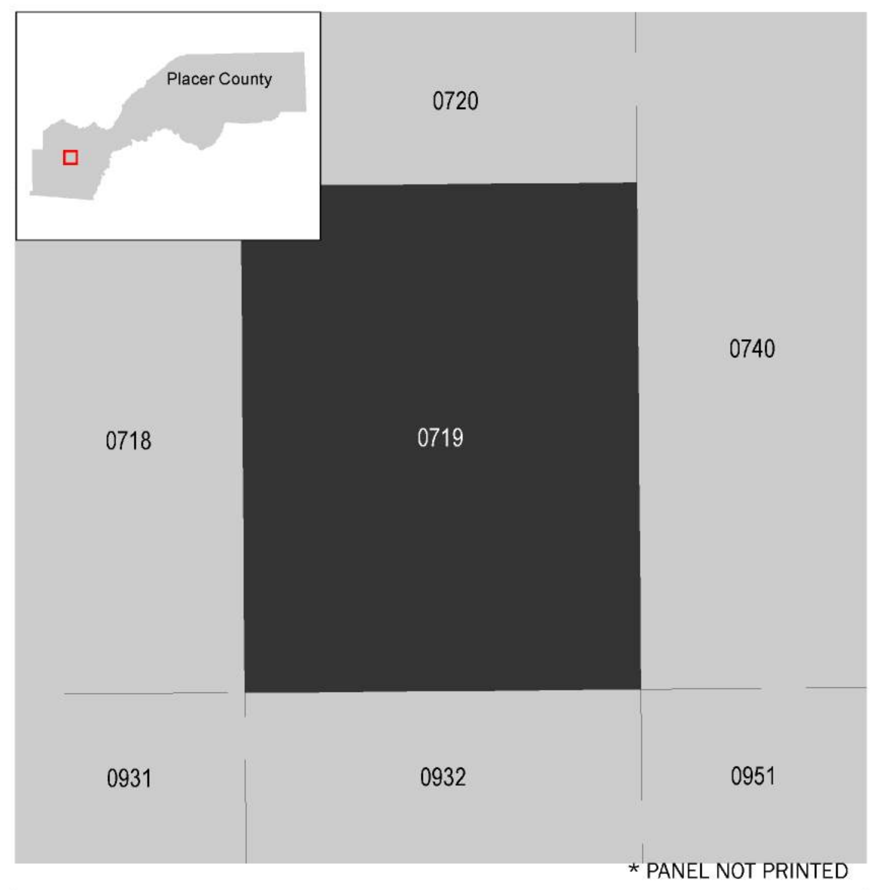
N

1 inch = 500 feet 1:6,000

0 500 1,000 2,000 Feet

0 125 250 500 Meters

## PANEL LOCATOR




* PANEL NOT PRINTED

FEMA

National Flood Insurance Program

**NATIONAL FLOOD INSURANCE PROGRAM**
FLOOD INSURANCE RATE MAP

**PLACER COUNTY, CA**
And Incorporated Areas

PANEL **719** OF **1060**

Panel Contains:

| COMMUNITY | NUMBER | PANEL | SUFFIX |
|---|---|---|---|
| LINCOLN, CITY OF | 060241 | 0719 | H |
| PLACER COUNTY | 060239 | 0719 | H |

VERSION NUMBER
2.3.3.3

MAP NUMBER
06061C0719H

MAP REVISED
NOVEMBER 2, 2018

# Attachment C

# HYDRULIC CALCULATIONS

# Calculation Cover Sheet

**Project:** Hemphill Diversion Project

**Client:** Nevada Irrigation District - NID **Proj. No.** 21-125

**Title:** Hemphill Hydraulics - 50% Submittal

**Prepared By, Name:** J. Burgi

**Prepared By, Signature:** ______________________ **Date:** ________________

**Peer Reviewed By, Name:** M. Cerucci

**Peer Reviewed, Signature:** ______________________ **Date:** ________________

| | | | | |
|---|---|---|---|---|
| **SUBJECT:** | Nevada Irrigation District - NID | **BY:** J. Burgi | **CHK'D BY:** | M. Cerucci |
| | Hemphill Diversion Project | **DATE:** | | |
| | Hemphill Hydraulics - 50% Submittal | **PROJECT NO.:** 21-125 | | |

**Table of Content**

**Hydraulics** **Page**

SUBJECT: Nevada Irrigation District - NID
Hemphill Diversion Project
Hemphill Canal Entrance Head Loss

BY: J. Burgi CHK'D BY: M. Cerucci
DATE: 1/19/2022
PROJECT NO.: 21-125

## Purpose

The purpose of this calculation sheet is to identify the hydraulic grade line from Auburn Ravine to the canal.

## References


•*Lindeburg, Michael. (2003). Civil Engineering Reference Manual, California, Professional Publications, Inc.*


## Information - Input

Hemphill canal flow conditions

| | | |
|---|---|---|
| $Q_{min}$ | 3 | cfs |
| $Q_{design}$ | 6 | cfs |
| $Q_{max}$ | 18 | cfs |

Auburn Ravine Water Surface Elevation

| | |
|---|---|
| $WSEL_{min}$ | 198.5 |
| $WSEL_{design}$ | |
| $WSEL_{max}$ | |

## Calculation

Headloss over weir entering cone screen alcove

$$Q=3.33*b*h^{\frac{3}{2}}$$ (Lindburg, 2003) eq. 19.51b

b 21.6 ft length of weir

| Q (cfs) | h (ft) |
|---|---|
| 3 | 0.12 |
| 6 | 0.19 |
| 18 | 0.40 |

Headloss through Cone Screen

Based on conversations with manufacturers representative - headloss is less than 1 inch through a cone screen. Assuming approach velocity is less than 0.33 fps.

Headloss through pipe from cone screen to canal.

$$h=\frac{3.022v^{1.85}L}{C^{1.85}D^{1.17}}$$ (Lindburg, 2003) eq. 17.30

| | | | |
|---|---|---|---|
| v | | fps | velocity |
| L | 75 | ft | length |
| C | 140 | | roughness coefficienet |
| D | 3 | ft | diameter |

| Q (cfs) | v (fps) | h (ft) |
|---|---|---|
| 3 | 0.424 | 0.0014 |
| 6 | 0.849 | 0.0050 |
| 18 | 2.546 | 0.0378 |

Minor Headloss in pipe from cone screen to canal

$$h_m = K * \frac{v^2}{2g}$$ (Lindburg, 2003) eq. 17.41

| Q (cfs) | v (fps) | $v^2/2g$ |
|---|---|---|
| 3 | 0.424 | 0.0028 |
| 6 | 0.849 | 0.0112 |
| 18 | 2.546 | 0.1007 |

K

| | |
|---|---|
| 0.9 | 90 elbow |
| 0.45 | 45 elbow |
| 1 | exit |

Minor losses in pipe include one 90 degree bend, two 45 degree bends and one exit.

| Q (cfs) | Total minor loss (ft) |
|---|---|
| 3 | 0.008 |
| 6 | 0.031 |
| 18 | 0.282 |

## Conclusion

Total losses from Auburn Ravine to the Canal include a weir, the cone screen, pipe, minor losses through the vertical and horizontal bends as well as the exit loss.

| | 3 cfs | 6 cfs | 18 cfs |
|---|---|---|---|
| Weir loss (ft) | 0.12 | 0.19 | 0.40 |
| Cone loss (ft) | 0.08 | 0.08 | 0.08 |
| Pipe loss (ft) | 0.00 | 0.00 | 0.04 |
| Minor Loss (ft) | 0.01 | 0.03 | 0.28 |
| **Total (ft)** | **0.21** | **0.31** | **0.80** |

**SUBJECT:** Nevada Irrigation District - NID
Hemphill Diversion Project
Hemphill Canal Head Loss

**BY:** J. Burgi **CHK'D BY:** M. Cerucci
**DATE:** 1/19/2022
**PROJECT NO.:** 21-125

## Purpose

*The purpose of this calculation sheet is to identify the hydraulic grade line between the first Turkey Creek Golf Club culvert and the outlet from the fish screen.*

## References


- *Tullis, J. Paul. (1989). Hydraulics of Pipelines, Pumps, Valves, Cavitation, Transients. New York: John Wiley & Sons.*
- *Miller, D.S. (1990). Internal Flow Systems, Design and Performance Prediction. Houston: Gulf Publishing Company.*


## Information - Input

Hemphill canal flow conditions

| | | |
|---|---|---|
| $Q_{min}$ | 3 | cfs |
| $Q_{design}$ | 6 | cfs |
| $Q_{max}$ | 18 | cfs |

Hemphill Canal

| | | | |
|---|---|---|---|
| $W_b$ | 7 | ft | bot width |
| z | 1 | :1 | side slope |
| S | 0.0002 | ft/ft | Slope for end of proposed fish screen to culvert |
| L | 790 | ft | Distance from culvert to outlet |
| h | 0.158 | | Change in elevation at the bottom of canal |
| n | 0.025 | | Manning's coefficient |

## Calculation

Based on HY-8 analysis of first culvert (located approximately 790 feet downstream from the proposed fish screen, flow in the culvert is outlet controled, and the WSE at the entrance of the culvert is calculated as:

| Flow cfs | WSE ft | depth ft |
|---|---|---|
| 3 | 197.26 | 0.92 |
| 6 | 197.65 | 1.31 |
| 18 | 198.72 | 2.38 |

Analysis will start from the hydraulically controled downstream end. The first culvert on Hemphill canal is approximatlet 790 feet downstream.

Calculation of Normal Depth between fish screen and culvert.

$$Q = \frac{1.486}{n} * A * R^{2/3} * S^{1/2}$$

$$A = \frac{Q * n}{1.49 * S^{1/2} * R^{2/3}}$$

$$A = d * (w + zd)$$

$$P = w + 2d(1 + t^2)^{0.5}$$

| Q, cfs | Normal Depth d, ft | A, ft$^2$ | P, ft | R, fr | V , fps |
|---|---|---|---|---|---|
| 3.00 | 0.81 | 6.34 | 9.29 | 0.68 | 0.47 |
| 6.00 | 1.09 | 8.77 | 10.07 | 0.87 | 0.68 |
| 18.00 | 1.72 | 15.03 | 11.87 | 1.27 | 1.20 |

Standard step backwater calculation to find length of canal required to transition from flow depth at the culvert entrance to normal depth.

Backwater calc for 6 cfs

| d (ft) | A (ft2) | V (ft/s) | E (ft) | delta E | R (ft) | Sf | Avg -Sf | So-Avg Sf | dl (ft) | Cum Dist | Elev. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1.31 | 10.89 | 0.55 | 1.31 | | 1.01689457 | 8.41E-05 | | | | | 197.65 |
| | | | | -0.02 | | | 8.63E-05 | 1.14E-04 | -174.46 | -174.46 | |
| 1.29 | 10.69 | 0.56 | 1.29 | | 1.00426617 | 8.86E-05 | | | | | 197.63 |
| | | | | -0.02 | | | 9.10E-05 | 1.09E-04 | -181.85 | -356.30 | |
| 1.27 | 10.50 | 0.57 | 1.28 | | 0.9915784 | 9.34E-05 | | | | | 197.61 |
| | | | | -0.02 | | | 9.60E-05 | 1.04E-04 | -190.49 | -546.80 | |
| 1.25 | 10.31 | 0.58 | 1.26 | | 0.97883032 | 9.86E-05 | | | | | 197.59 |
| | | | | -0.02 | | | 1.01E-04 | 9.86E-05 | -200.73 | -747.53 | |
| 1.23 | 10.12 | 0.59 | 1.24 | | 0.96602094 | 1.04E-04 | | | | | 197.57 |
| | | | | -0.02 | | | 1.07E-04 | 9.29E-05 | -213.03 | -960.56 | |
| 1.21 | 9.93 | 0.60 | 1.22 | | 0.95314928 | 1.10E-04 | | | | | 197.55 |
| | | | | -0.02 | | | 1.13E-04 | 8.67E-05 | -228.05 | -1188.61 | |
| 1.19 | 9.75 | 0.62 | 1.20 | | 0.9402143 | 1.16E-04 | | | | | 197.53 |
| | | | | -0.02 | | | 1.20E-04 | 8.01E-05 | -246.78 | -1435.38 | |
| 1.17 | 9.56 | 0.63 | 1.18 | | 0.92721497 | 1.23E-04 | | | | | 197.51 |
| | | | | -0.02 | | | 1.27E-04 | 7.30E-05 | -270.75 | -1706.14 | |
| 1.15 | 9.37 | 0.64 | 1.16 | | 0.91415023 | 1.31E-04 | | | | | 197.49 |
| | | | | -0.02 | | | 1.35E-04 | 6.53E-05 | -302.46 | -2008.60 | |
| 1.13 | 9.19 | 0.65 | 1.14 | | 0.90101898 | 1.39E-04 | | | | | 197.47 |
| | | | | -0.02 | | | 1.43E-04 | 5.70E-05 | -346.31 | -2354.90 | |
| 1.11 | 9.00 | 0.67 | 1.12 | | 0.88782011 | 1.47E-04 | | | | | 197.45 |
| | | | | -0.02 | | | 1.52E-04 | 4.80E-05 | -410.76 | -2765.66 | |
| 1.09 | 8.82 | 0.68 | 1.10 | | 0.87455248 | 1.57E-04 | | | | | 197.43 |

Backwater calc for 18 cfs

| d (ft) | A (ft2) | V (ft/s) | E (ft) | delta E | R (ft) | Sf | Avg -Sf | So-Avg Sf | dl (ft) | Cum Dist | Elev. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2.38 | 22.32 | 0.81 | 2.39 | | 1.6257616 | 1.07E-05 | | | | | 198.72 |
| | | | | -0.05 | | | 1.11E-05 | 1.89E-04 | -261.79 | -261.79 | |
| 2.33 | 21.74 | 0.83 | 2.34 | | 1.59959704 | 1.15E-05 | | | | | 198.67 |
| | | | | -0.05 | | | 1.20E-05 | 1.88E-04 | -262.78 | -524.57 | |
| 2.28 | 21.16 | 0.85 | 2.29 | | 1.57325399 | 1.24E-05 | | | | | 198.62 |
| | | | | -0.05 | | | 1.29E-05 | 1.87E-04 | -263.89 | -788.46 | |
| 2.23 | 20.58 | 0.87 | 2.24 | | 1.54672675 | 1.34E-05 | | | | | 198.57 |
| | | | | -0.05 | | | 1.40E-05 | 1.86E-04 | -265.13 | -1053.59 | |
| 2.18 | 20.01 | 0.90 | 2.19 | | 1.52000941 | 1.46E-05 | | | | | 198.52 |
| | | | | -0.05 | | | 1.52E-05 | 1.85E-04 | -266.51 | -1320.10 | |
| 2.13 | 19.45 | 0.93 | 2.14 | | 1.49309576 | 1.58E-05 | | | | | 198.47 |
| | | | | -0.05 | | | 1.65E-05 | 1.84E-04 | -268.07 | -1588.17 | |
| 2.08 | 18.89 | 0.95 | 2.09 | | 1.46597933 | 1.72E-05 | | | | | 198.42 |
| | | | | -0.05 | | | 1.79E-05 | 1.82E-04 | -269.83 | -1858.00 | |
| 2.03 | 18.33 | 0.98 | 2.04 | | 1.43865339 | 1.87E-05 | | | | | 198.37 |
| | | | | -0.05 | | | 1.95E-05 | 1.80E-04 | -271.82 | -2129.82 | |
| 1.98 | 17.78 | 1.01 | 2.00 | | 1.41111086 | 2.04E-05 | | | | | 198.32 |
| | | | | -0.05 | | | 2.13E-05 | 1.79E-04 | -274.09 | -2403.90 | |
| 1.93 | 17.23 | 1.04 | 1.95 | | 1.38334438 | 2.23E-05 | | | | | 198.27 |
| | | | | -0.05 | | | 2.33E-05 | 1.77E-04 | -276.68 | -2680.58 | |
| 1.88 | 16.69 | 1.08 | 1.90 | | 1.35534624 | 2.44E-05 | | | | | 198.22 |
| | | | | -0.05 | | | 2.56E-05 | 1.74E-04 | -279.67 | -2960.25 | |
| 1.83 | 16.16 | 1.11 | 1.85 | | 1.32710835 | 2.68E-05 | | | | | 198.17 |
| | | | | -0.05 | | | 2.81E-05 | 1.72E-04 | -283.13 | -3243.38 | |
| 1.78 | 15.63 | 1.15 | 1.80 | | 1.29862228 | 2.94E-05 | | | | | 198.12 |
| | | | | -0.05 | | | 3.10E-05 | 1.69E-04 | -287.17 | -3530.55 | |
| 1.73 | 15.10 | 1.19 | 1.75 | | 1.26987915 | 3.25E-05 | | | | | 198.07 |

**Conclusion**

The cumulative distance to transition from known flow depth of 1.31 feet (6 cfs) at the Turkey Creek Golf Club culvert to a calculated normal depth of 1.09 feet (6 cfs) results in a length of 2,765.66 feet. The proposed fish screen will be located approximately 790 feet upstream of the culvert. Therefore, normal depth will not be reached, and flow at the outlet of the fish screen will be controled by the flow conditions in the culvert. The calculated depth at normal flow of 6 cfs at the outlet from the fish screen is 1.23 feet. For the max flow of 18 cfs, the depth at the outlet from the fish screen is 2.23 ft.

McMILLEN JACOBS ASSOCIATES

| | | | |
|---|---|---|---|
| **SUBJECT:** | Nevada Irrigation District - NID | **BY:** J. Burgi | **CHK'D BY:** M.Cerucci |
| | Hemphill Diversion Project | **DATE:** 1/19/2022 | |
| | Roughened Channel - Rock sizing | **PROJECT NO.:** 21-125 | |

## Purpose

The purpose of this calcualtion sheet is to compare different methods of calculating D50 based on CDFW XII methods for rock ramps and the Bureau of Reclemation Rock Ramp sizing methods.

## References


- *CDFW. (2009). California Salmonid Stream Habitat Restoration Manual - Part XII Fish Passage and Implementation. CDFW.*
- *USBR. (2007). Rock Ramp Design Guidelines. Denver: U.S. Department of Interior, Bureau of Reclamation*


## Information - Input

| | | | |
|---|---|---|---|
| $Q_{MAX}$ = | 5000 | $ft^3/s$ | Estimated bank full flow |
| $Q_{100\ YR}$ = | 15000 | $ft^3/s$ | From FEMA |
| Channel $Width_{MAX}$ = | 90 | ft | Bank full width |
| Channel $Width_{100\ YR}$ = | 500 | ft | Approximate floodplain width |
| S = | 0.04 | ft/ft | Roughened Channel Slope |
| $q_{MAX}$ = | 55.56 | $ft^2/s/ft$ | |
| $q_{100\ YR}$ = | 30.00 | $ft^2/s/ft$ | |

## Calculation

**CDFW XII**
**Equation XII-I ACOE(1994)**

$$D_{30-ACOE}=\frac{1.95*S^{0.555}*(1.25q)^{\frac{2}{3}}}{g^{\frac{1}{3}}}$$

$$D_{84-ESM}=1.5*D_{30-ASCOE}$$

$$D_{50-ESM}=0.4*D_{84-ESM}$$

Where:

$D_{30\text{-}ACOE}$ = D30 stable particle size based on rock gradation provided by ACOE 1994 (ft)
S = Hydraulic slope (ft/ft)
q = unit discharge within active channel at stable bed design flow (cfs/ft)
g = gravitation acceleration (32.2 $ft/s^2$)

**Max**

| | | |
|---|---|---|
| S = | 0.04 | ft/ft |
| $q_{MAX}$ = | 55.56 | $ft^3/s/ft$ |
| g = | 32.2 | $ft/s^2$ |
| $D_{30\text{-}ACOE\ MAX}$ = | 1.735 | ft |
| $D_{84\text{-}ESM\ MAX}$ = | 2.603 | ft |
| $D_{50\text{-}ESM\ MAX}$ = | 1.041 | ft |

**100 Year**

| | | |
|---|---|---|
| S = | 0.04 | |
| $q_{100\ YR}$ = | 30.00 | ft3/s/ft |
| g = | 32.2 | $ft/s^2$ |
| $D_{30\text{-}ACOE\ 100\ YR}$ = | 1.151 | ft |
| $D_{84\text{-}ESM\ 100\ YR}$ = | 1.726 | ft |
| $D_{50\text{-}ESM\ 100\ YR}$ = | 0.690 | ft |

**BOR**
**Abt and Johnson (1991) Equation 4-2**

$$q_{sizing}=1.35*q_{design}$$

$$D_{50}=\varphi_e*\varphi_c*a*5.23*S^{0.43}q_{sizing}{}^{0.56}$$

Where:

$D_{50}$ = $D_{50}$ median diameter of rock layer (in)
ϕe = coefficient for empirical envelope on the regression relationship =1.2
ϕc = coefficient of flow concentration due to channelization within revetment
a = shape factor for rounded versus angular material
S = profile slope of rock ramp (ft/ft)
$q_{sizing}$ = design unit discharge ($ft^3/s/ft$)

**Max**

| | | | |
|---|---|---|---|
| $q_{Sizing\ MAX}$ = | 75 | $ft^3/s/ft$ | |
| ϕe = | 1.2 | | |
| ϕc = | 1.2 | | assuming sheet flow |
| a = | 1.4 | | rounded material |
| S = | 0.04 | ft/ft | |
| $D_{50\ MAX}$ = | 29.64 | in | |
| $D_{50\ MAX}$ = | 2.47 | ft | |

**100 Year**

| | | | |
|---|---|---|---|
| $q_{Sizing\ 100\ YR}$ = | 40.5 | $ft^3/s/ft$ | |
| ϕe = | 1.2 | | |
| ϕc = | 1.2 | | assuming sheet flow |
| a = | 1.4 | | rounded material |
| S = | 0.04 | ft/ft | |
| $D_{50\ 100\ YR}$ = | 20.99 | in | |
| $D_{50\ 100\ YR}$ = | 1.75 | ft | |

**BOR**
**Ullmann (2000) Equation 4-5**

$$q_{sizing}=1.35*q_{design}$$

$$D_{50}=6.84*S^{0.43}*q^{0.56}*C_u^{0.25}*(1.12*R+0.39)$$

Where:

$D_{50}$ = $D_{50}$ median diameter of rock layer (in)
S = profile slope of rock ramp (ft/ft)
$q_{sizing}$ = design unit discharge ($ft^3$/s/ft)
$C_u$ = Coefficient of uniformity, $D_{60}/D_{10}$
R = percent roundness in decimal form

| Max | | |
|---|---|---|
| $q_{Sizing\ MAX}$ = | 75 | $ft^3$/s/ft |
| S = | 0.04 | ft/ft |
| $C_u$ = | 2.4 | |
| R = | 0.7 | |
| $D_{50\ MAX}$ = | 28.10 | in |
| $D_{50\ MAX}$ = | 2.34 | ft |

| 100 Year | | |
|---|---|---|
| $q_{Sizing\ 100\ YR}$ = | 40.5 | $ft^3$/s/ft |
| S = | 0.04 | ft/ft |
| $C_u$ = | 2.4 | |
| R = | 0.7 | |
| $D_{50\ 100\ YR}$ = | 19.90 | in |
| $D_{50\ 100\ YR}$ = | 1.66 | ft |

**BOR**
**Ferro (1999) Equation 4-6**

$$D_{50}=B*(\varphi_e*\frac{0.95}{(\sigma_g^2)^{0.562}}*(\frac{Q*S}{B^{\frac{5}{2}}*g^{\frac{1}{2}}}*\frac{\gamma_s-\gamma}{\gamma})^{\frac{1}{2}}$$

Where:

$D_{50}$ = $D_{50}$ median diameter of rock layer (in)
S = profile slope of rock ramp (ft/ft)
Q = total discharge ($ft^3$/s)
ϕe = coefficient for empirical data in regression relationship =1.4
$\sigma_g^2$ = geometric variance of gradation, $D_{84}/D_{16}$
g = gravitation acceleration (32.2 ft/$s^2$)
$\gamma_s$ = specific weight of stone (lbs/$ft^3$)
$\gamma$ = specific weight of water (lbs/$ft^3$)

| Max | | |
|---|---|---|
| B = | 90 | ft |
| S = | 0.04 | ft/ft |
| Q = | 5000 | $ft^3$/s |
| ϕe = | 1.4 | |
| $\sigma_g^2$ = | 4 | |
| g = | 32.2 | ft/$s^2$ |
| $\gamma_s$ = | 156.075 | lbs/$ft^3$ |
| $\gamma$ = | 62.43 | lbs/$ft^3$ |
| $D_{50\ MAX}$ = | 1.441 | ft |

| 100 Year | | |
|---|---|---|
| B = | 500 | ft |
| S = | 0.04 | ft/ft |
| Q = | 15000 | $ft^3$/s |
| ϕe = | 1.4 | |
| $\sigma_g^2$ = | 4 | |
| g = | 32.2 | ft/$s^2$ |
| $\gamma_s$ = | 156.075 | lbs/$ft^3$ |
| $\gamma$ = | 62.43 | lbs/$ft^3$ |
| $D_{50\ 100\ YR}$ = | 1.625 | ft |

**BOR**
**Robinson et al. (1998) Equation 10-6**

$$q_{sizing}=1.35*q_{design}$$

$$D_{50}=(\frac{q_{sizing}}{9.76*10^{-7}*S^{-1.50}})^{\frac{1}{1.89}}$$

Where:

$D_{50}$ = $D_{50}$ median diameter of rock layer (in)
S = profile slope of rock ramp (ft/ft)
$q_{sizing}$ = design unit discharge ($ft^3$/s/ft)

| Max | | |
|---|---|---|
| S = | 0.04 | $ft^3$/s/ft |
| $q_{sizing}$ = | 75 | ft/ft |
| $D_{50\ MAX}$ = | 328.69 | mm |
| $D_{50\ MAX}$ = | 12.94 | in |
| $D_{50\ MAX}$ = | 1.08 | ft |

| 100 Year | | |
|---|---|---|
| S = | 0.04 | $ft^3$/s/ft |
| $q_{sizing}$ = | 40.50 | ft/ft |
| $D_{50\ 100\ YR}$ = | 237.242 | mm |
| $D_{50\ 100\ YR}$ = | 9.340 | in |
| $D_{50\ 100\ YR}$ = | 0.778 | ft |

**BOR**
**USACE Bed (1991) Equation 4-8 and 4-9**

$$D_{30}=\frac{1.95*S^{0.555}*(1.25q)^{\frac{2}{3}}}{g^{\frac{1}{3}}}$$

$$D_{50}=D_{30}*(\frac{D_{85}}{D_{15}})^{\frac{1}{3}}$$

Where:

$D_{30}$ = Rock diameter for which 30% is smaller by mass (ft)
S = Slope of rock ramp (ft/ft)
q = unit discharge within active channel at stable bed design flow (cfs/ft)
g = gravitation acceleration (32.2 ft/$s^2$)
$D_{85}$ = Rock diameter for which 85% is smaller by mass (ft)
$D_{15}$ = Rock diameter for which 85% is smaller by mass (ft)

| Max | | |
|---|---|---|
| S = | 0.04 | ft/ft |
| $q_{MAX}$ = | 55.56 | $ft^3$/s/ft |
| g = | 32.2 | ft/$s^2$ |
| $D_{30\ MAX}$ = | 1.735 | ft |
| $D_{85}/D_{15}$ = | 2.7 | |
| $D_{50\ MAX}$ = | 2.416 | ft |

| 100 Year | | |
|---|---|---|
| S = | 0.04 | |
| $q_{100\ YR}$ = | 30.00 | ft3/s/ft |
| g = | 32.2 | ft/$s^2$ |
| $D_{30\ 100\ YR}$ = | 1.151 | ft |
| $D_{85}/D_{15}$ = | 2.7 | |
| $D_{50\ 100\ YR}$ = | 1.602 | ft |

**Conclusion**

| Reference | Equation | $D_{50}$ (ft) | | $D_{50}$ (in) | |
|---|---|---|---|---|---|
| | | Max | 100 Yr | Max | 100 Yr |
| CDFW XII | Equation XII-I ACOE(1994) | 1.04 | 0.69 | 12.49 | 8.28 |
| BOR | Abt and Johnson (1991) Equation 4-2 | 2.47 | 1.75 | 29.64 | 20.99 |
| BOR | Ullmann (2000) Equation 4-5 | 2.34 | 1.66 | 28.10 | 19.90 |
| BOR | Ferro (1999) Equation 4-6 | 1.44 | 1.63 | 17.29 | 19.50 |
| BOR | Robinson et al. (1998) Equation 10-6 | 1.08 | 0.78 | 12.94 | 9.34 |
| BOR | USACE Bed (1991) Equation 4-8 and 4-9 | 2.42 | 1.60 | 28.99 | 19.23 |
| | Average | 1.80 | 1.35 | | |

The CDFW rock sizing equation was compared with five other rock sizing equations for both the 100-year flow as defined by FEMA and a "max channel" flow estimating the maximum flow at bankfull flow. Due to the spread of the water for the 100-yr flow, the channel velocities may be lower than that of the bankfull flow. As a result the rock sizes for the max flow are greater than those for the 100-yr.

The D50's ranged from 2.47' - 1.04' with the CDFW method returning the smallest rock. The average rock size for the max flow is 1.8' (22 inches). Final rock size will be determined for the 90% design package.