FEASIBILITY STUDY| DECEMBER 2025

STORMWATER RECHARGE
LOS MOLINOS SUBBASIN

PREPARED FOR

TEHAMA COUNTY FCWCD

PREPARED BY

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1. INTRODUCTION

The enactment of Water Code §1242.1 in 2023 provided an avenue to divert stormwater for recharge without obtaining appropriative water rights. Stormwater provides a cost-effective water source to be utilized for recharge. Stormwater diversions have the added benefit of reducing flooding from excess flows. This feasibility study summarizes four technical memorandums (TMs) (**Attachments A – D**) documenting watershed analyses conducted on four different creeks within the Los Molinos Subbasin. The TMs aimed to establish stormflow thresholds to be utilized by the County to determine diversion windows during storm events. In addition to determining flood flow threshold, the TMs also identified locations impacted by flooding that would benefit from stormwater diversions.

2. CREEKS

The creeks analyzed as part of the watershed analysis were Deer Creek, Mill Creek, Antelope Creek, and Cottonwood Creek. All four creeks are located within the Los Molinos Subbasin and experience flows high enough to allow for diversions. These creeks are depicted on Figure 1.

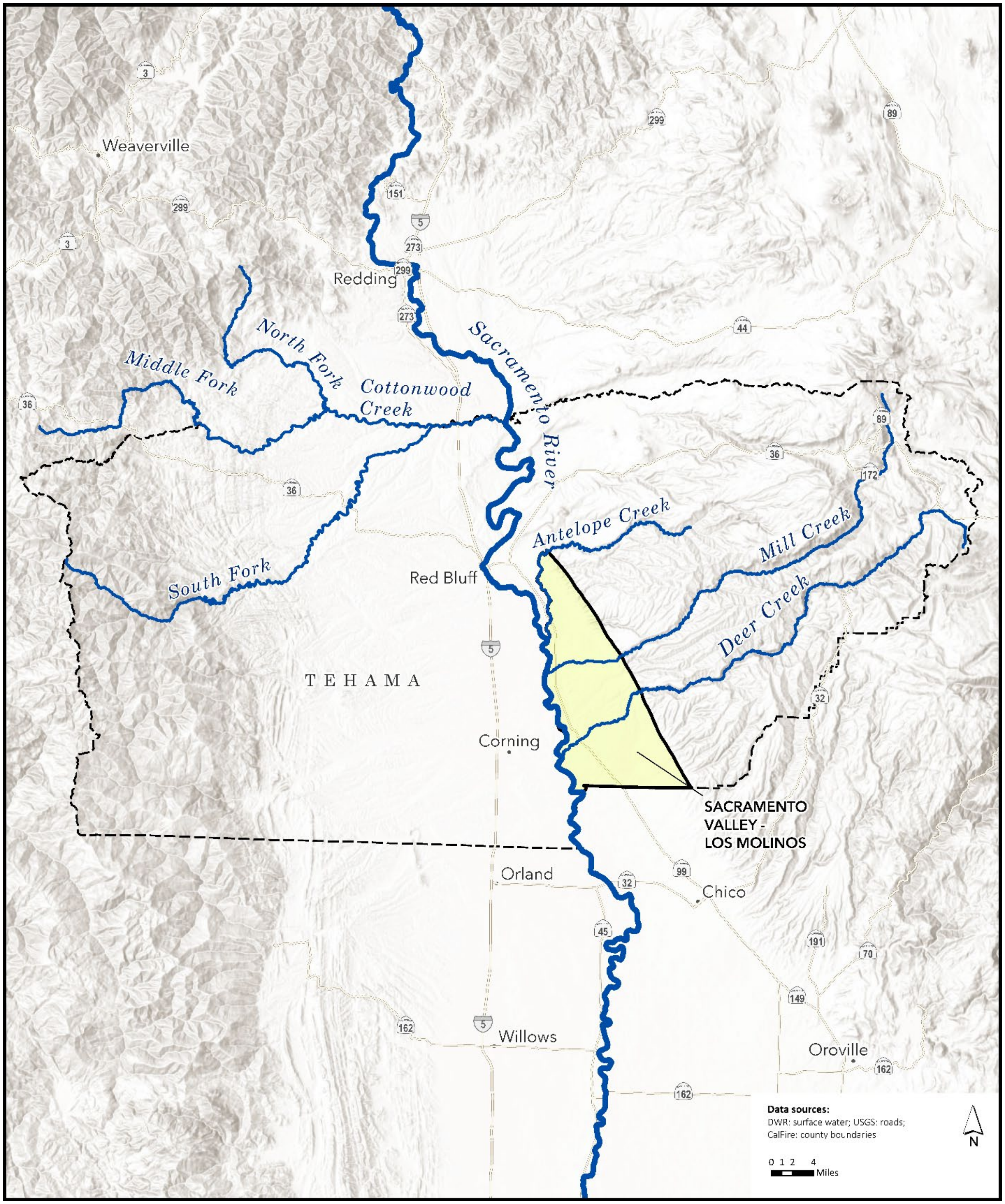

Figure 2 Location of Creeks and Sacramento River

2.1. Deer Creek

Deer Creek converges with the Sacramento River after passing through Los Molinos Subbasin. The land surrounding the creek is primarily pasture land or covered with deciduous fruit or nut crops as per the

2023 DWR land use survey. Currently, there are six claimed[1] and one licensed[2] water right(s) on Deer Creek within the Los Molinos Subbasin.

2.2. Mill Creek

Mill Creek converges with the Sacramento River after flowing through the center of the Los Molinos Subbasin. The land surrounding the creek is primarily used for pasture as per the 2023 DWR land use survey. Currently, there are five claimed and three licensed water rights on Mill Creek within the Los Molinos Subbasin.

2.3. Antelope Creek

Antelope Creek converges with the Sacramento River after across the northern portion of the Los Molinos Subbasin. The land surrounding the creek is primarily pasture land or covered with deciduous fruit or nut crops as per the 2023 DWR land use survey. Currently, there is one claimed water right on the Little Antelope Creek portion of Antelope Creek within the Los Molinos Subbasin.

2.4. Cottonwood Creek

Cottonwood Creek converges with the Sacramento River north of the Los Molinos Subbasin. The land surrounding the creek is primarily pasture land or covered with deciduous fruit or nut crops as per the 2023 DWR land use survey.. Currently, there are no active water rights on Cottonwood Creek within the Los Molinos Subbasin.

3. MODELS

Hydrologic and hydraulic modeling was conducted using the U.S. Army Corps of Engineers' HEC-HMS (Hydrologic Engineering Center - Hydrologic Modeling System) and HEC-RAS (Hydrologic Engineering Center - River Analysis System) software to evaluate watershed behavior, channel hydraulics, and flood response under various storm conditions. HEC-HMS provides a realistic representation of watershed behavior under both existing and modified conditions, supporting the assessment of watershed response and potential flood conditions.

HEC-HMS was used to simulate watershed hydrology by representing precipitation, infiltration, runoff generation, and flow routing to characterize how water moves across the landscape. The modeling effort focused on generating accurate flow hydrographs for specific storm events, capturing the timing and magnitude of runoff entering the stream system. These hydrographs, along with other hydrologic outputs such as runoff distributions, form the foundation for subsequent hydraulic analysis.

[1] Claimed - An active claim of water diversion and use under riparian or pre-1914 rights. May also be used to report water diversion and use while an appropriative right or registration is pending

[2] Licensed - Permitted Water Right has been issued a Water Right license

The hydrologic outputs from HEC-HMS were input into HEC-RAS to generate detailed hydraulic predictions. The HEC-RAS analysis is to evaluated how flow conditions generated by HEC-HMS translated into on-the-ground hydraulic impacts, particularly under different storm scenarios.

4. RESULTS

4.1. Hydrologic and Hydraulic Analysis

Using the HEC-HMS and HEC-RAS models, hydrologic and hydraulic conditions were simulated under various storm scenarios to determine diversion thresholds, assess flood impacts, and estimate recharge potential. The following sections summarize the key findings.

4.2. Diversion Thresholds and Flood Mitigation

Deer Creek

The hydrologic and hydraulic modeling established a flood diversion threshold of 13,574 cubic feet per second (cfs), corresponding to the 10-year storm event at USGS gage 11383500. This threshold was derived from a flood frequency analysis using 109 years of annual peak flow data. Flows exceeding this threshold pose significant flood risks to properties and infrastructure within the Lower Deer Creek Watershed.

Modeling results indicate that diversions at the DCID Deer Creek Diversion and Cone Kimball Dam can effectively reduce peak flows during major storm events. For a 25-year event, diverting 3,272 acre-feet (ac-ft) of water would lower water levels at the Highway 99 Bridge by approximately 1.2 feet, while a 50-year event would require 7,620 ac-ft of diversion to achieve a 2-foot reduction in water depth. These reductions significantly decrease flood hazards and demonstrate the potential of diversion operations to protect vulnerable communities during high-flow conditions.

Mill Creek

The flood diversion threshold was established at 12,222 cfs, corresponding to the 10-year storm event recorded at USGS gage 11381500. Flows exceeding this threshold pose an imminent flooding threat to properties along Mill Creek, particularly near Ellis Street. This threshold serves as a critical benchmark for initiating diversions during high-flow events to minimize flood damage.

Historical data highlight the importance of this threshold. The model predicted that an event like the 2024 flood event, which had an approximate 19-year recurrence interval, could cause significant impacts including road closures, property damage, and restricted access along the north bank of Mill Creek downstream of Highway 99.

To mitigate such risks, the study evaluated diversion strategies aimed at reducing peak flows. Lowering the 2024 flood peak of 15,400 cfs to the 10-year threshold of 12,222 cfs would require diverting approximately 3,178 cfs of water. This diversion could be achieved through coordinated operations at the Upper Diversion Dam and Ward Diversion Dam, with each facility handling half of the required volume.

Implementing this strategy would lower water levels at the Highway 99 Bridge by approximately 1.1 feet, significantly reducing flood hazards in the affected areas.

Antelope Creek

The hydrologic and hydraulic modeling established a flood diversion threshold of 5,125 cfs, corresponding to the 2-year storm event at the USGS gage. The modelling predicted that flows exceeding this threshold, such as during the November 2024 event, which had an approximate 5-year recurrence interval and a peak flow of 8,879 cfs, could cause flooding in the west floodplain of Antelope Creek downstream of Edwards Diversion Dam. The model also predicted that this event could result in road closures, property damage, and restricted access near Highway 99 and Craig Avenue.

Modeling results indicated that diversions at four proposed points could effectively reduce water levels during high-flow events. For example, during the November 2024 event, diversions would have lowered water depth at the Cone Grove Road Bridge by approximately 0.9 feet, significantly reducing flood hazards for residential properties and transportation routes.

Cottonwood Creek

The hydrologic and hydraulic modeling established a flood diversion threshold of 38,000 cfs at the CNRFC gage CWAC1, which corresponds to a 5-year flood event. This threshold was selected to reduce flooding in the High-Risk Flood-Prone Zone, where inundation is predicted to begin once flows exceed this level. The model predicted that roads and properties such as Main Street, Evergreen Road Bridge, and residences at 18700 and 18750 Evergreen Road would experience significant flood impacts when flows surpass the flood diversion threshold.

Modeling results indicated that diversions upstream of the High-Risk Flood-Prone Zone can substantially reduce flood hazards. For example, during the 2023 event, diverting 2,516 cfs from the South Fork Cottonwood Creek over a 24-hour period would have lowered water levels by approximately 0.3 foot in the flood-prone zone. This reduction would help prevent road closures and property damage, demonstrating the effectiveness of diversion operations in mitigating flood impacts during high-flow conditions.

4.3. Groundwater Recharge Potential

Deer Creek

During the February 2025 event, which had a peak flow of 17,110 cfs corresponding to a 20-year return period, diversions could have captured substantial volumes of water for recharge purposes. To infiltrate diverted volumes efficiently, suitable recharge areas near diversion dams must be identified. For example, the infiltration rate assumed in the analysis was 0.5 feet per day, which would require hundreds of acres of land to accommodate recharge during large storm events.

Mill Creek

During the 2024 event, an estimated 330 ac-ft of water could have been captured for recharge purposes. To infiltrate this volume within a single day, approximately 660 acres of land would be required, assuming a recharge rate of 0.5 feet per day.

Antelope Creek

An estimated volume 3,587 ac-ft of water could have been diverted during the November 2024 event for recharge purposes. To infiltrate this volume within a single day, approximately 7,174 acres of land would be required, assuming a recharge rate of 0.5 feet per day. Larger storm events would yield even greater recharge opportunities; for instance, a 10-year event would provide 7,182 ac-ft, requiring 14,364 acres, while a 50-year event could generate 16,834 ac-ft, requiring 33,668 acres.

Cottonwood Creek

During the 2023 event, diverting 1,066 ac-ft of water could have been diverted for recharge. This volume would require approximately 2,132 acres of recharge area, assuming an infiltration rate of 0.5 foot per day. Larger storm events would provide even greater recharge opportunities.

4.4. Water Availability for Recharge

Deer Creek

Annual water availability for recharge was assessed using two operational approaches. Method 2 (Threat of Flood Conditions) estimated an average annual recharge volume of 481 ac-ft, reflecting the limited frequency of flood conditions, which occur in approximately 11% of years. In contrast, Method 1 (90th Percentile/20 Percent Method) yielded a much higher estimate of 10,889 ac-ft, with water available for diversion in 95% of years.

Mill Creek

Annual water availability for recharge was assessed using two methods. Under Method 2 (Threat of Flood Conditions), the average annual recharge volume was estimated at 178 ac-ft, with flood conditions occurring in only 11% of years. Conversely, Method 1 (90th Percentile/20 Percent) yielded a much higher estimate of 7,345 ac-ft, with water available for diversion in 95% of years..

Antelope Creek

Annual water availability for recharge was assessed using the 90th Percentile/20 Percent Method (Method 1), which estimated annual recharge volumes ranging from 369 to 13,416 ac-ft, with an average of 4,877 ac-ft. Historical data indicate that diversion opportunities would occur in approximately 95% of years, making this approach highly reliable for long-term water management.

Cottonwood Creek

Under Method 2 (Threat of Flood Conditions), the average annual recharge volume was estimated at 224 ac-ft, reflecting the limited frequency of flood conditions. In contrast, Method 1 (90th Percentile/20 Percent) yielded a much higher estimate of 15,000 ac-ft, with water available for diversion in approximately 95% of years.

5. SUMMARY TABLES

The following tables present a consolidated view of the key findings from the hydrologic and hydraulic analyses. Table 1 summarizes the diversion thresholds and associated return periods for each creek, while Table 2 compares annual groundwater recharge volumes estimated using two operational approaches: Method 1 (90th Percentile/20 Percent) and Method 2 (Threat of Flood Conditions). These tables provide a quick reference for evaluating flood mitigation and recharge potential across the study area.

Table 1 Summary of Diversion Thresholds by Creek

Creek	Diversion Threshold (cfs)	Return Period
Deer Creek	13,574	10-year event
Mill Creek	12,222	10-year event
Antelope Creek	5,125	2-year event
Cottonwood Creek	38,000	5-year event

Table 2 Annual Water Availability for Recharge

Creek	Method 1 (90th Percentile/20%)	Method 2 (Threat of Flood Conditions)
Deer Creek	10,889 ac-ft	481 ac-ft
Mill Creek	7,345 ac-ft	178 ac-ft
Antelope Creek	Avg. 4,877 ac-ft (range: 369–13,416)	N/A
Cottonwood Creek	15,000 ac-ft	224 ac-ft

6. CONCLUSION

The results of this analysis demonstrate that establishing a diversion threshold of 13,574 cfs for Deer Creek, 12,222 cfs for Mill Creek, 5,125 cfs for Antelope Creek and 38,000 cfs for Cottonwood Creek could substantially reduce flood risks in the Los Molinos Subbasin. Furthermore, utilizing diverted floodwaters for groundwater recharge offers a dual benefit: mitigating flood hazards while enhancing regional water supply reliability.

To advance this work, this study recommends conducting land suitability analyses for recharge areas near potential diversion points, evaluating additional upstream diversion points, improving gage data, assessing local runoff not mitigated by channel diversions, and creating a flood diversion plan for operational planning. These measures will support the development of ranked alternatives for flood mitigation and groundwater recharge, ensuring a resilient and sustainable water management approach for Tehama County.

ATTACHMENT A

GEI Consultants

Technical Memorandum

11010 White Rock Road, Suite 200 ● Rancho Cordova, CA 95670 ● 916.631.4500

Via Email:	pdhaliwal@lsce.com, wanderson@lsce.com
To:	Will Anderson and Pavan Dhaliwal, Luhdorff & Scalmanini Consulting Engineers Tehama County Flood Control and Water Conservation District, California
From:	Bryan Thoreson; Yi Shen (GEI)
cc:	Chris Ferrari (GEI)
Date:	November 19, 2025
Re:	Hydrologic and Hydraulic Model Approach and Evaluation for the Deer Creek Groundwater Recharge and Flood Reduction Project
Project No.	2403778

1. Introduction and Purpose

Deer Creek is a 60-mile-long tributary of the Sacramento River that flows southwest through Tehama County, California. Its watershed spans 229 square miles, with the headwater originating in Lassen National Forest at an elevation of 7,320 feet. In its upper reaches, the creek winds through meadows and dense forests before descending into a steep rock canyon. As it approaches its lower course, Deer Creek enters the flat expanse of the Sacramento Valley (Figure 1).

The purpose of this Technical Memorandum (TM) is to develop a Flood Diversion Threshold to allow diversion of surface water from Deer Creek for groundwater recharge to help alleviate flooding. This TM discusses the following:

- Developing the Hydrologic (HEC-HMS)[1] and hydraulic (HEC-RAS)[2] models for Deer Creek Watershed.
- Utilizing U.S. Geological Survey (USGS) 1-m high-resolution Terrain.
- Validating/calibrating the hydrologic and hydraulic models at USGS gage 11383500 – Deer Creek near Vina, a census-designated place in Tehama County, California (USGS gage) (Figure 1).
- Analyzing hydraulic model floodplain results based on statistical 10-, 25- and 50-year storm events.
- Developing a Flood Diversion Threshold following the State Water Resources Control Board Code 1242.1 guidelines[3] (State Water Board 2025)

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[1] HEC-HMS refers to the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's (HEC) Hydrologic Modeling System (HMS) version 4.13

[2] HEC-RAS refers to USACE's Hydrologic Engineering Center's (HEC) River Analysis System (RAS) version 6.6

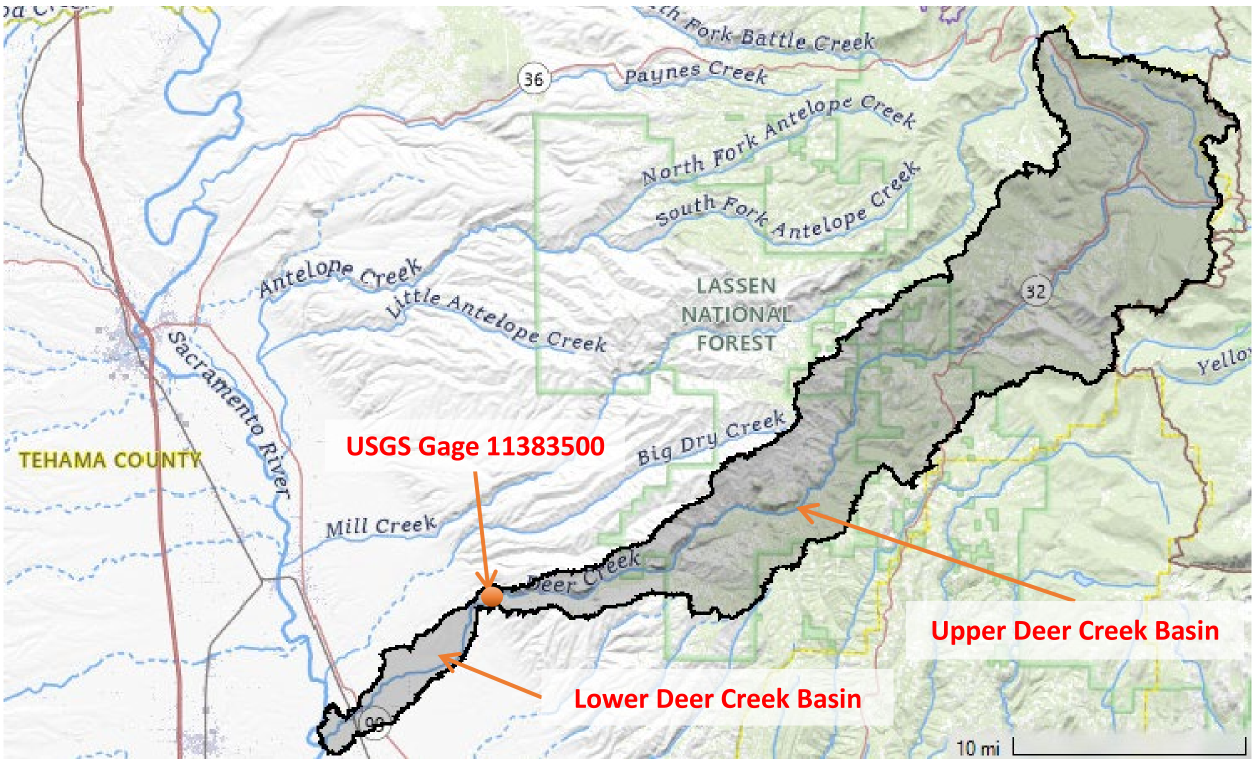
[3] https://www.waterboards.ca.gov/waterrights/water_issues/programs/groundwater-recharge/recharge-diversions.html

- Estimating potential groundwater recharge volume from diversions under State Water Board Code 1242.1 (2025) guidelines when flow is above the Flood Diversion Threshold.

As flood events become more frequent, preparing for flood diversions and groundwater recharge is crucial for sustainable water management. State Water Board Code §1242.1 (2023) allows the diversion of flood flows for groundwater recharge without a water right, provided certain conditions are met.

The goal of this TM is to describe the steps used to develop results that inform and forecast flood flows and extents for local emergency managers. This TM also evaluates project alternatives for diverting flows to recharge groundwater and presents the estimated flood reduction results.

Figure 1. Deer Creek Watershed



2. Background

The Deer Creek watershed includes numerous small tributaries in the Upper Deer Creek Basin- such as Rock, Sulphur, Wildcat, Little Pine, and Big Smoky creeks- which converge with the main channel before it enters the Sacramento Valley.

The Lower Deer Creek Basin, located downstream of USGS gage 11383500, includes the mainstem of Deer Creek and a few irrigation ditches. Below the Deer Creek Irrigation District (DCID) Diversion Dam, flows are diverted into the Cone Kimball Ditch, South Main Canal, North Main Canal, and West North Main Canal (Figure 2).

Deer Creek exhibits a natural hydrologic pattern, with high flows and peak runoff events occurring in winter, and low flows during summer and fall. The largest recorded peak runoff event occurred in

December 1937, reaching 23,800 cubic feet per second (cfs). In its lower reaches within the Sacramento Valley, three diversion dams divert water from the creek into four ditches supplying agricultural lands. Notably, there are no storage dams on the creek (Table 1).

The largest diversion structure is the Stanford-Vina Ranch Diversion Dam, which supplies irrigation water to the Stanford Vina Ranch Irrigation Company (SVRIC). According to the State Water Board, SVRIC is authorized to divert up to 15 cfs between May 1 and October 1 annually.

The only long-term streamflow monitoring station on Deer Creek is USGS gage 11383500, located approximately 9 miles upstream of its confluence with the Sacramento River. An additional gage, operated by the California Department of Water Resources California Data Exchange Center (CDEC), is located just below the Stanford-Vina Ranch Dam (gage ID: DVD), though it does not record high-flow data (Table 2).

Table 1. Diversions in the Lower Deer Creek Basin

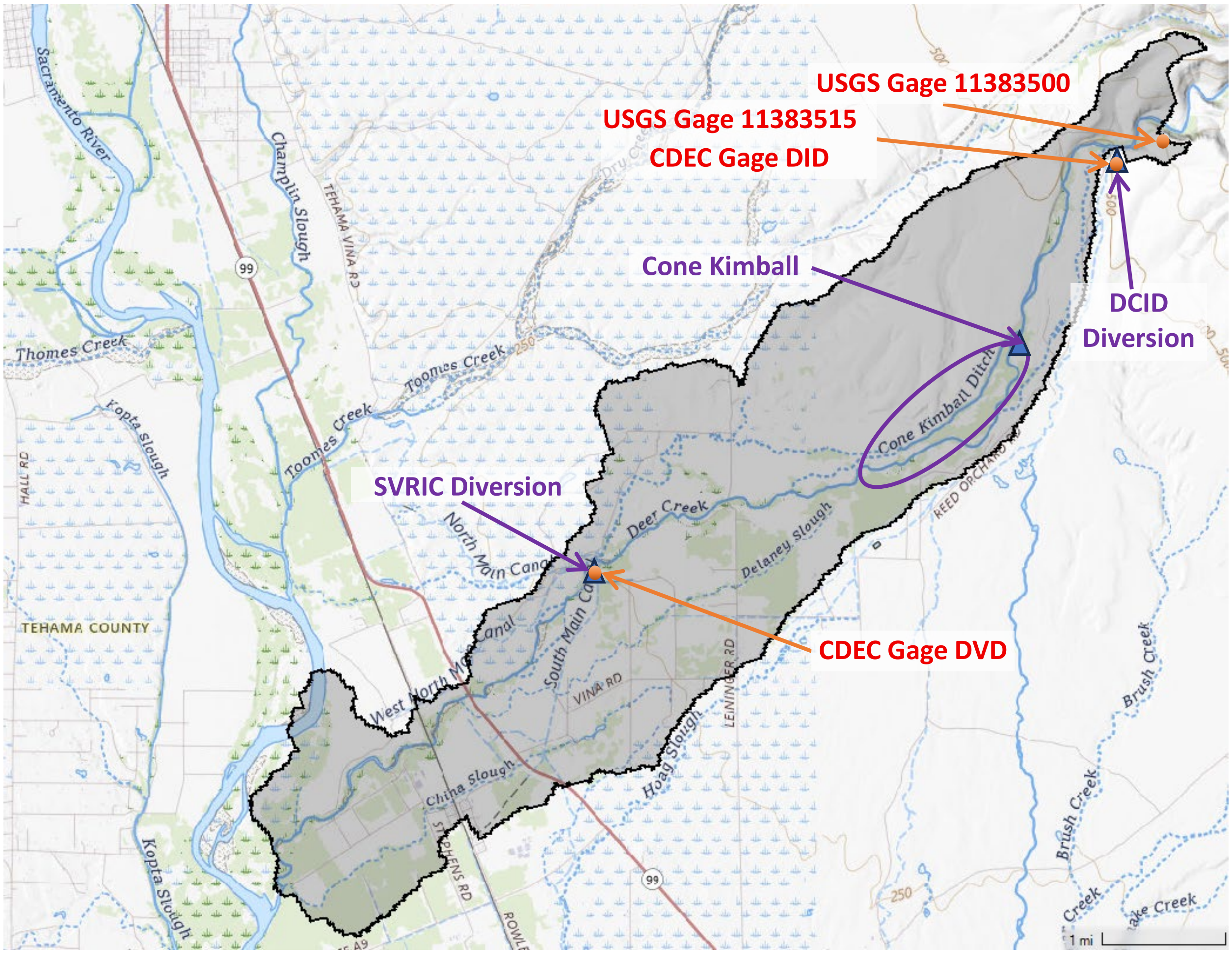
Name	Station ID	Location (Lat, Long)	Notes
Stanford Vina Ranch Irrigation Company (SVRIC) Diversion Dam	SVRIC	39.9632, -122.0342	Inline Weir
Deer Creek Irritation District (DCID) Diversion Dam, Deer Creek Diversion near Vina, CA	DCID	40.0095, -121.9572	Diversion Ditch
Cone Kimball Ditch	-	39.9909, -121.9697	Diversion Ditch
South Main Canal/North Main Canal	-	39.9632, -122.0342	Diversion Ditches

Table 2. Gages in the Lower Deer Creek Basin

Agency Name	Station ID	Gage Name	Location (Lat, Long)	Notes
USGS/CDEC/CNRFC	11383500/DCV/DCVC1	Deer Creek near Vina, CA	40.0140, -121.9482	Same gage
USGS/CDEC	11383515/DID	DCID Deer CK Diversion NR Vina, CA	40.0095, -121.9572	Measurement in Deer Creek Irrigation Ditch
CDEC	DVD	Deer Creek below Stanford Vina Dam	39.9636, -122.0344	No high flow record

Notes: CDEC = California Data Exchange Center; CNRFC - California Nevada River Forecast Center; DID = DCID Deer Creek Diversion Nr Vina; USGS = U.S. Geological Survey

Figure 2. Basins, Diversions, and Stream Gages in the Lower Deer Creek Basin



3. Hydrologic and Hydraulic Models Development

This section outlines the methodology used to develop HEC-HMS hydrologic and HEC-RAS hydraulic models for evaluating existing conditions, analyzing various diversion alternatives and utilizing gage data to forecast flood flows in Deer Creek.

3.1. HEC-HMS Model Development

The HEC-HMS hydrologic model was developed using HEC-HMS 4.13 software. A 10-meter resolution Digital Elevation Model (DEM) was downloaded from the USGS website and applied to define the hydrologic model domain. Rainfall data required for the hydrologic model was obtained from National Oceanic and Atmospheric Administration (NOAA). Specifically, NOAA Atlas 14 hypothetical design storms for 2-, 5-, 10-, 25- and 50-year return periods were downloaded in gridded format and used to represent total 24-hour rainfall amounts. The designed 24-hour temporal distribution was also sourced from NOAA.

The Deer Creek Watershed was divided into two subbasins, contributing runoff to downstream watersheds (Figure 1):

- The Upper Deer Creek Basin (208 square miles): Captures flows from the glaciated slopes of Butt Mountain down to USGS gage 11383500.
- The Lower Deer Creek Basin (21 square miles): Encompasses the area downstream of USGS gage 11383500 to the Sacramento River.

Precipitation losses due to infiltration were calculated using the SCS Curve Number loss method, developed by the Soil Conservation Service (now NRCS). This widely used empirical method estimates direct runoff based on land use, soil type, and antecedent moisture conditions. Curve Numbers were estimated for each basin based on soil drainage characteristics and land use data.

Runoff modeling was performed using the SCS Unit Hydrograph method, which requires specification of each basin's lag time (L) and Peak Rate Factor. The lag time parameter was estimated as a function of flow path length, maximum potential retention, and average watershed land slope.

Table 3 presents the model parameters used to evaluate statistical design storms and calibrate the model for historical storm events.

Table 3. HEC-HMS Hydrologic Model Parameters

Basin	Area (mi^2)	T_{lag} (min)	Peak Rate Factor	Curve Number
Upper Deer Creek	208	499	300	68
Lower Deer Creek	21	134	250	84
Total	**229**	N/A		

Notes: min = minute; mi^2 = square miles; N/A = not applicable; T_{lag} = time lag, in minutes

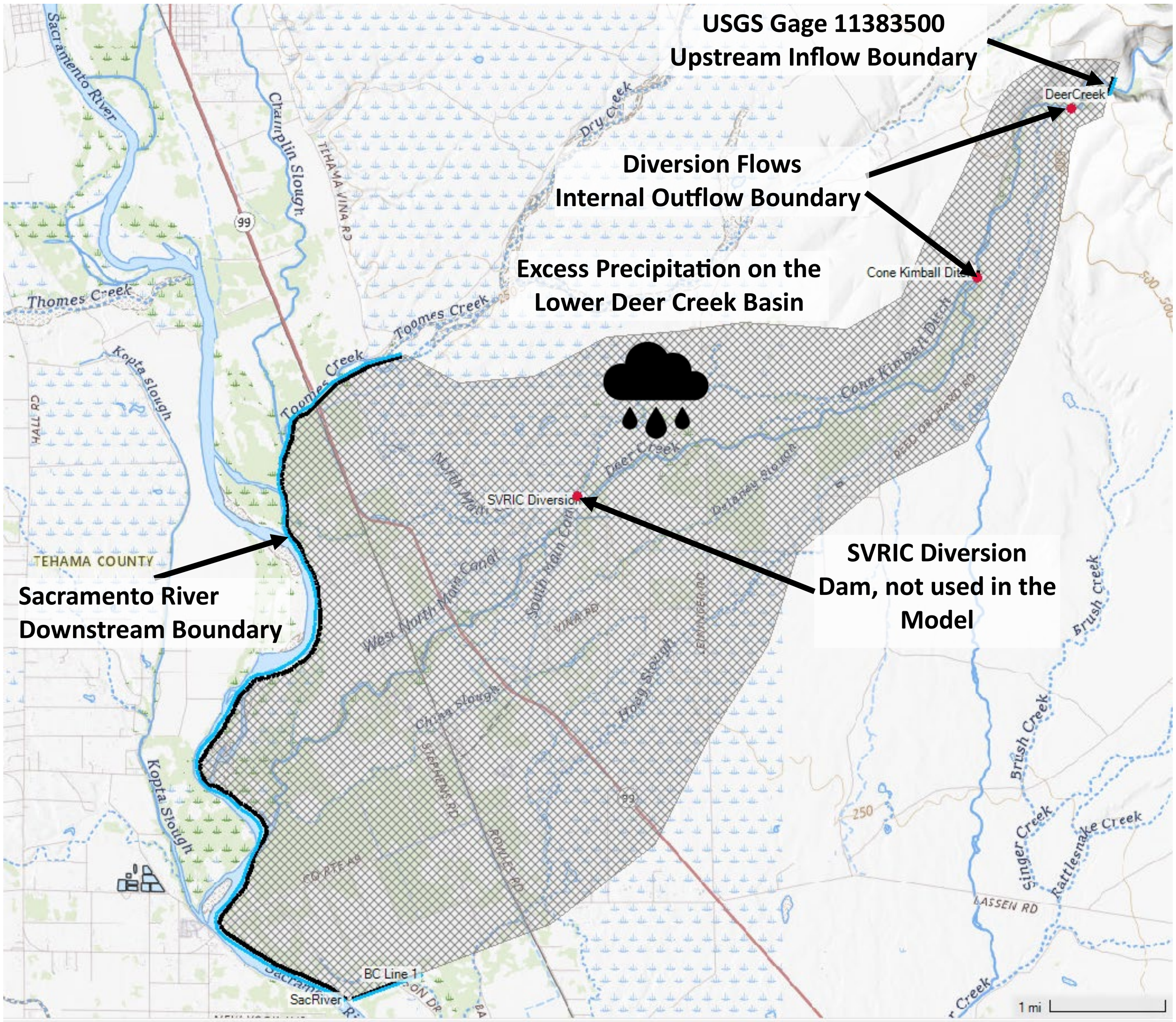
3.2. HEC-RAS Model Development

The HEC-RAS hydraulic model for this study was developed using HEC-RAS 6.6 software. A single two-dimensional (2D) mesh was created within HEC-RAS, covering only the Lower Deer Creek Basin (Figure 3), because most if not all local properties seem to be within the Lower Basin. The upstream boundary was defined at USGS gage 11383500, while the downstream boundary was set at the Sacramento River.

Outlet flow hydrographs from the Upper Deer Creek Basin, generated by the HEC-HMS model, were applied at the upstream boundary of the HEC-RAS mesh. These hydrographs were further adjusted to match the peak flow values derived from the statistical analysis of USGS gage records. A normal depth boundary condition was specified at the downstream boundary of the HEC-RAS mesh to represent flow discharge into the Sacramento River.

Additionally, using the rain-on-mesh method, precipitation excess computed by HEC-HMS for the Lower Deer Creek Basin was applied directly to the mesh. This allowed for the simulation of local run-off within both the channel and surrounding agricultural fields.

Figure 3. HEC-RAS Hydraulic Model 2D Mesh and Boundary Lines



4. Model Results and Findings:

This section discusses the evaluation of the HEC-HMS hydrologic and HEC-RAS hydraulic models:

- The HEC-HMS hydrologic model flow hydrograph was calibrated to the flow hydrographs from USGS gage 11383500 for the February 2025 event. The 2025 event is the latest major storm that impacts the watershed. The model was not calibrated to the CDEC gage DVD, because it does not completely cover the entire high flow period for this event. The HEC-HMS hydrologic model was validated to the peak flows from USGS gage 11383500 for four statistical storm events: 5-, 10-, 25-, and 50-year.
- The HEC-HMS hydrologic model was used to simulate three statistical design storms for the 10-, 25- and 50-year storm and the February 2025 historical event. Initial model parameters were calculated using formulas recommended in the HEC-HMS User Manual, with some adjustments made during the calibration process.

- The HEC-RAS hydraulic model was used to calculate stage hydrographs and flood inundation maps for all simulated events.

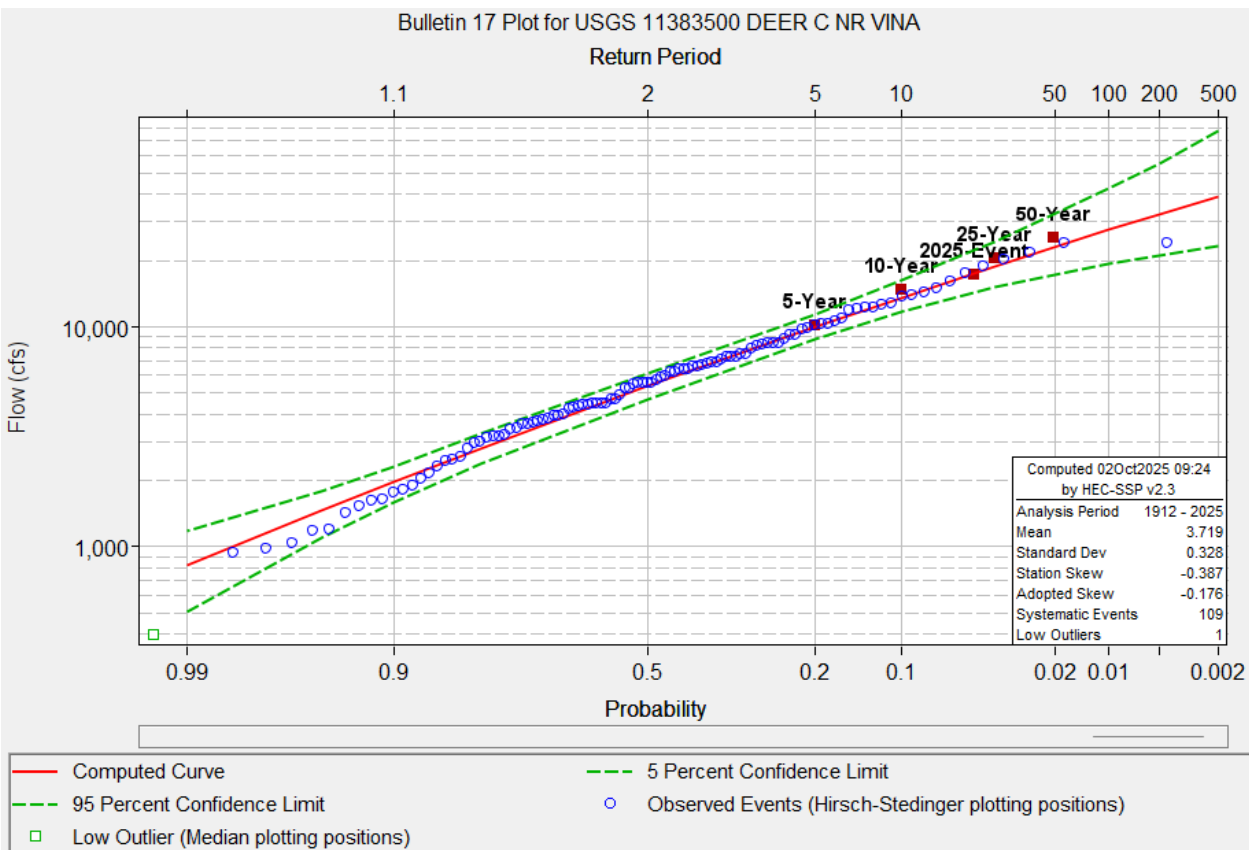
4.1. Gage Frequency Analysis

Figure 4 presents the flood frequency analysis for USGS gage 11383500 using the Hydrologic Engineering Center Statistical System Program (HEC-SS[4]) software. The computed curve was developed using the available 109 years of annual peak flow data from 1911 to 2025. Missing data from 1916 and 1920 were estimated to fall within the range of 0 to 24,000 cfs.

The red squares on Figure 4 represent peak flows from the HEC-HMS hydrologic model for the 5-, 10-, 25- and 50-year return periods, as well as the February 2025 event, allowing for direct comparison with the HEC-HMS hydrologic model software for comparison. The percentage error between the HEC-HMS hydrologic model results and USGS gage-based frequency results for the three modeled peak flow events was less than 10 percent.

Table 4 provides the peak flow estimates at USGS gage 11383500 based on the HEC-SSP analysis.

Figure 4. Flood Frequency Results at USGS Gage



Source: USACE HEC-SSP 2025

[4] HEC-SSP refers to the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's (HEC) Hydrologic Modeling System (HMS) version 4.13.

Table 4. HEC-SSP Statistical Peak Flow Estimates on Deer Creek at USGS Gage 11383500

90% Exceedance	5-Year (cfs)	10-Year (cfs)	25-Year (cfs)	50-Year (cfs)	100-Year (cfs)
1,967	9,946	13,574	18,740	22,967	27,485

Notes: % = percentage; cfs = cubic feet per second

According to the HEC-HMS hydrologic model, the peak flow at USGS gage 11383500 for the February 2025 event was calculated at 17,110 cfs. This corresponds approximately to a 20-year return period (or a 5% annual chance storm) for the Upper Deer Creek Basin.

Table 5 presents the peak flow estimates for various return periods, as calculated by the HEC-HMS hydrologic model at the outlets of the two basins–not the USGS gage location.

Table 5. HEC-HMS Hydrologic Model Statistical Peak Flow Calculated at the two Basins of the Deer Creek Watershed

Basin	Shed Area (mi 2)	5-Year (cfs)	10-Year (cfs)	25-Year (cfs)	50-Year (cfs)
Upper Deer Creek	208	10,159	14,607	20,578	25,181
Lower Deer Creek	21	732	950	1,232	1,446

Note: cfs = cubic feet per second

4.2. HEC-HMS Hydrologic Model Verification for Design Storms

Table 6 summarizes the verification results of the HEC-HMS hydrologic model. Overall, the model demonstrated acceptable performance in predicting peak flows, with all modeled errors remaining below 10 percent when compared to gage-recorded statistics.

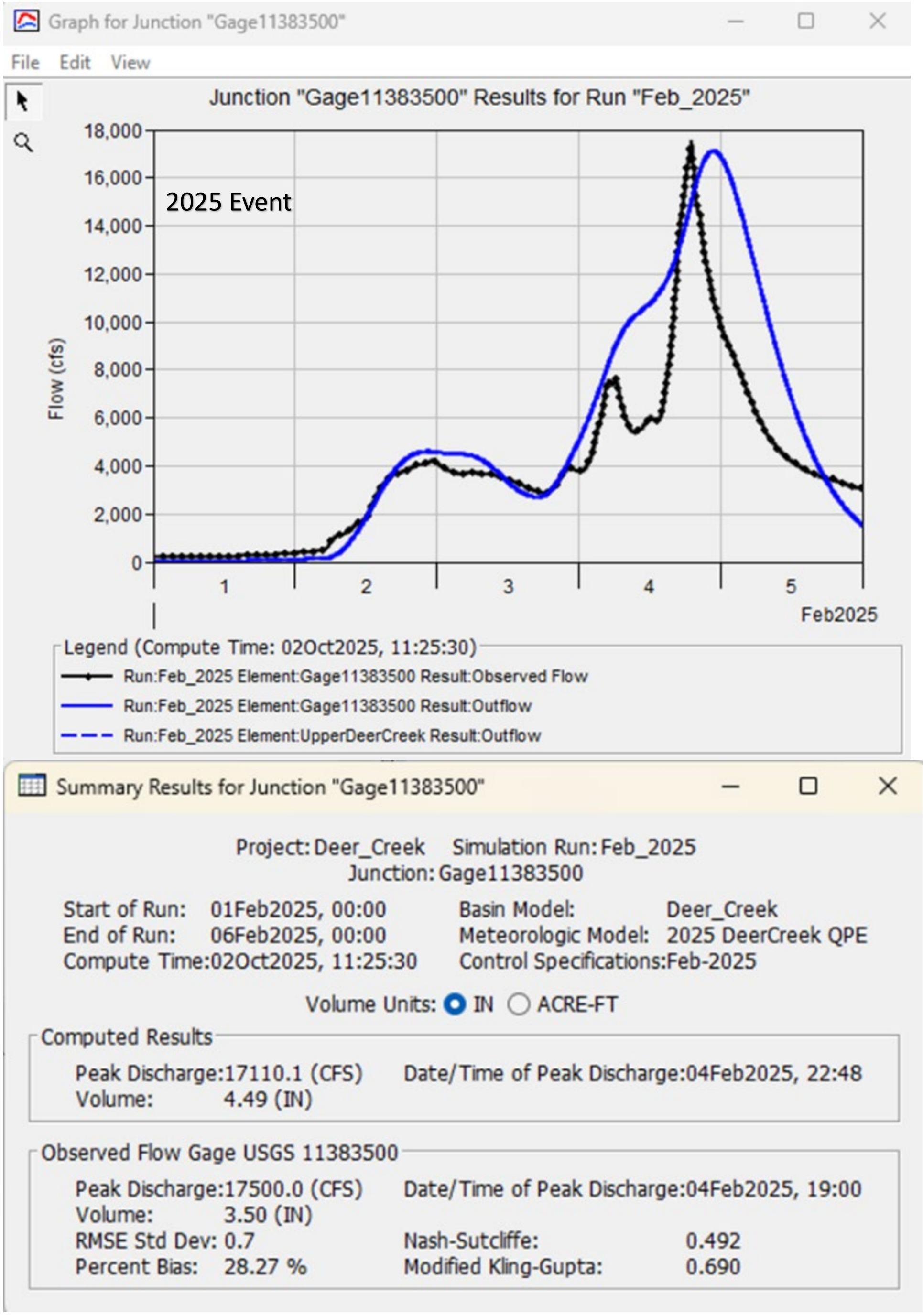
Figures 5 and 6 illustrate comparisons of the simulated and observed flow hydrographs at the outlet of the Upper Deer Watershed.

Table 6. HEC-HMS Hydrologic Model Calibration Summary at USGS Gage 11383500

	Statistical Design Storm				Historical Storm
	50-Year	25-Year	10-Year	5-Year	2025
Statistical/Observed Peak Flow (cfs)	22,967	18,740	13,574	9,946	17,500
Modeled Peak Flow (cfs)	25,181	20,578	14,607	10,159	17,110
Percentage Difference	9.6%	9.8%	7.6%	2.1%	-2.2%

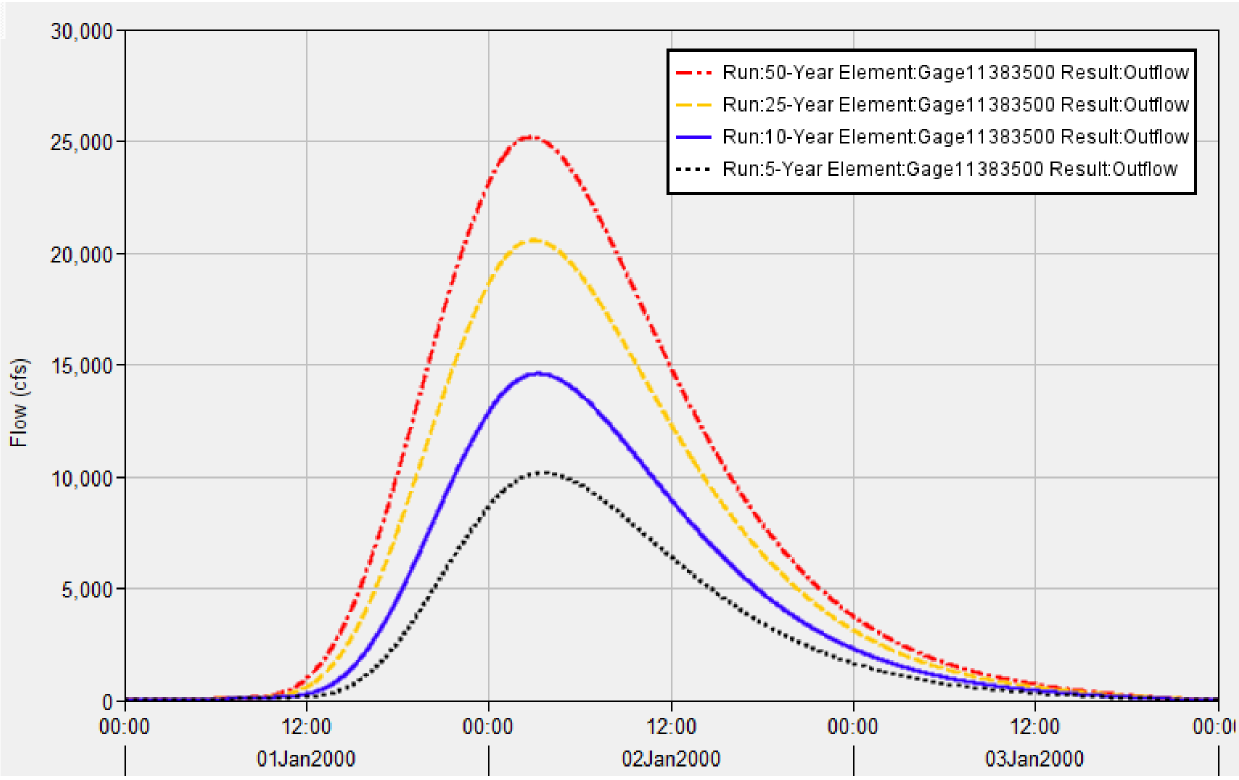
Note: cfs = cubic feet per second

Figure 5. Upper Deer Creek Basin Flow Hydrograph Comparison (February 2025 Event)



Source: USACE HEC-HMS hydrologic model 2025

Figure 6. Upper Deer Creek Basin Flow Hydrographs for Statistical Flows



Source: USACE HEC-HMS hydrologic model 2025

4.3. February 2025 Flood Event Impact on Lower Deer Creek Watershed

The February 2025 flood event was a 20-year event. The HEC-RAS hydraulic model results predicted this event could cause some road closures and property damage in Vina, CA. Figure 7 shows the comparison between the 10-year and the 2025 historical storm.

According to the State Water Board Code 1242.1 guidelines (2025), flood flows are defined as,

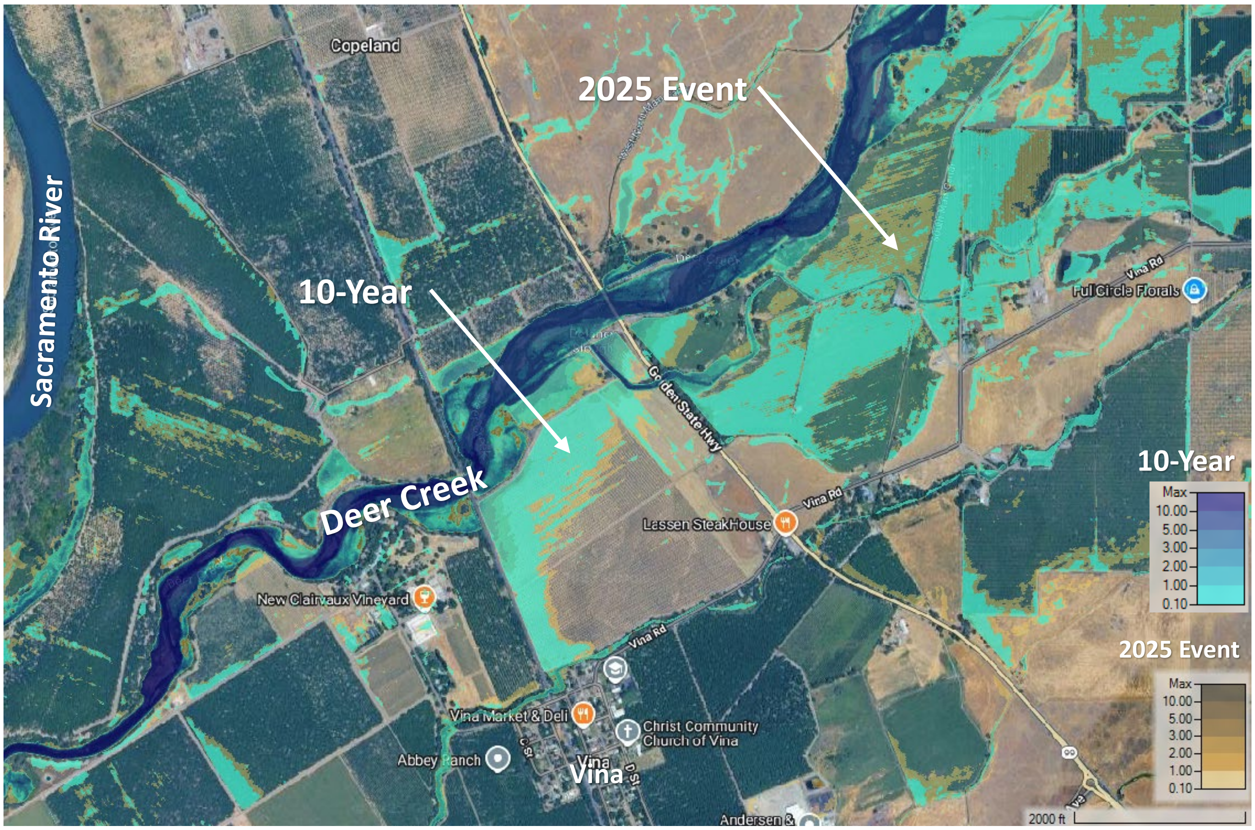
> …surface water that has escaped from or is imminently likely to escape from a channel or water body causing or threatening to cause inundation of residential or commercial structures, or roads needed for emergency response.

The 10-year event maximum inundation map shows no impact to structures or roads critical to emergency services, aside from minor flooding on agricultural lands. In contrast, during the February 2025 event, the maximum inundation map shows one residential property located north of 7th Street in Vina, CA, along China Slough, would be affected by floodwaters reaching depths of up to 0.6 feet (Figure 8). Roads potentially impacted and relevant to emergency response include Golonka Lane and Leininger Road.

Figures 7 and 8 indicate that flooding on the Lower Deer Creek Watershed would occur during the February 2025 event (a 20-year event), but not during a 10-year event. In other words, floods with a return period of greater than 10 years are likely to cause property inundation or road closures.

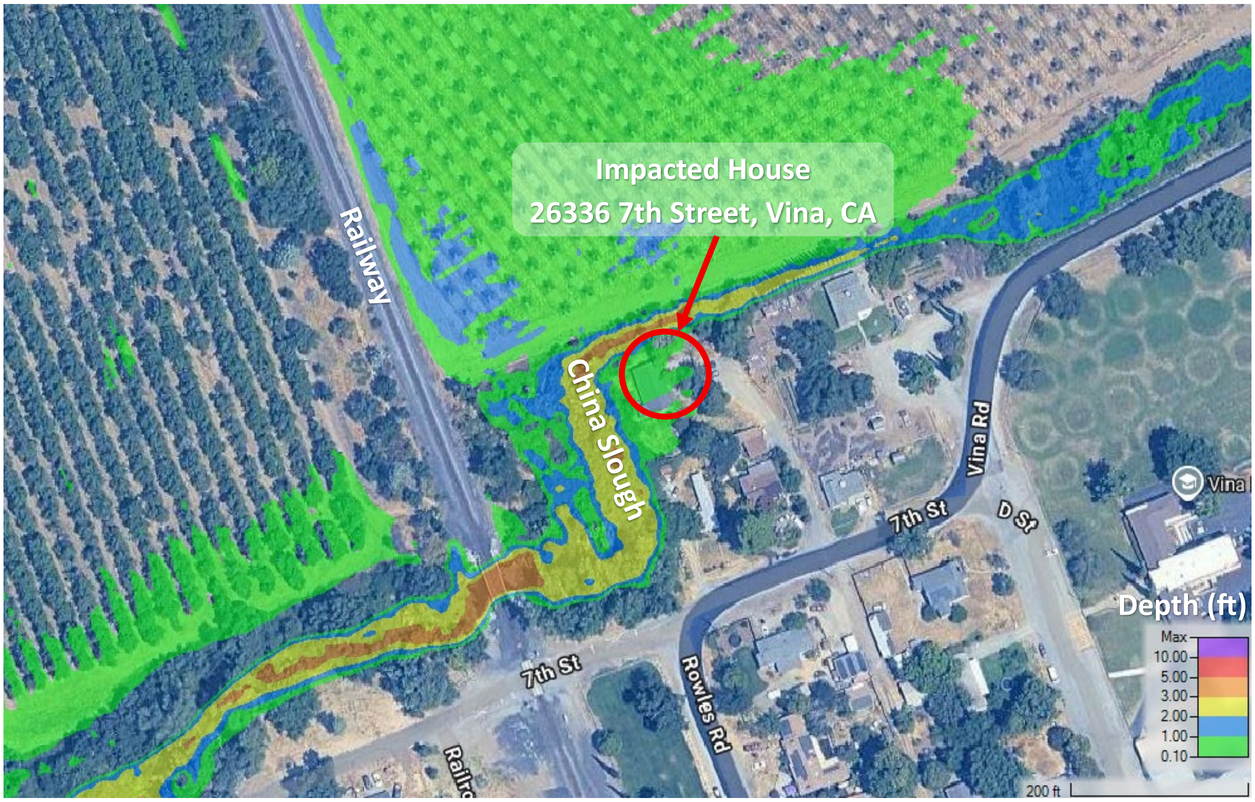
Assuming flood statistics from USGS gage 11383500 are representative of the entire Lower Deer Creek Watershed, inundation of residential structures becomes imminent when flows exceed 13,574 cfs, as observed during the February 2025 event. Therefore, the flood diversion threshold was set at the peak flow 13,574 cfs, corresponding to the 10-year event (10% annual chance) at USGS gage 11383500 (also known as CNRFC gage DCVC1).

Figure 7. Max Depth Inundation Comparison Between 10-Year (blue) and the February 2025 Event (brown)



Source: USACE HEC-RAS hydraulic model 2025

Figure 8. Inundation Map of the Residential Area in Vina, CA during the February 2025 Event



Source: USACE HEC-RAS hydraulic model 2025

5. Flood Reduction and Recharge Analysis

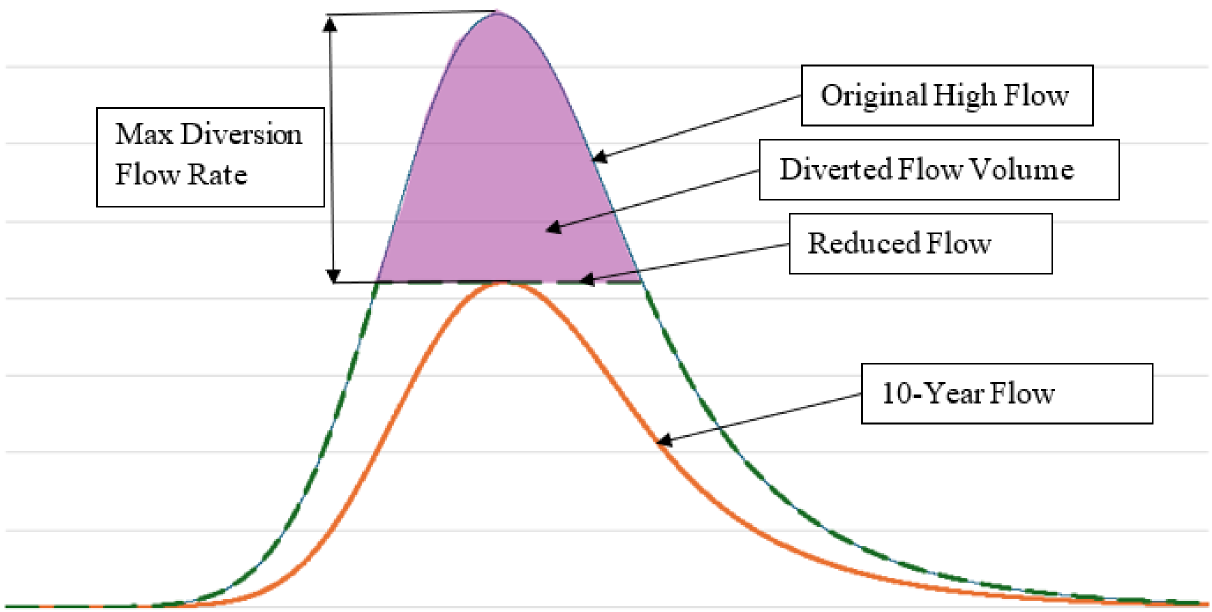
To help reduce floodwater levels within potential inundation areas, diversions under State Water Board Code 1242.1 guidelines (2025) would be allowed when flows exceed the flood diversion threshold. Based on a comparison of historical flooding events and results from the Watershed model discussed in the previous section, flooding is expected to begin when the flow at the CNRFC gage DCVC1 is predicted to exceed approximately 13,574 cfs, corresponding to a 10-year flood event.

Flow scenarios with greater return periods were evaluated to estimate the volume of floodwater that would need to be diverted from the main channel into groundwater recharge basins or designated recharge areas. For evaluation purposes, diversions are proposed at two existing locations: DCID Deer Creek Diversion and Cone Kimball Dam. These two upstream diversions are expected to prevent excess floodwater from overflowing the creek and inundating adjacent agricultural area. No additional diversion is proposed at the existing downstream SVRIC Diversion Dam.

- Figure 9 illustrates the flow hydrograph volume (shown in purple) that would be diverted to reduce the Statistical Design Storm 25-year event peak flow to the 10-year flow threshold. The purple area represents the total estimated volume required to be diverted out of the channel (Table 7).
- As an example, Figure 10 illustrates the peak drainage runoff and peak diversions required to reduce the 25-year event peak flow to the 10-year event peak flow.

- Figure 11 presents the flow-versus-stage rating curve generated from the HEC-RAS hydraulic model for a cross section located at HWY 99 Bridge. The results show the reduction in maximum water depth in the channel – from a 25-year flow without diversion to an equivalent 10-year flow after diversion.
- Table 7 represents the calculated diverted volumes at the proposed diversion points. The table includes a range of return periods (first column) and the corresponding flow of volume reductions (third, fourth, and fifth columns) needed to lower in-channel water depths to the 10-year flood level. The sixth column shows the reduced water depths at the HWY 99 Bridge after diversion, and the final column estimates the area required to recharge the diverted groundwater within 1 day.

Figure 9. Conceptual Flow Reduction Hydrographs



Source: USACE HEC-HMS hydrologic model 2025

Figure 10. Peak Flow Diversions Proposed at two existing diversion dams for Preventing Flooding during the 25-Year Event in the Deer Creek Watershed

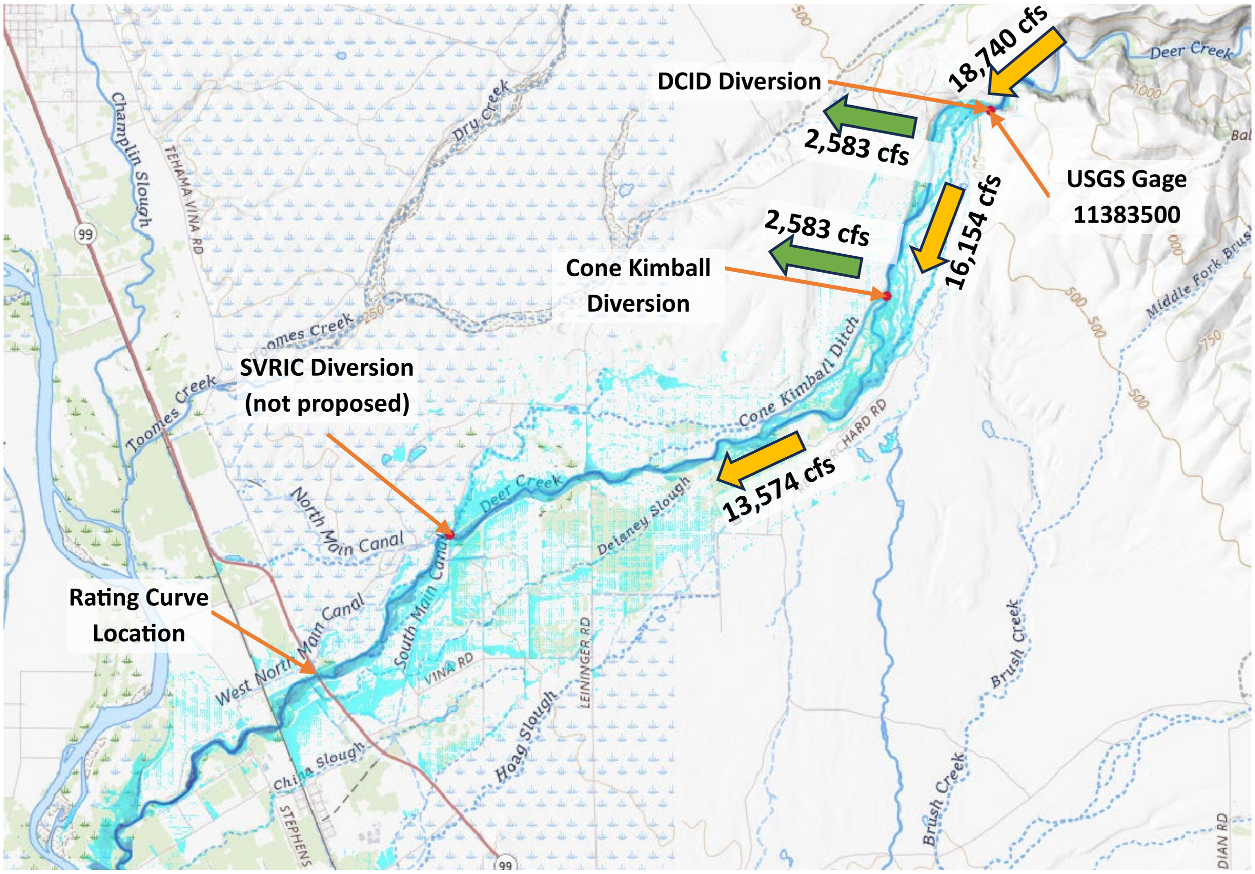
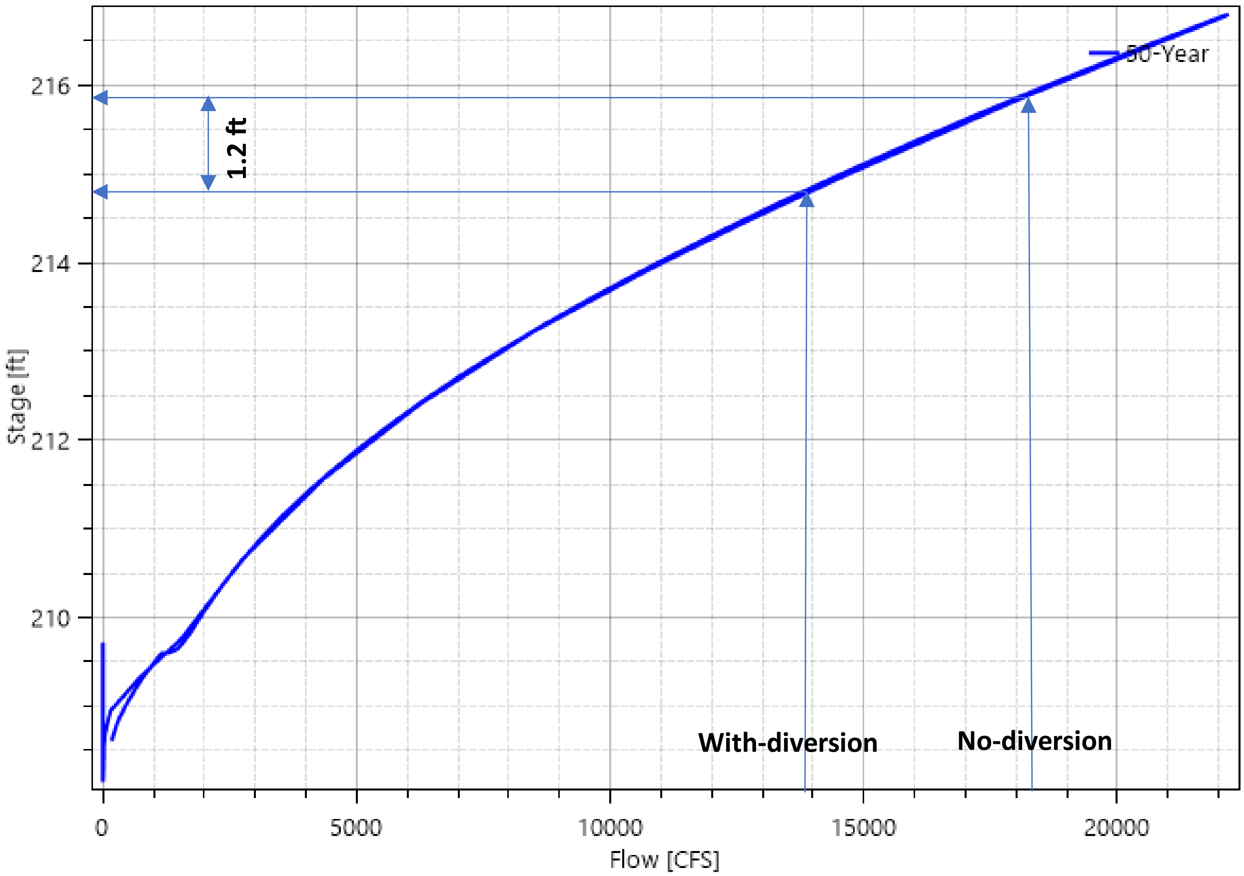


Figure 11. Water Depth Reduction between with- and without-diversion at HWY 99 Bridge on Lower Deer Creek based on a 25-Year Frequency Event



Source: USACE HEC-RAS hydraulic model 2025

Table 7. Estimated Diversions Required to Reduce Flows to Flood Diversion Threshold (10-Year Flood)

Return Period	Peak Flow (cfs) at the USGS gage (from Table 4)	Peak Diversion Flow Required* (cfs) at the two Diversion Dams		Total Diversion Flow Volume (ac-ft) **	Water Depth Reduction at HWY 99 Bridge (ft)	Area Required (acres)
		DCID	Cone Kimball			
10	13,574	0	0	0	0	0
25	18,740	2,583	2,583	3,272	1.2	6,544***
50	22,967	4,697	4,697	7,620	2.0	15,240

Notes: ac-ft = acre-feet / foot; cfs = cubic feet per second

*It is assumed each location will divert 50% of the total required diversion amount.

** This is the total water volume required to be diverted out of the creek at all the diversion points for each event. During a flood event, it is assumed flows in the channel exceeding the peak flow of the 10-year return period threshold would be immediately diverted out of the system into a groundwater recharge basin.

***Assuming a recharge rate of 0.5 foot per day, recharging 3,272 ac-ft of water within 1 day would require 6,544 acres of land. (0.5 foot/day x 6,544 ac =3,272 ac-ft/day)

5.1. Example Diversion to Recharge

Using the hypothetical 50-year event as an example, proposed diversions to groundwater recharge basins and agricultural fields were assumed possible. This event represents an approximate 50-year recurrence interval at the Upper Deer Creek Basin. However, flood frequencies for the Lower Deer Creek Basin remain unknown due to the absence of a local gage with sufficient historical data to calculate reliable statistics.

To reduce the 50-year peak flow (22,967 cfs) to the 10-year peak flow (13,574 cfs), Deer Creek flows were modeled as being diverted into groundwater recharge basins and agricultural fields during the duration of the event. The maximum peak diversion rate in this event required to prevent flooding was approximately 4,697 cfs at each diversion dam.

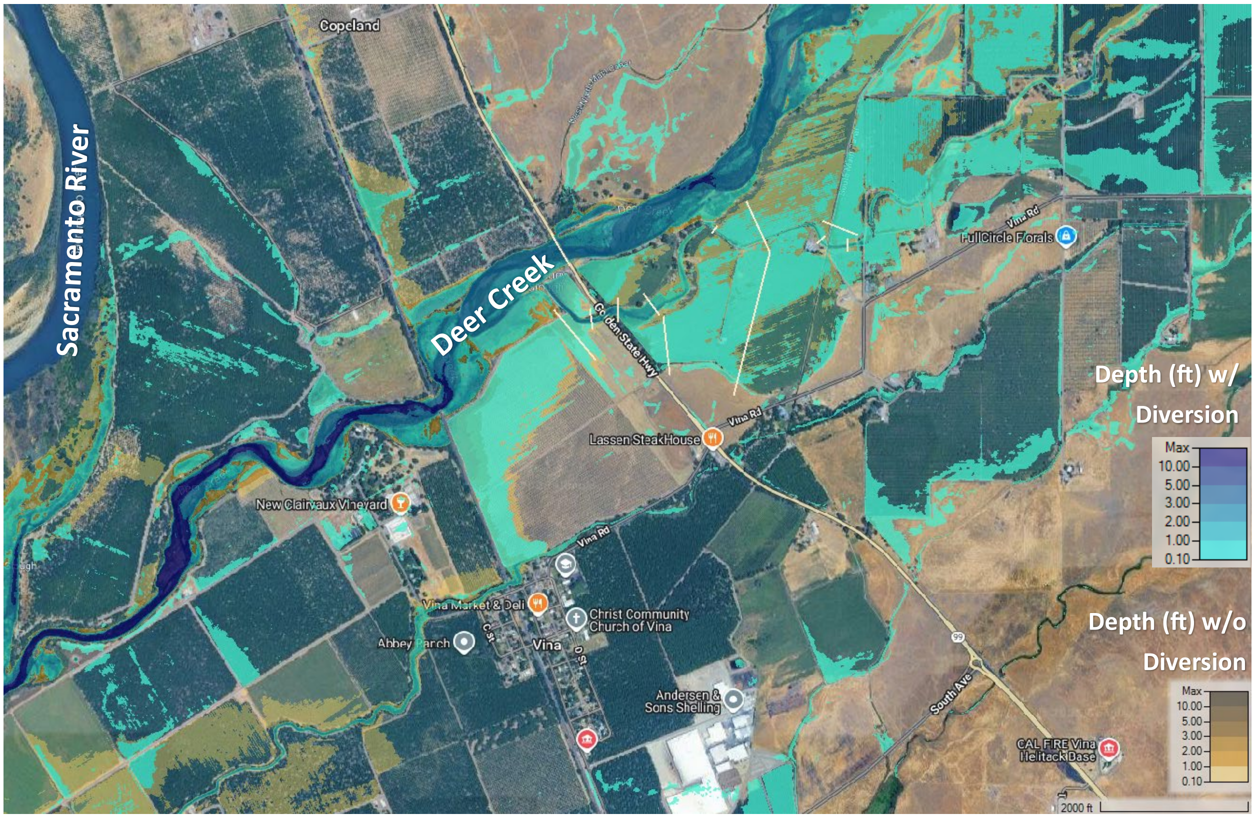
As shown in Figure 12, inundation area in the Lower Deer Creek Watershed, depicted in brown for the pre-diversion scenario, is significantly reduced to the blue area under the diversion scenario. Unlike the original 50-year event, the revised scenario indicates no impact to residential or commercial structures or emergency access roads, demonstrating that the diversion effectively reduces the flood impact to a comparable 10-year event.

It is important to note that the localized runoff from rainfall may spread across agricultural areas and may not be fully diverted from the watershed.

Figure 13 illustrates the flow hydrograph without and with diversion for groundwater recharge during the event, with a total diversion duration of approximately 16 hours.

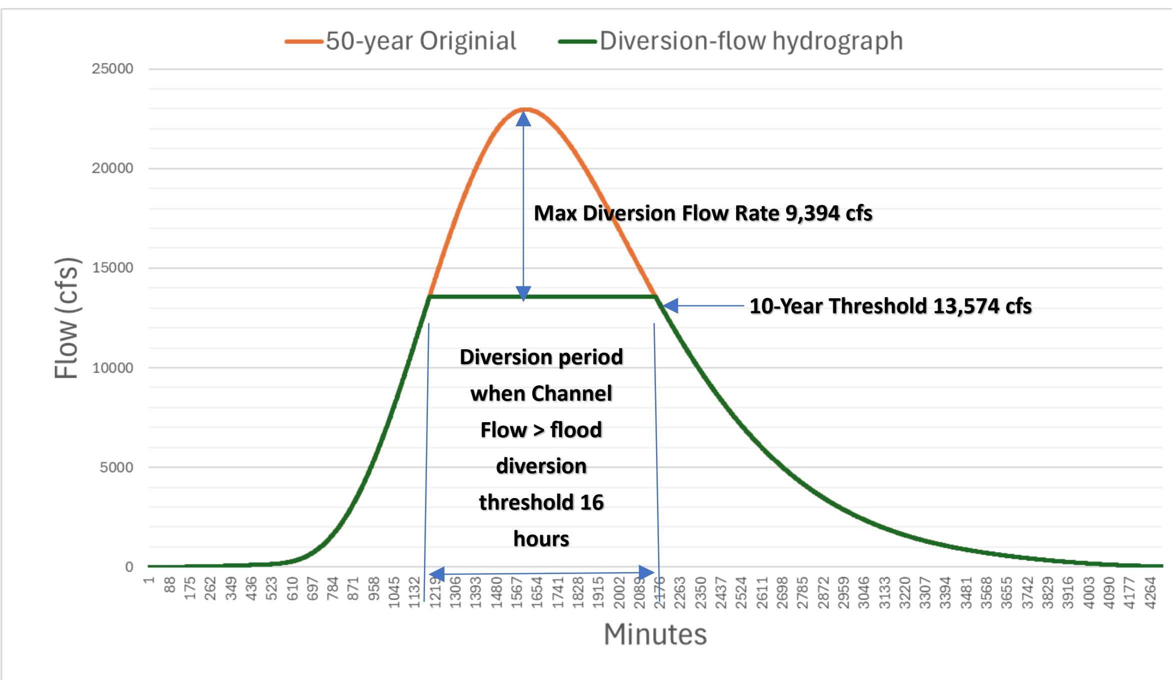
Figure 14 shows that the water levels at HWY 99 Bridge would be reduced by approximately 2 feet with the diversions in place.

Figure 12. Maximum Inundation Area Comparison between Original Scenario (brown) and Diversion Scenario (blue) for the 50-Year Event in the Lower Deer Creek below USGS Gage 11383500



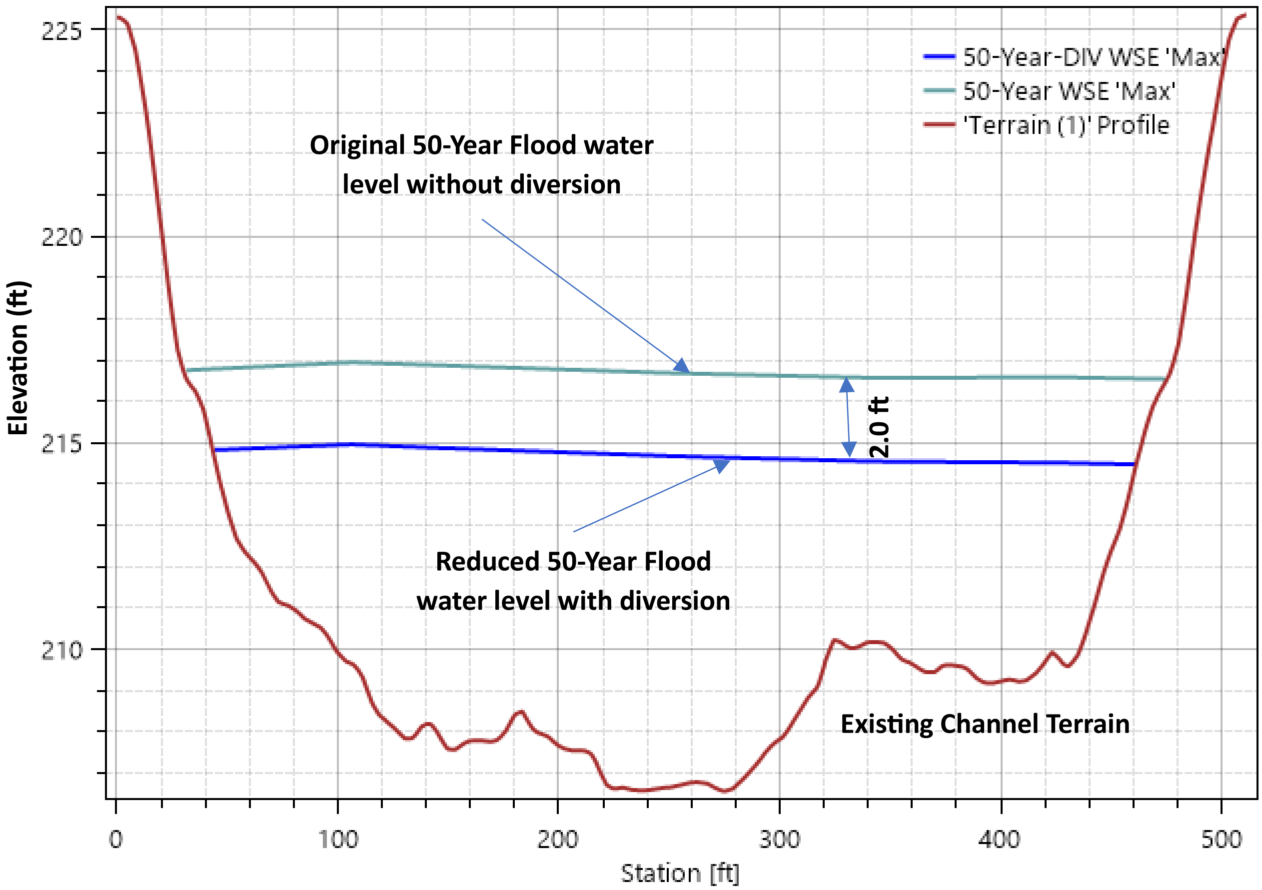
Source: USACE HEC-RAS hydraulic model 2025

Figure 13. Modeled Diverted Flow Hydrograph for Reducing the 50-Year Event Flow to Prevent Flooding.



Source: USACE HEC-RAS hydraulic model 2025

Figure 14. Comparison of Maximum Stage Hydrographs for the 50-Year Event at the HWY 99 Bridge Between the Original Scenario and the Diversion Scenario



5.2. Estimated Water Availability

The State Water Board's *Water Availability Analysis for Streamlined Recharge Permitting Guidelines* (2017) outlines two methods for estimating water availability. This study primarily focuses on Method 2 - the Threat of Flood Conditions, which defines the flood diversion threshold as the 10-year flood event, based on measurements at USGS gage 11383500.

In contrast, Method 1 - The 90th Percentile/ 20 Percent Method allows for diversions of up to 20 percent of daily streamflow when flows exceed the 90th percentile of historical daily flow between December 1 and March 31. This method assumes that sufficient water remains in-stream to satisfy senior water rights.

Table 8 provides estimates of water available for recharge using both methods, based on the historical daily stream USGS gage 11383500 records from Upper Deer Creek Basin. Table 9 lists annual statistics of the number of days that water can be diverted out of the channel for all water year types. The definition of water year types was determined by DWR based on Sacramento Valley (WSIHIST).

Table 8. Statistics of Annual Groundwater Recharge Volumes Availability based on USGS Gage 11383500 records at the Deer Creek.

Method		Min, acre-ft	Max, acre-ft	Average (including years with no volume available), acre-ft	Percent of Years with Volume greater than 0 acre-ft
Method 1	90th Percentile/20 Percent*	30	43,319 (2017)	10,889	89
Method 2	Threat of Flood Conditions**	11	17,155 (1997)	481	11

Notes: ac-ft = acre-feet / foot; cfs = cubic feet per second; No. = number
*The gage daily flow records are from 1911 to 2025. 90th percentile table was obtained from USGS gage website [USGS Surface Water data for USA: USGS Surface-Water Daily Statistics](). Statistics were calculated based on water year (from December to March for each year).

Table 9. Annual Days Diversion Available Based on Method 1 at Deer Creek

Water Year Type	Days Water Available for Diversion			No. Years**
	Min	Max	Average*	
Wet	3	53	22	32
Above Normal	1	33	12	20
Below Normal	1	17	7	16
Dry	1	17	4	17
Critical	1	12	3	24
All Years	1	53	11	109

*For each water year type, the Average values calculations include all years with zero diversion. However, the minimum estimates did not include years with zero diversion.
** The gage data is not complete between 1916 and 1920. So, calculations did not include these five years.

6. Conclusion and Next Steps

This evaluation utilized the latest high-resolution terrain and gage data to demonstrate that higher flow events could result in road closures and property impacts within the Lower Deer Creek Watershed. The analysis showed that diverting flow at the two existing diversion points could effectively reduce water levels across all modeled events.

Flood conditions happen infrequently with an average annual recharge volume of 481 af (assuming all the available volume can be used). The Method 1 90th percentile/20 percent average annual volume and years available are much greater and likely enough larger to make pursuing long term flood diversion water right permits a viable option in addition to diverting as much flow as possible under Method 2.

Recommended Information for Flooding Notices:

- Properties north of 7th Street in Vina, CA may be under imminent threat of flooding when flows exceed the flood diversion threshold of 13,574 cfs (refer to Figure 8).
- Diversions under State Water Board Code 1242.1 guidelines (2025) at any location along the entire geographic extent of Deer Creek would be expected to reduce an imminent flooding threat.

- The flood diversion threshold of 13,574 cfs at USGS gage 11383500 is the key indicator of imminent flood risk.

Proposed Next Steps for Deer Creek:

1. Complete land suitability and availability for recharge analysis for area near the three existing diversion dams and along both sides of the creek between the dams.
2. Assess local runoff which may not be mitigated by the proposed channel diversion measures.
3. Compile real time data upstream of the diversions and capacities within the Deer Creek to further calibrate the system.
4. Quality check and finalize the draft HEC-HMS hydrologic and HEC-RAS hydraulic models.
5. Review and develop alternative strategies to rank potential solutions for flood risk reduction and identify optimal groundwater recharge locations.

7. References

State Water Resources Control Board (State Water Board). 2017. *Water Availability Analysis for Streamlined Recharge Permitting*. Sacramento CA. Accessed October 2025: https://www.waterboards.ca.gov/waterrights/water_issues/programs/applications/groundwater_recharge/docs/streamlined_waa_guidance.pdf

State Water Board. 2025. Updated to Reflect Executive Order N-16-25. January 2025.

ATTACHMENT B

Technical Memorandum

11010 White Rock Road, Suite 200 ● Rancho Cordova, CA 95670 ● 916.631.4500

Via Email: pdhaliwal@lsce.com, wanderson@lsce.com

To: Will Anderson and Pavan Dhaliwal, Luhdorff & Scalmanini Consulting Engineers
Tehama County Flood Control and Water Conservation District, California

From: Bryan Thoreson; Yi Shen (GEI)

cc: Chris Ferrari (GEI)

Date: November 19, 2025

Re: Hydrologic and Hydraulic Model Approach and Evaluation
for the Mill Creek Groundwater Recharge and Flood Reduction Project

Project No. 2403778

1. Introduction and Purpose

Mill Creek originates on the southern slopes of Lassen Peak and generally flows southwest for approximately 60 miles before joining the Sacramento River. Upper Mill Creek is confined within a narrow, steep-sided canyon, while Lower Mill Creek flows for 8 miles through agricultural land before entering the Sacramento River near the northern boundary of Los Molinos, California (Figure 1).

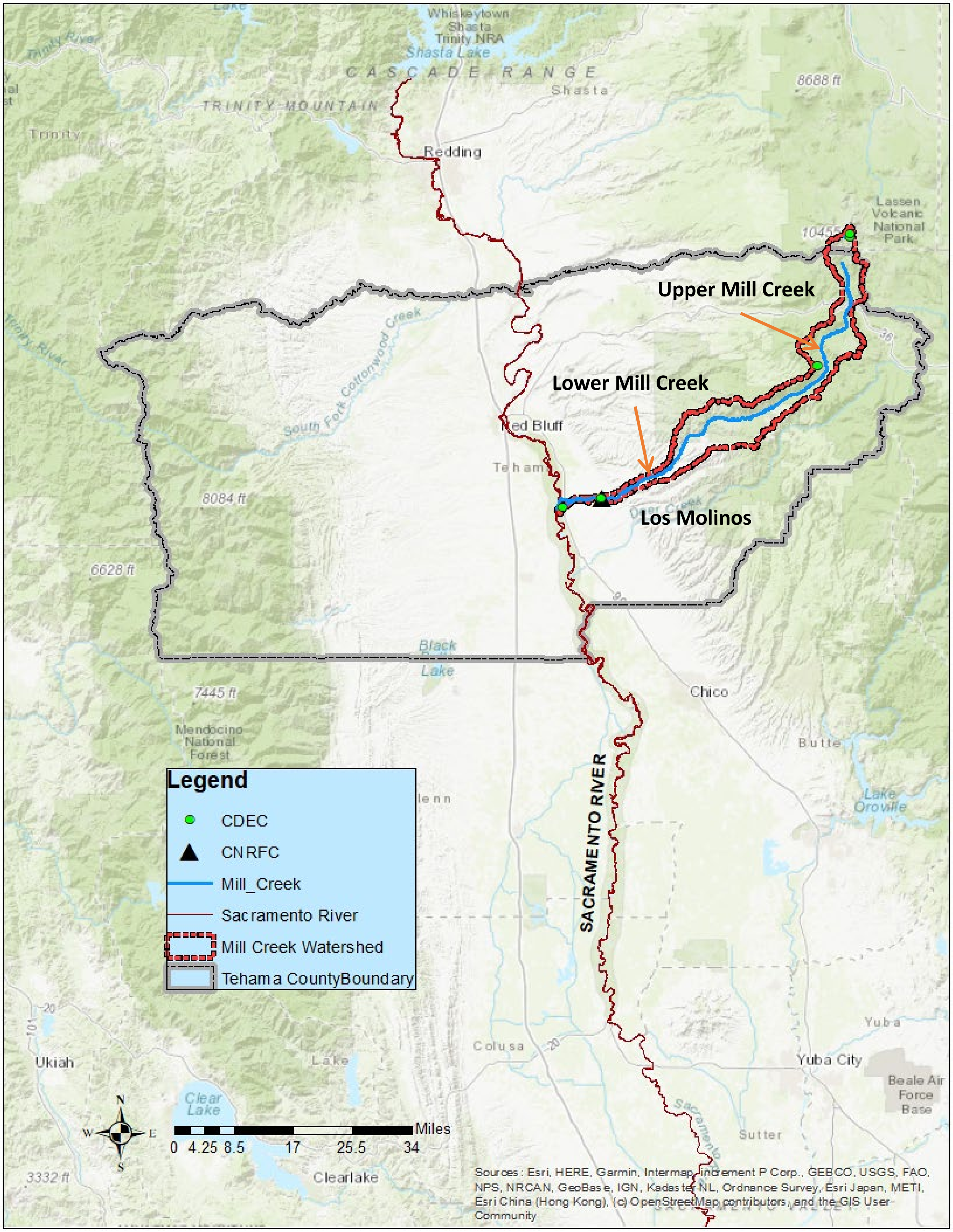
The purpose of this Mill Creek Technical Memorandum (TM) is to support an investigation into the feasibility of diverting surface water from Mill Creek for groundwater recharge, with the goal of alleviating flooding. This TM discusses the following:

- Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) and Hydraulic Engineering Center River Analysis System (HEC-RAS) model development for the Mill Creek watershed
- Utilizing United States Geological (USGS) Survey Terrain
- Validating/calibrating the hydrologic and hydraulic models at USGS gage data
- Hydraulic model floodplain results based on statistical 2- to 100-year storm events.
- Flow and stage forecasting using California Nevada River Forecast Center (CNRFC) tools.
- Developing a Flood Diversion Threshold following California Water Code §1242.1 guidelines
- Estimating potential recharge volume from diversions under California Water Code §1242.1 declared flood emergency.

As flood events become more frequent, preparing for flood diversions and groundwater recharge is crucial for sustainable water management. Under California Water Code §1242.1, allows parties to divert flood flows for groundwater recharge without a water right if in compliance with certain requirements.

The goal of this TM is to outline the methodology and results developed to inform and forecast flood flows and inundation extents, supporting local emergency management efforts. In the final section of this TM, the existing conditions model will be used to evaluate various project alternatives for diverting flow to recharge groundwater, along with the estimated flood reduction results.

Figure 1. Project Location



Source: Miil Creek Watershed

2. Background

The Mill Creek Watershed, located in Tehama County, east of the Sacramento River, spans approximately 133 square miles (Figure 2). Elevations within the watershed range from 8,000 feet in Lassen National Park to 200 feet near the Sacramento River. Tributaries, Boat Gunwale Creek and Little Mill Creek, merge with the main channel of Mill Creek before it enters the Sacramento Valley.

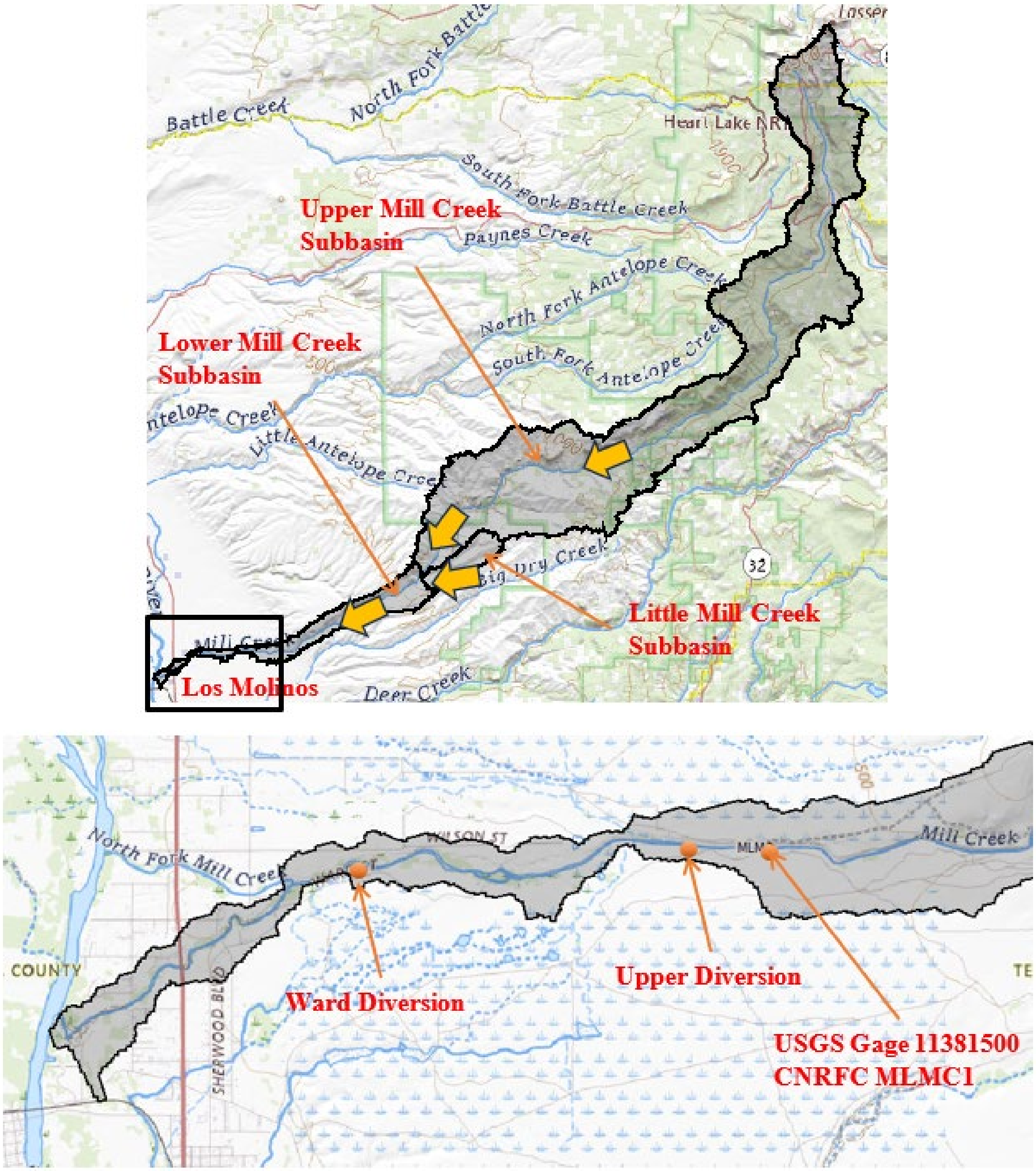
Mill Creek has a natural hydrologic pattern characterized by high flows and peak runoff events during winter, and low flows in summer and fall. The average annual daily flow is approximately 400 cubic feet per second (cfs). The highest recorded peak runoff event was 36,000 cfs in December 1937.

Major irrigation diversions from Mill Creek include the Upper Diversion (River Mile [RM] 5.4) and Ward Diversion (RM 2.8), both operated by the Los Molinos Mutual Water Company. According to the U.S. Fish and Wildlife Service (2000), the authorized maximum diversion rates are 123 cfs for the Upper Diversion and 60 cfs for the Ward Diversion.

There is no storage dam or reservoir within the watershed. However, significant groundwater resources exist in the lower elevation areas near Los Molinos, supporting both domestic and agricultural water needs.

The only stream gage that monitors high flows in Mill Creek is USGS 11381500 (also known as CNRFC MILL CREEK - LOS MOLINOS [MLMC1]), located at RM 5.8 near Los Molinos, California, above both major diversions (Figure 2). Figure 2 shows the three subbasins of the Mill Creek Watershed, the existing diversion locations and the USGS flow gage.

Figure 2. Subbasins, Diversions, and Stream Gage in the Mill Creek Watershed



Source: Mill Creek Watershed

3. Hydrologic and Hydraulic Models Development

This section discusses the approaches for setting up hydrologic and hydraulic models. These models will then be utilized to evaluate existing conditions and project alternatives and how to use the CNRFC information to forecast flood flows for Mill Creek.

3.1. HEC-HMS Model Development

The hydrologic model was developed using HEC-HMS 4.12 software. For the Mill Creek study, a 1-meter resolution digital elevation model (DEM) was downloaded from the USGS's website and applied to the hydrologic model domain. Rainfall information required by the hydrologic model was obtained from the National Oceanic and Atmospheric Administration's (NOAA) website. Specifically, NOAA Atlas 14 hypothetical design storms for return periods of 2-, 5, 10-, 25-, 50-, 100-, 200-, and 500-year were downloaded in a gridded format and used as the total rainfall amount for a 24-hour period. The designed 24-hour temporal distribution of the rainfall was also downloaded from NOAA's website.

The Mill Creek Watershed is divided into three subbasins, as shown in Figure 2, which illustrates both watershed boundaries and terrain. The Upper Mill Creek Subbasin (117 square miles) encompasses flows originating from the glaciated slopes of Mount Lassen down to the confluence with Little Mill Creek. The Little Mill Creek Subbasin (5 square miles) represents the flow contribution from Little Mill Creek itself. The Lower Mill Creek Subbasin (11 square miles) extends from the confluence of Upper Mill Creek and Little Mill Creek downstream to the Sacramento River.

Precipitation loss due to infiltration was calculated using the Soil Conservation Service (SCS) Curve Number loss method, which accounts for soil drainage characteristics and land use within each subbasin. Runoff modeling in this study was performed using the SCS unit hydrograph approach, which requires specification of each subbasin's lag time (L) and Peak Rate Factor.

The lag time (L) parameter for each subbasin was estimated based on flow path length, maximum potential retention, and average watershed land slope. Table 1 summarizes the model parameters used to simulate statistical storm events and those applied during calibration for historical storm events.

Table 1. HEC-HMS Model Parameters

Subbasin	Area (mi^2)	T_{lag} (min)	Peak Rate Factor	Curve Number
Upper Mill Creek	117.3	150	500	72
Little Mill Creek	4.67	91	484	87
Lower Mill Creek	11.3	211	484	85
Total	**133.3**	**N/A**		

Notes: mi^2 = (square miles); N/A = not applicable; Tlag (minutes)

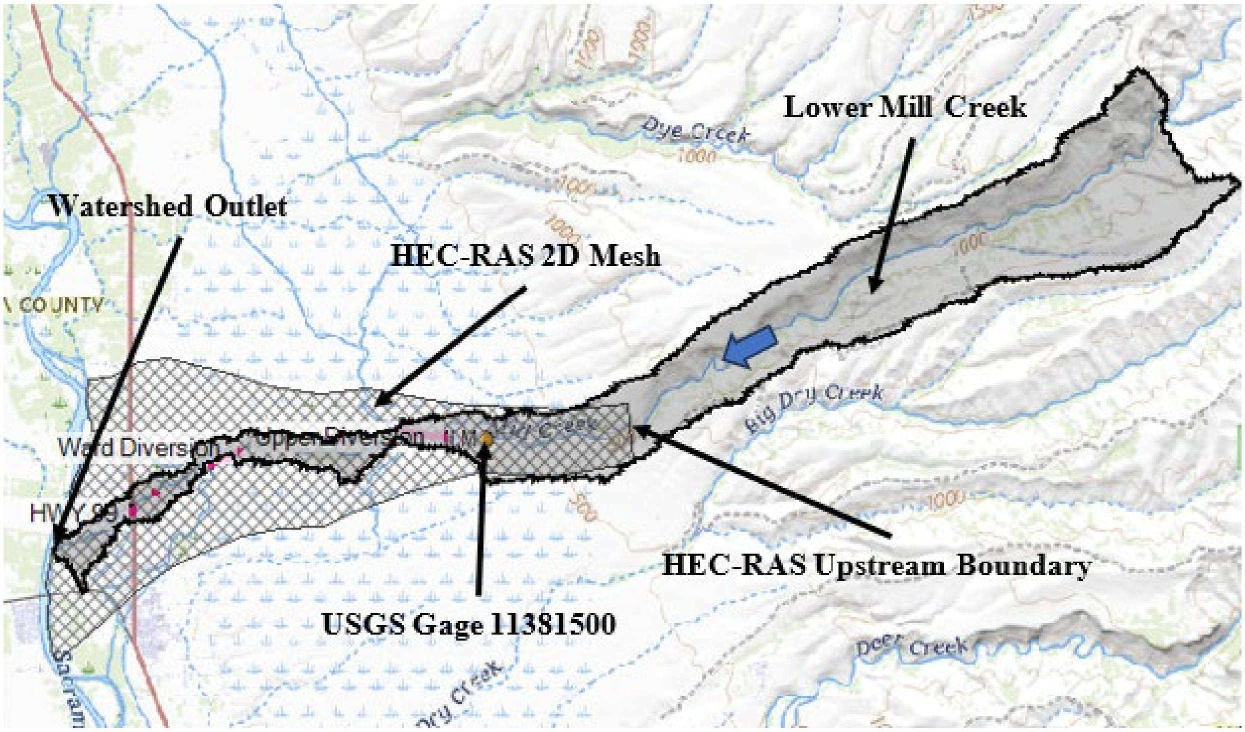
3.2. HEC-RAS Model Development

The hydraulic model for this study was developed using HEC-RAS 6.6 software. A single two-dimensional (2D) mesh was created within HEC-RAS, covering the Lower Mill Creek Watershed (Figure 3). A conservative modeling approach was applied by assigning flow hydrographs at the upstream boundary of the HEC-RAS model, corresponding to the outlet of the entire Mill Creek Watershed.

Peak flows in the hydrographs were calculated from either gage records or from the Hydrologic Engineering Center Statistical System Program (HEC-SSP). These values were then adjusted using the drainage-area ratio method to account for the differences between the gage location and the watershed outlet. The drainage area ratio method is a way to estimate streamflow at the watershed outlet by multiplying the streamflow at the gage by the ratio of the drainage areas of the outlet and gage sites.

The temporal distribution of the flow hydrographs followed the results generated by HEC-HMS at the watershed outlet.

Figure 3. HEC-RAS Model 2D Mesh and Upstream Boundary Inflows



Source: HEC-RAS

4. Model Results and Findings:

This section discusses the hydrologic and hydraulic model evaluation:

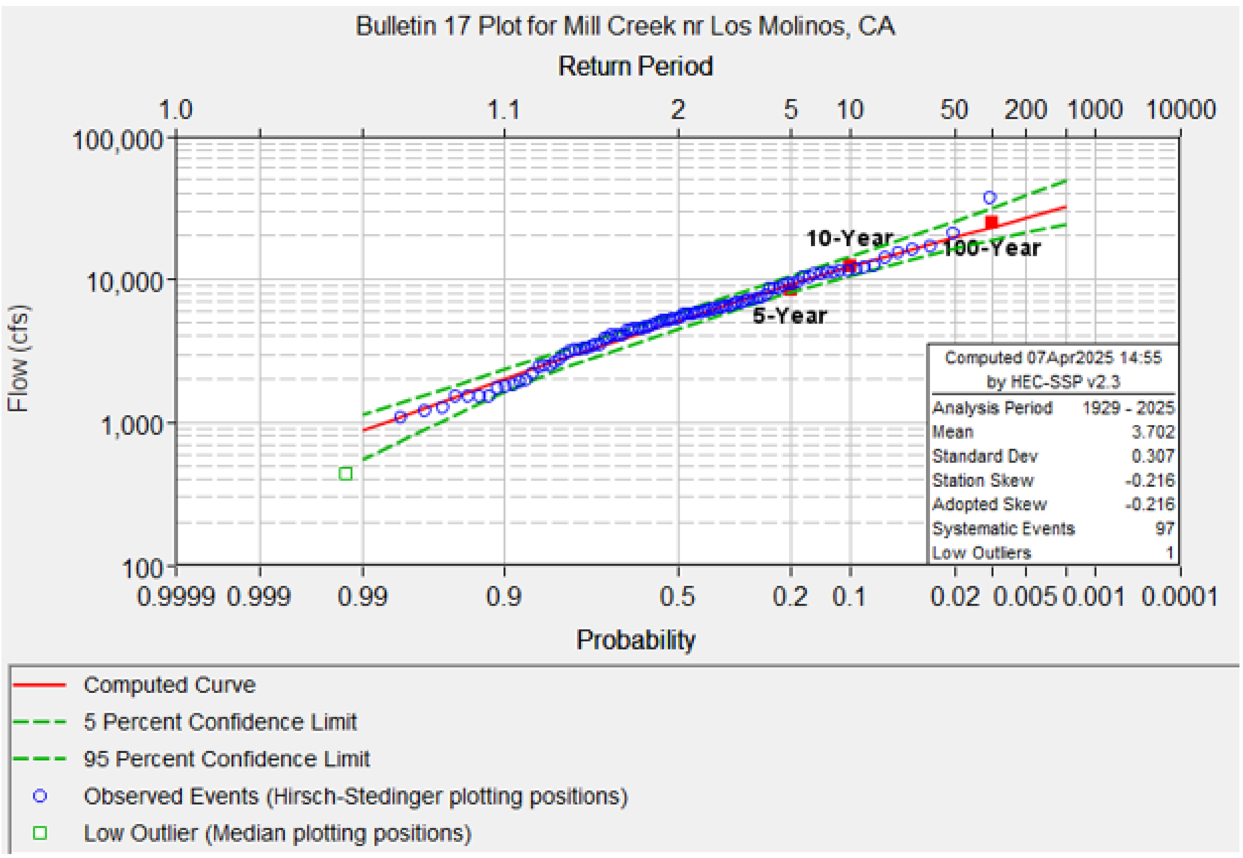
- The hydrologic model flow hydrograph was calibrated to the USGS gage 11381500 – Mill Creek near Los Molinos, CA for the 1997 event. The model was then validated at the same gage location for the 2024 event. The USGS gage is the same gage as the CNRFC gage MILL CREEK - LOS MOLINOS (MLMC1). Unlike the USGS gage, the CNRFC gage does not keep long-term historical records in its database but does provide a 5-day forecast. The calibrated HEC-HMS model was also used to verify the peak flows at the MLMC1 USGS gage for the three statistical storms (5-, 10-, and 100-year).
- HEC-HMS was used for five statistical design storms for the 5-, 10-, 50-, 100-, and 500-year storm frequency. All the model parameters were calculated using formulas recommended by the HEC-HMS user manual.
- The hydraulic model using HEC-RAS was used to calculate stage hydrographs and flood inundation maps for all the events.

4.1. Gage Frequency Analysis

Figure 4 illustrates the flood frequency analysis conducted using the HEC-SSP at USGS gage 11381500. The computed curve was developed based on 96 years of annual peak flow data, spanning from 1929 to 2024.

Red dots on the figure represent peak flows for the 5-, 10-, and 100-year return periods, as computed by the HEC-HMS hydrologic model, allowing for direct comparison. The percent error between the gage-derived frequency results and the hydrologic model estimates for these three peak flow events was less than 10 percent, indicating strong agreement. Table 2 provides the peak flow estimates at the gage from the HEC-SSP analysis.

Figure 4. Flood Frequency Results at USGS Gage 11381500



Source: HEC-SSP

Table 2. HEC-SSP Statistical Peak Flow Estimates on Mill Creek at USGS Gage 11381500

90% Exceedance	2-Year (cfs)	5-Year (cfs)	10-Year (cfs)	50-Year (cfs)	100-Year (cfs)	500-Year (cfs)
2,003	5,159	9,175	12,222	19,764	23,258	31,989

Notes: % = percent; cfs = cubic feet per second

Table 3 presents peak flow events recorded at USGS gage 11381500 for the past 10 years, beginning with Water Year 2015. Each event is accompanied by its estimated storm frequency, based on the statistical analysis using the HEC-SSP. All events, except for 2024, were classified as less than a 10-year event (i.e., a storm with a 10% annual chance of occurrence). The 2024 event, however, was estimated to correspond to an approximate 19-year frequency event, or 5 percent annual chance storm.

Table 3. Historical Events for the past 10 water years* (2015-2025)

Water Year*	2025	2024	2023	2022	2021	2020	2019	2018	2017	2016	2015
Calendar Year	Nov 2024	Feb 2024	Jan 2023	Oct 2021	Jan 2021	Jan 2020	Feb 2019	Apr 2018	Dec 2016	Mar 2016	Dec 2014
Gage Peak Flow (cfs)	15,400	5,740	7,370	5,070	1,180	1,740	8,500	5,670	10,800	8,670	9,520
Return Period	19	2	3	<2	<2	<2	4	2	7	4	6

Note: cfs = cubic feet per second

4.2. HEC-HMS Model Verification for the Design Storms

The hydrologic HEC-HMS model verification and calibration results are summarized in Table 4. The flow hydrograph comparisons between the hydrologic model and the gage records for the two historical events are shown in Figures 5 and 6. Overall, the hydrologic and hydraulic models performed well to forecast peak flows and water surface elevation stages, with all modeled errors remaining below 10 percent when compared to observed data from the gage records.

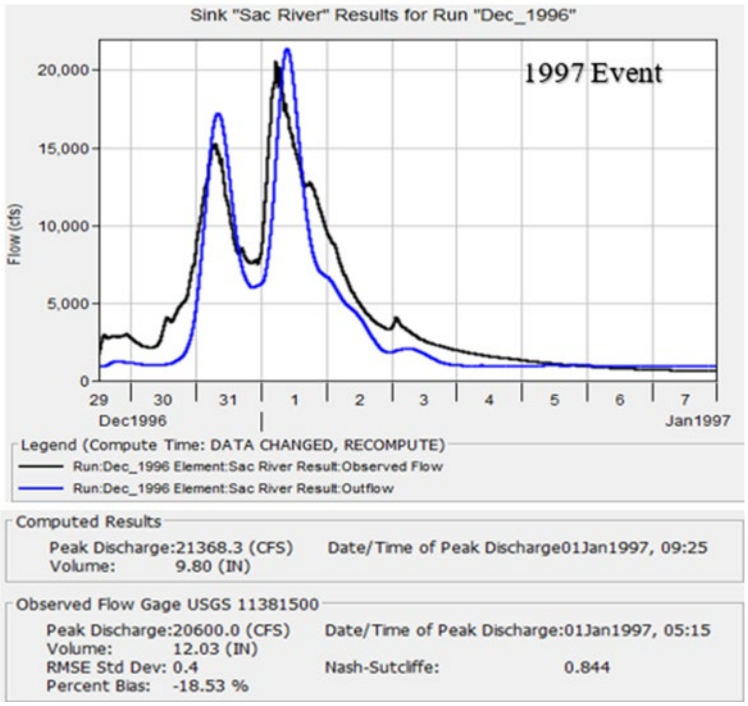
Comparisons of the flow hydrographs at the watershed outlet are shown in Figure 7.

Table 4. HEC-HMS Model Calibration Summary

	Statistical Design Storm			Historical Storm	
	100-Year	10-Year	5-Year	1997	2024
Statistical/Observed Peak Flow (cfs)	23,258	12,222	9,175	20,600	15,400
Modeled Peak Flow (cfs)	25,159	12,375	8,428	21,368	14,892
Percent Difference (%)	6.3	-0.5	-9.7	3.7	-3.3

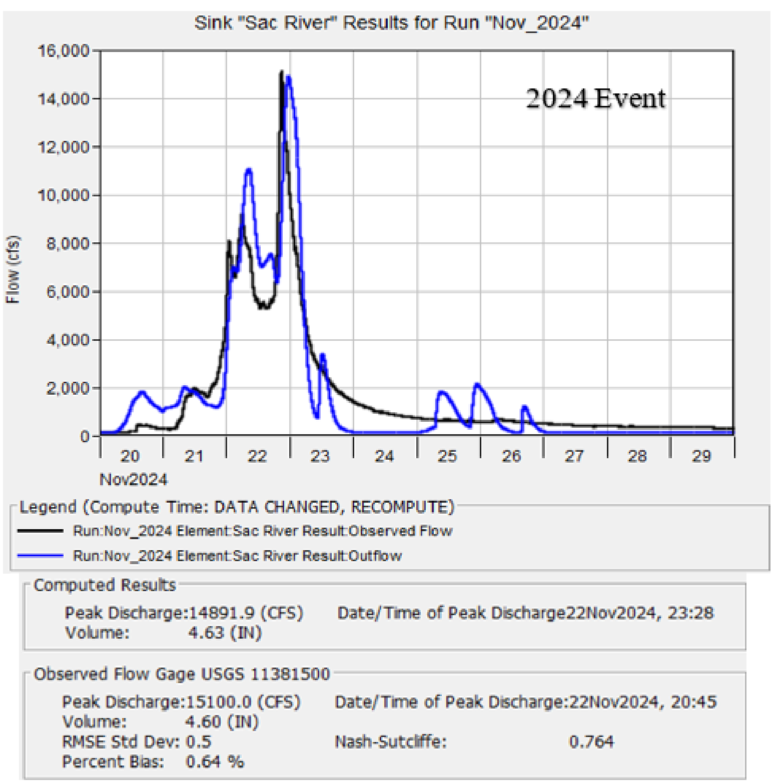
Note: cfs = cubic feet per second

Figure 5. Flow Hydrograph Comparison for the January 1997 Event



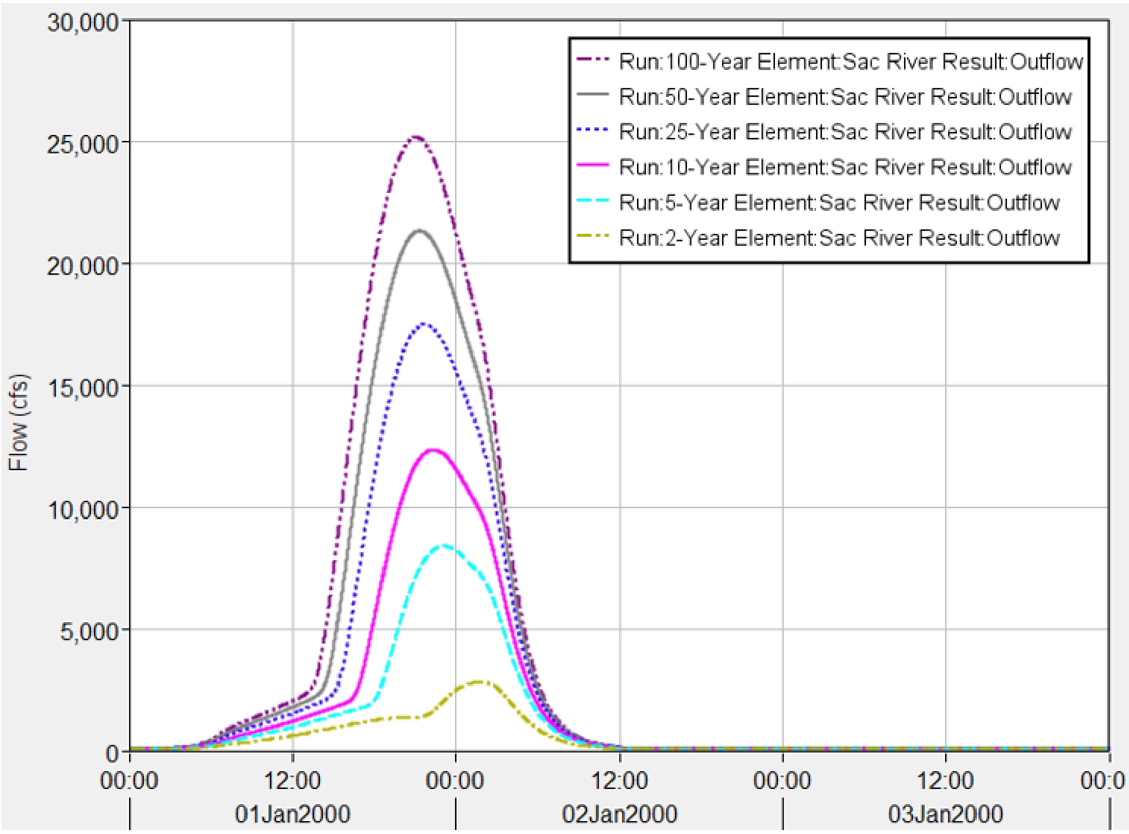
Source: HEC-HMS

Figure 6. Flow Hydrograph Comparison for the November 2024 Event



Source: HEC-HMS

Figure 7. Mill Creek Flow Hydrographs at the Watershed Outlet



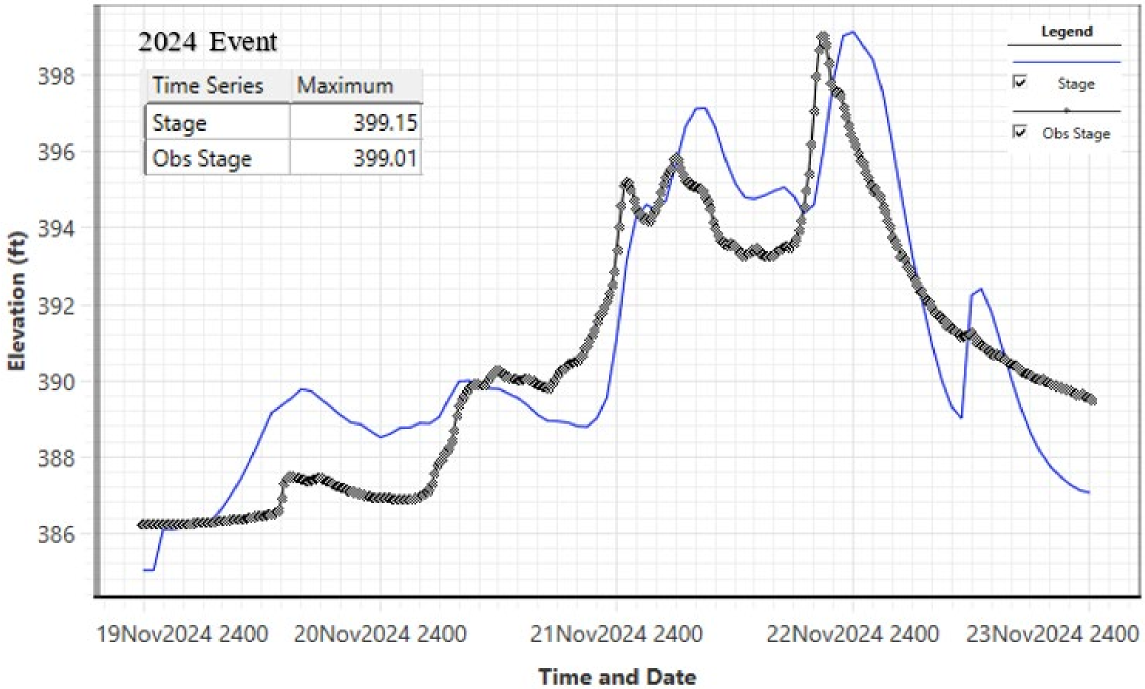
Source: HEC-HMS

4.3. Hydraulic Model Verification Based on 2024 Historical Storm

The 2D hydraulic models utilized flow hydrographs from the hydrologic models. The hydraulic model routes the flow hydrographs to provide water depth, flood extents, and velocities for the different return periods. Maximum depth inundation maps and stage hydrographs for the different storm events were based on the USGS gage 11381500.

Comparisons of the stage hydrographs at the gage location are presented for the 2024 event (Figure 8). The 1997 event was not included because there is no stage gage record for that period. The stage hydrograph shape of the model generally follows the observed characteristics. The maximum stage difference between the model and the gage records is less than 10%. Since the stage comparison appears to be reasonable, no additional calibration was performed for the hydraulic model.

Figure 8. Stage Comparison for February 2024 Event



Source: HEC-HMS

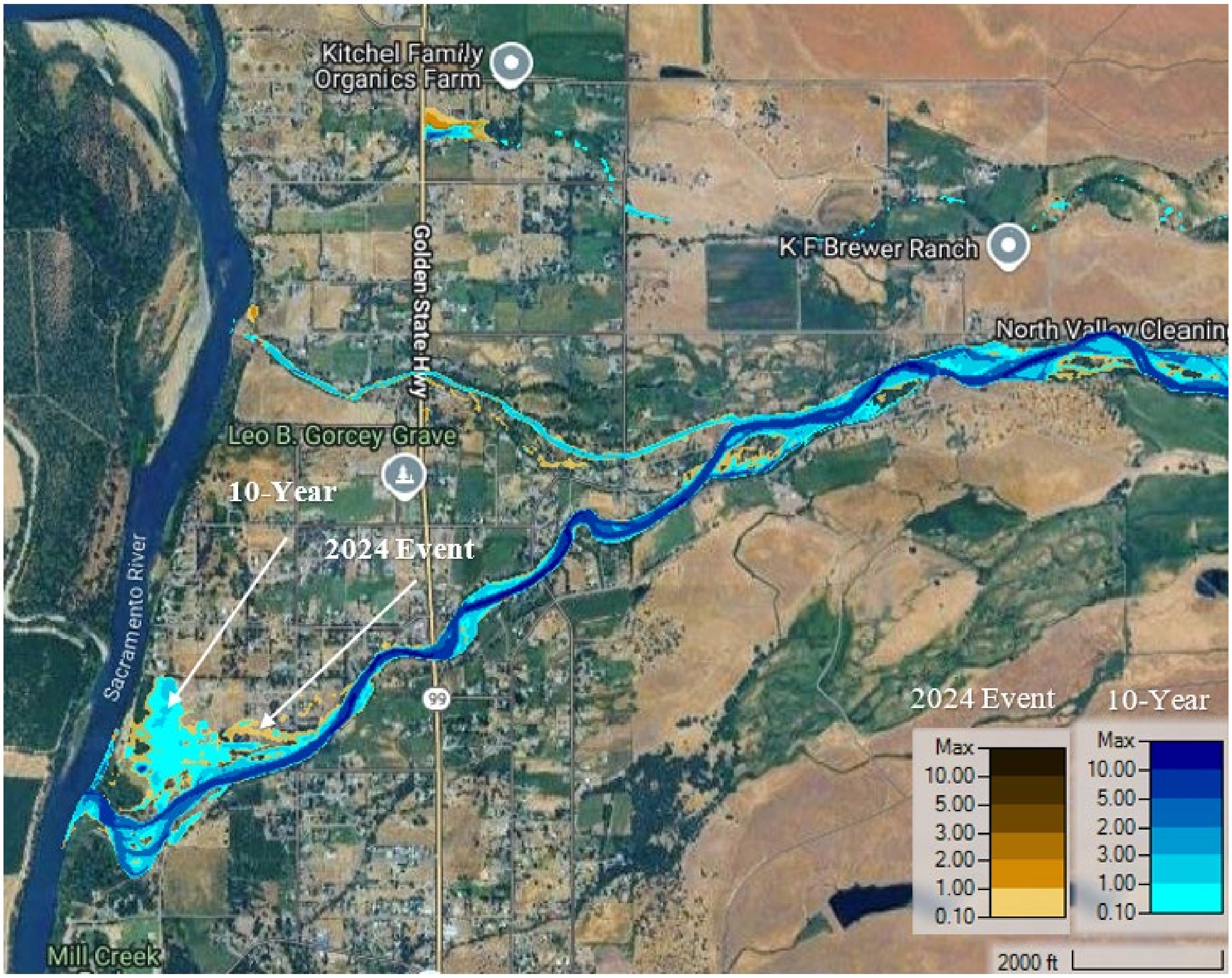
4.4. 2024 Flood Event Impact in Los Molinos

The model results predicted the 2024 flood event may cause road closures, property damage, and restricted access on the north bank of Mill Creek downstream of Highway 99 (HWY 99).

In order to find a flood threshold below which no potential flood impact to properties would occur, a comparison between the 10-year event and the 2024 historical storm (19-year event) was made in Figure 9. Compared to the 10-year event, the flood inundation area in the 2024 event was slightly expanded on the north bank of Mill Creek.

A detailed examination of the modeled 2024 event revealed that several local properties along Ellis Street could be impacted by a water depth up to 0.5 foot (Figure 10). Possible road impacts include Ellis Street and Hollis Street.

Figure 9. Max Depth Inundation comparison between 10-year (blue) and 2024 Event (brown)



Source: HEC-RAS

Figure 10. Inundation Map on the north bank of Mill Creek below HWY 99 at the 2024 Event

Source: Google

Comparison of the inundation maps (Figures 9-10) indicate flooding on the Lower Mill Creek Watershed happened during the 2024 event (19-year), but not the 10-year event. It can be speculated that any flow exceeding the threshold 12,222 cfs would overtop the banks and be an imminent flooding threat to local residential structures (Figure 10). Therefore, the flood diversion threshold was set at a peak flow of 12,222 cfs, corresponding to the 10-year event at the USGS gage 11381500.

5. Flood Reduction and Recharge Analysis

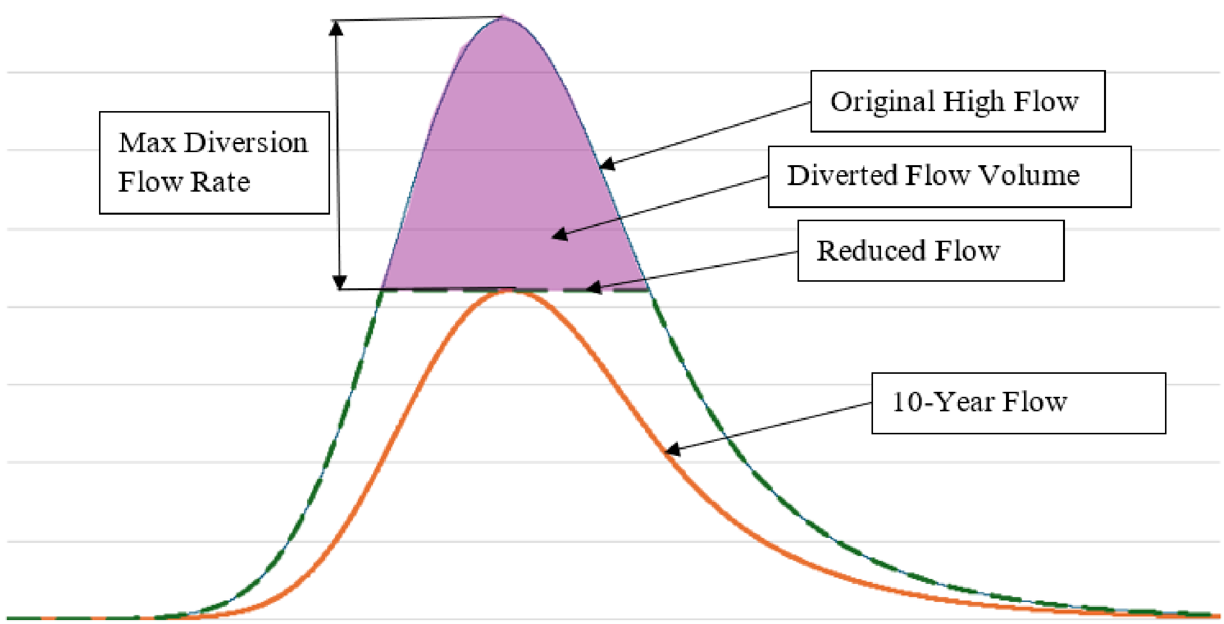
To reduce water levels within the potential flood inundation area, diversions under California Water Code §1242.1 would be allowed when the flood diversion threshold flow is exceeded. Based on the comparison of historical flooding to the watershed model results in the previous section, flooding would begin if the flow forecasted at the CNRFC gage MLMC1 exceeds approximately 12,222 cfs (the 10-year event).

In this study, higher flows with higher return periods were evaluated to estimate the floodwater volume that would need to be diverted from the main channel into a groundwater recharge basin or on to allowable recharge areas. For evaluation purposes, the diversions would be made at the two existing water diversion locations at the Upper Diversion and the Ward Diversion. It was assumed that each diversion point would divert 50 percent of the total required diversion volume during a flood event.

- Figure 11 shows the flow hydrograph volume (purple color) that would be removed by reducing the peak flow to the 10-year flow threshold. The purple area represents the total estimated volume required to be diverted out of the channel as presented in Table 5. Each diversion point would divert half of the volume. The maximum diversion rate at each diversion point would also be half of the Maximum Diversion Flow Rate shown in the figure.
- Figure 12 illustrates the diversion flows required at the two diversions along the Mill Creek to reduce the 19-year flow (Qpeak 15,400 cfs) in 2024 to a 10-year flow (Qpeak 12,222 cfs).
- Figure 13 presents the flow versus stage rating curve from the HEC-RAS model for a cross section located at HYW 99 Bridge. The result shows the maximum water depth reduction in the channel from a 19-year flow to a 10-year flow.

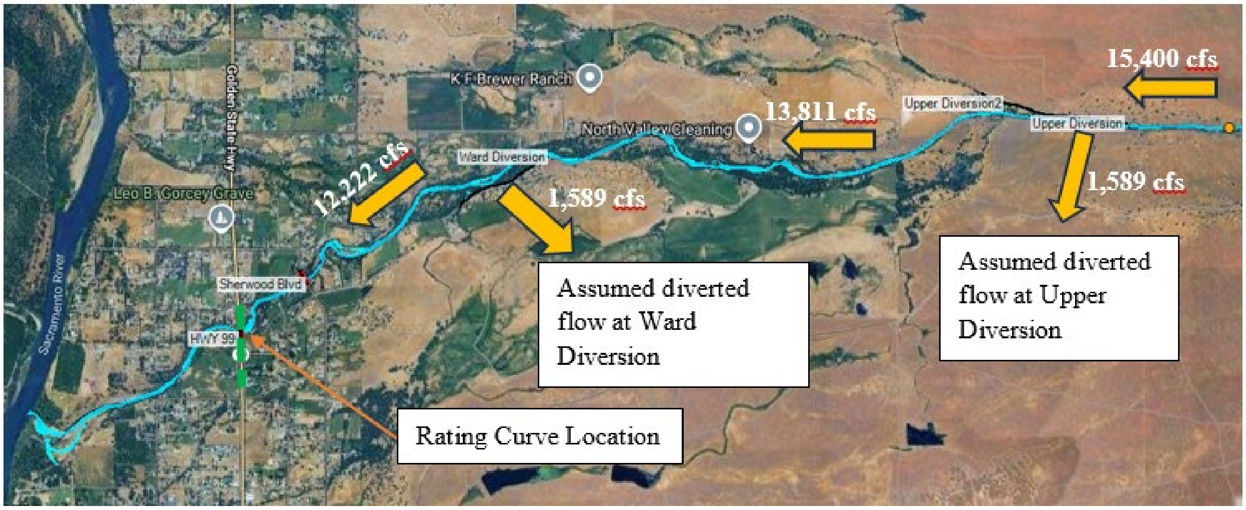
Table 5 represents the proposed diverted volume at both the Upper Diversion and the Ward Diversion. The results show a range of return periods (1st column of Table 5) to reduce the in-channel water depths to a 10-year water level by diverting flows and volumes (2nd and 4th columns) to the two diversion points. The reduced water depths (5th column) after diversion (to a 10-year water level) are represented by the green dashed line in Figure 11. The last column of the table is the area required to recharge the groundwater within 1 day assuming the recharge rate is 0.5 foot per day. A smaller area could be used if berms or levees were available to hold water on the smaller area.

Figure 11. Conceptual Flow Reduction Hydrographs



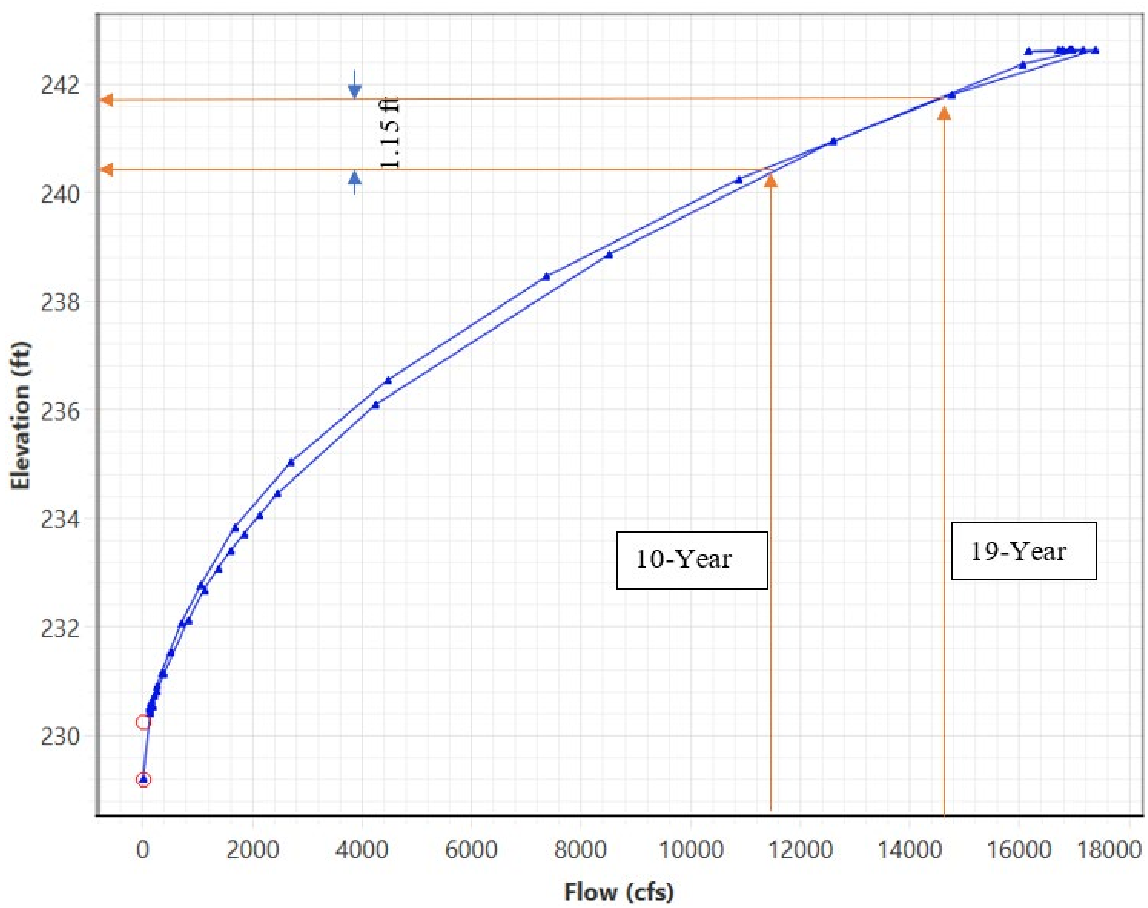
Source: HEC-HMS

Figure 12. Peak Flow Diversions Required for Preventing Flooding from the 2024 Event in Lower Mill Creek



Source: HEC-RAS

Figure 13. Water Depth Reduction between 10- and 19-Year Frequency (2024 Event) on the Downstream Side of the HWY 99 Bridge on Mill Creek



Source: HEC-RAS

Table 5. Estimated Diversions Required to Reduce Flows to Flood Diversion Threshold (12,222 cfs)

Return Period	Peak Flow (cfs) at the Gage (from Table 1)	Peak Diversion Flow (cfs) **	Total Diversion Flow Volume (ac-ft) ***	Water Depth Reduction at HWY 99 Bridge (ft)	Area Required (acres)
10	12,222	0	0	0	0
19*	15,400	3,178	330	1.1	660****
50	19,764	8,759	4,717	7.8	9,434

Notes: ac-ft =acre-feet; cfs = cubic feet per second; ft = foot/feet

*The 2024 event was a 19-year event used as an example to calculate diversions in the table.

**This is the proposed peak diversion flow rate required at both diversion points.

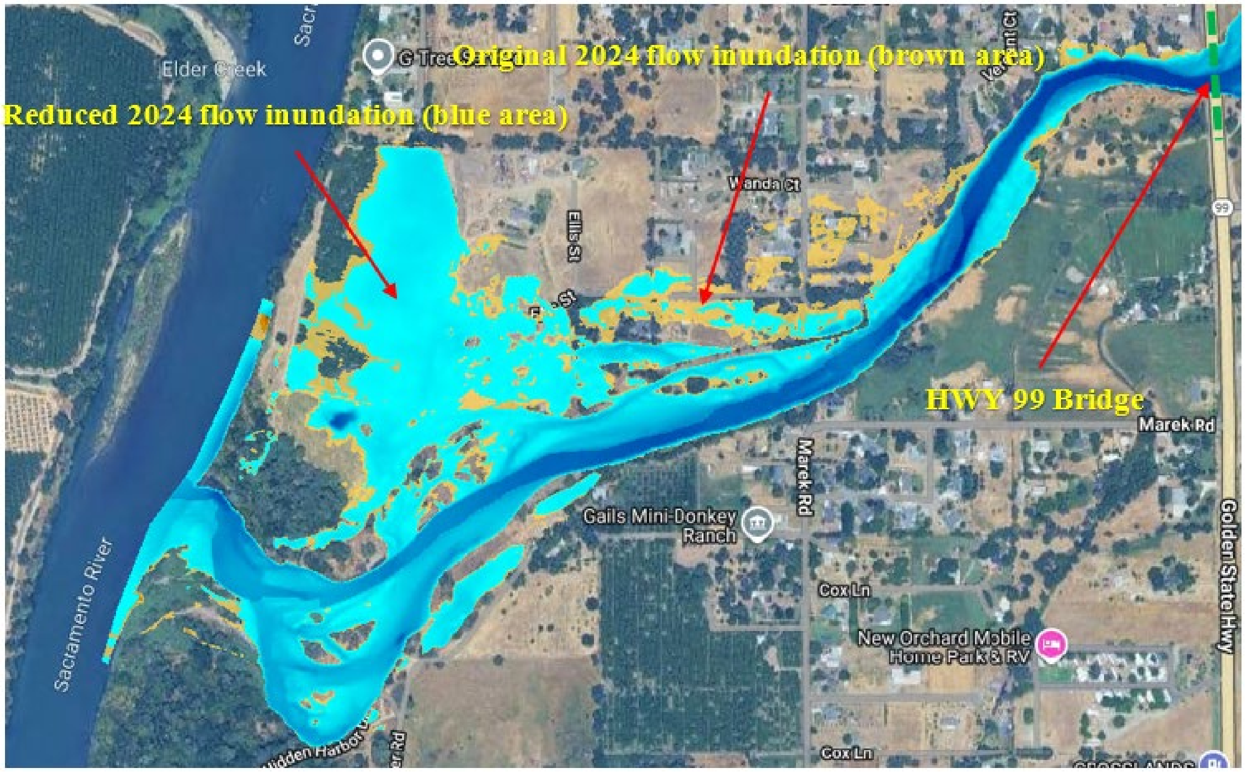
***This is the model calculated total water volume required to be diverted out of the river at both diversion points and additional proposed diversions for each event. During a flood event, it is assumed any portion of flow in Mill Creek exceeding the peak flow of the 10-year return period threshold (12,222 cfs) would be immediately diverted out of the system into a groundwater recharge basin or onto allowable recharge areas.

****If we assume the recharge rate is 0.5 foot per day. To completely recharge 330 ac-ft water into the groundwater basin within 1 day, we need 660 acres of land. (0.5 foot per day * 660 ac = 330 ac-ft/day)

Example Diversion to Recharge

Using the 2024 flow event as an example, in Figure 14, proposed diversions to recharge basins and agricultural fields for groundwater recharge were assumed possible, at both Upper Diversion Dam and Ward Diversion Dam. The 2024 event was simulated and all flows above the 10-year peak flow (12,222 cfs) were modeled as being diverted from the channel into the groundwater recharge basins and agricultural fields during the event period. As indicated in Table 5, the peak diversion rate in this event required to prevent flooding was 3,187 cfs. Assuming, due to capacity restrictions in the downstream canals, each existing diversion dam would be capped at maximum existing authorized diversion of 123 cfs and 60 cfs by USFWS (2000), an additional 2,995 cfs would need to be diverted.

Figure 14. Maximum Inundation Area Comparison between Original and Reduced 2024 Event in Lower Mill Creek below HWY 99 Bridge

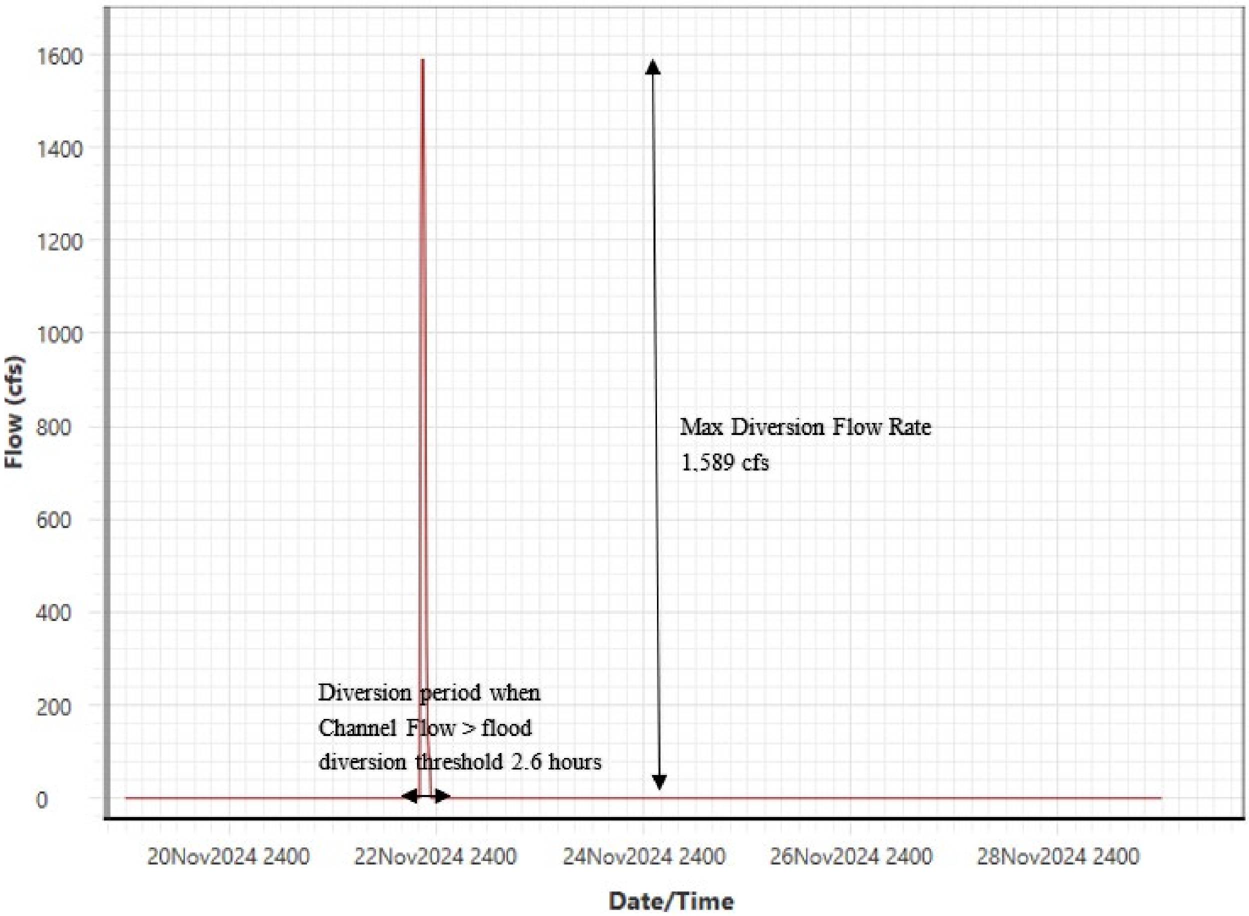


Source: Google Earth™

In the Lower Mill Creek below HWY 99 Bridge, the brown inundation area, representing conditions without diversions, will be reduced to the blue inundation area when the diversion is implemented. This change indicates that the diversion would reduce the 19-year flood event down to a 10-year event.

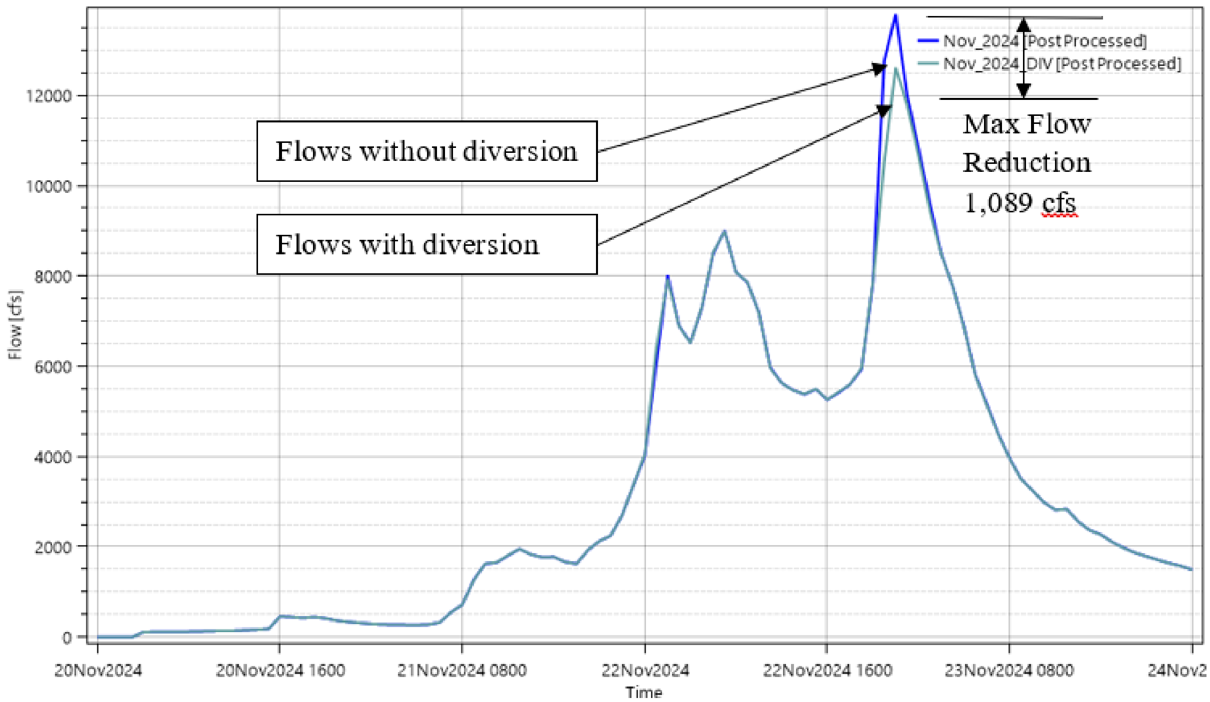
Figure 15 illustrates the proposed flow hydrograph diverted into the groundwater recharge basin during this event for approximately 2.6 hours. Figure 16 shows the reduction in flow hydrograph at the cross section with and without the diversion. Figure 17 demonstrates that the water level at the HWY 99 Bridge would be lowered by approximately 1.1 feet due to the diversion.

Figure 15. Modeled Diverted Flow Hydrograph for Reducing the 2024 event flow to prevent flooding at Upper Diversion Dam



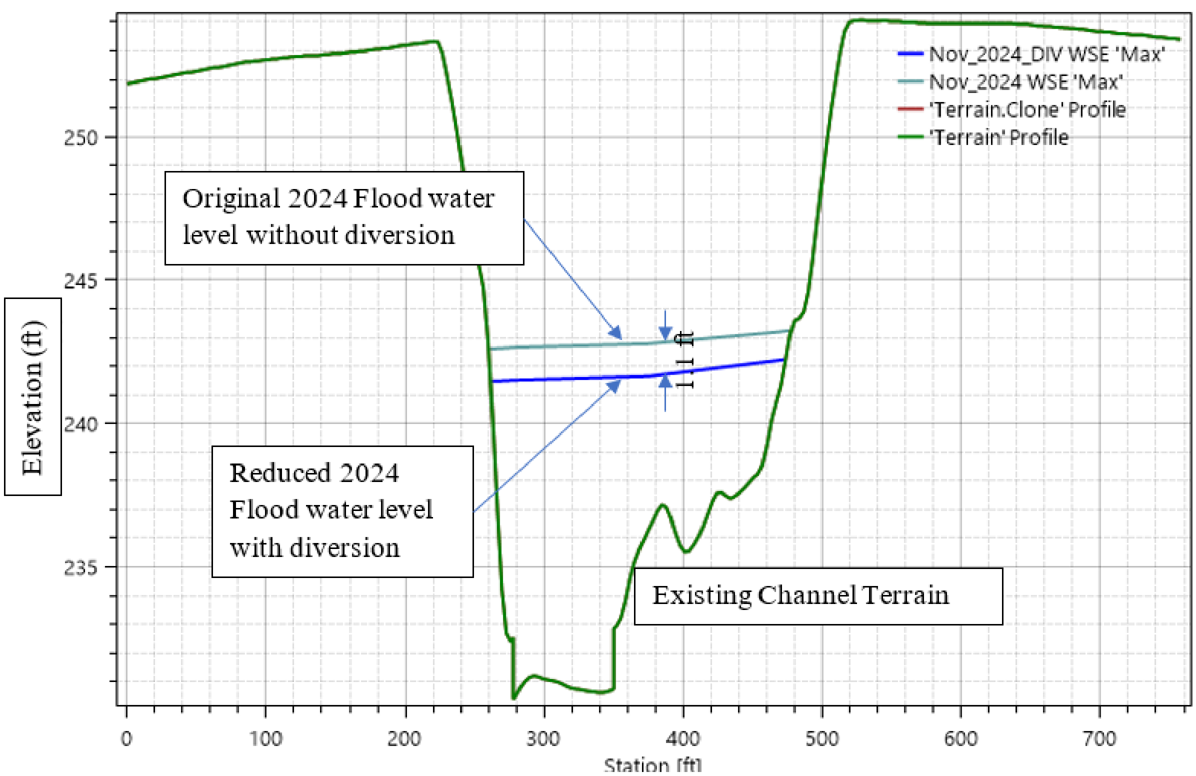
Source: HEC-HMS

Figure 16. Modeled Flow Hydrograph Reduction for the 2024 Event at the Cross Section at HWY 99 Bridge



Source: HEC-RAS

Figure 17. Modeled Stage Reduction with and Without Diversion for the 2024 Event at the Cross Section at HWY 99 Bridge



Source: HEC-RAS

Estimated Water Availability

The California State Water Resources Control Board (State Water Board) describes two methods to estimate water availability in the *Water Availability Analysis for Streamlined Recharge Permitting Guidelines* (Guidelines) dated 2023. These methods, Method 1 and Method 2, provide simplified approaches for demonstrating water availability.

This report focuses on Method 2: Threat of Flood Conditions, which defines water availability based on flood risk. For this analysis, the flood diversion threshold was set at the 10-year flood event, measured as a peak flow of 12,222 cfs at USGS gage 11381500.

Method 1: the 90th Percentile/20 Percent Method, allows for diversion of 20 percent of the daily flow when flows exceed the 90th percentile daily flow, provided that sufficient water remains in-stream to satisfy senior water rights.

Table 6 provides estimates of water available for recharge applying these two methods based on the historical stream gage record of Mill Creek. The last column in the table represents the total number of years in the record that the annual diversion happens. Table 7 lists annual statistics of the number of days that water can be diverted out of the channel for all water year types. The definition of water year types was determined by DWR based on Sacramento Valley (WSIHIST).

Table 6. Annual Recharge Volumes Available Based on Methods 1 and 2 based on USGS Gage 11381500 at the Mill Creek

Method		Min, AF	Max, AF	Average (including years with no volume available), AF	Percent of Years with Volume greater than 0 acre-ft
Method 1	90th Percentile/20 Percent*	54	27,897	7,345	95
Method 2	Threat of Flood Conditions**	210	5,116	178	11

Notes: AF = acre-feet

*The gage daily flow records are from 1928 to June 2025. 90th percentile table was obtained from USGS gage website USGS Surface Water data for USA: USGS Surface-Water Daily Statistics.

**The gage 15-min flow records are from 1988 to June 2025. Assumes any flow exceeding the flood diversion threshold (12,222 cfs) will be diverted.

Table 7. Annual Days Diversion Available Based on Method 1 at Mill Creek

Water Year Type	Days Water Available for Diversion			No. Years
	Min	Max	Average*	
Wet	1	39	22	29
Above Normal	2	29	12	17
Below Normal	1	18	8	13
Dry	1	22	5	16
Critical	1	11	4	22
All Years	1	39	11	97

*For each water year type, the average values calculations include all years, including those with zero diversion. However, the minimum estimates did not include years with zero diversion.

6. Conclusion and Next Steps

This evaluation utilized the latest available high-resolution terrain and stream gage data to identify higher frequency flood events that could result in road closures and property impacts within the Lower Mill Creek Watershed. Analysis showed that, in addition to diverting the maximum flow using two existing diversion dams, an additional 2995 cfs would need to be diverted to reduce the 2024 event (an approximate 19-year frequency event) to a water level below the defined flood diversion threshold of 12,222 cfs. Water would be diverted for about 2.6 hours to keep the flow in Mill Creek from exceeding the defined flood diversion threshold of 12,222 cfs.

Flood conditions happen infrequently with an average annual recharge volume of 178 af (assuming all the available volume can be used). The Method 1 90th percentile/20 percent average annual volume and years available are much greater and likely enough larger to make pursuing long term flood diversion water right permits a viable option in addition to diverting as much flow as possible under Method 2.

The following information is recommended for inclusion in the flood diversion guidelines:

- Properties at 24865 and 24939 Ellis Street are under imminent threat of flooding by flows over the flood diversion threshold of 12,222 cfs (Figure 10).
- Diversions under California Water Code §1242.1 at any location along the entire geographic extent of the stream of Mill Creek would be expected to reduce an imminent flooding threat.
- The flow threshold (flood diversion threshold) of 12,222 cfs at USGS Gage 11381500 is associated with the imminent threat of flooding.

Next steps for Mill Creek include:

1. Complete analysis to determine land suitability and availability for recharge near the two existing diversion dams (i.e., Upper Diversion and Ward Diversion) and along both sides of the creek between the dams.
2. Calculate total diversions and capacity of the Mill Creek upstream of the flooding area.
3. Conduct initial screening of additional diversions off the Mill Creek upstream of flooding area.
4. Complete final review of hydrologic and hydraulic models
5. Review and develop alternatives to rank potential solutions for flood risk reduction and groundwater recharge locations.

7. References

State Water Resources Control Board (State Water Board) Water Availability Analysis for Streamlined Recharge Permitting Guidelines. https://www.waterboards.ca.gov/waterrights/water_issues/programs/applications/groundwater_recharge/docs/streamlined_waa_guidance.pdf Sacramento CA.

U.S. Fish and Wildlife Service. 2000. Draft Finding of No Significant Impact. Anadromous Fish Restoration Actions in Lower Mill Creek Tehama County, California. Sacramento, CA: U.S. Fish and Wildlife Service.

ATTACHMENT C

Technical Memorandum

11010 White Rock Road, Suite 200 ● Rancho Cordova, CA 95670 ● 916.631.4500

Via Email: pdhaliwal@lsce.com, wanderson@lsce.com

To: Will Anderson and Pavan Dhaliwal, Luhdorff & Scalmanini Consulting Engineers
Tehama County Flood Control and Water Conservation District, California

From: Bryan Thoreson; Yi Shen (GEI)

cc: Chris Ferrari (GEI)

Date: November 19, 2025

Re: Hydrologic and Hydraulic Model Approach and Evaluation for the Antelope Creek Groundwater Recharge and Flood Reduction Project

Project No. 2403778

1. Introduction and Purpose

Antelope Creek originates in the southwestern Cascade Range and flows generally to the west, and then to the south to its confluence with the Sacramento River near Red Bluff, California (Figure 1). In this study, Antelope Creek is divided into two reaches (Upper Antelope Creek and Lower Antelope Creek) for the purpose of modeling flood reduction. Upper Antelope Creek is confined within a narrow steep-sided canyon and ends at Edwards Diversion Dam. Starting from Edwards Diversion Dam, Lower Antelope Creek flows to the south through agricultural land before entering the Sacramento River, on the north boundary of Los Molinos, in northern California. Lower Antelope Creek has flowed through four major channels over time: New Creek, Antelope Creek, Craig Creek, and Butler Slough (Figure 2).

The purpose of this Antelope Creek Technical Memorandum (TM) is to support an investigation into the feasibility of diverting surface water from Antelope Creek for groundwater recharge to help alleviate flooding. This TM discusses the following.

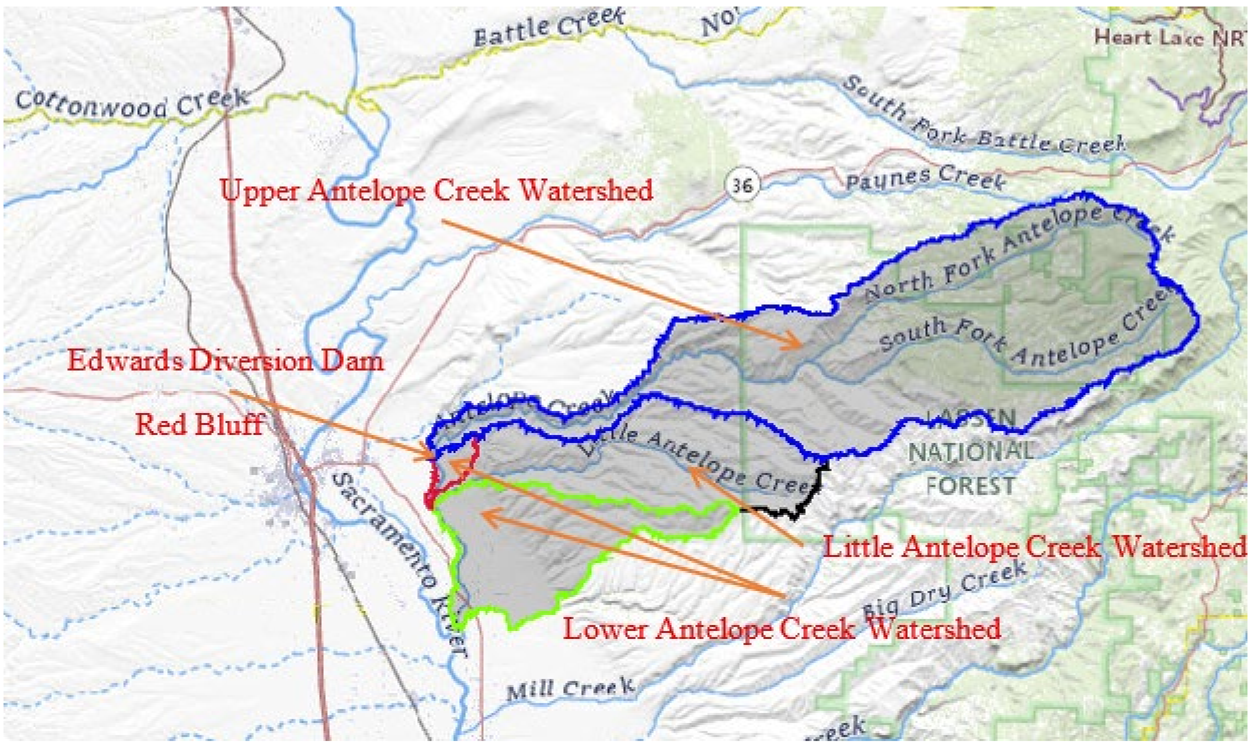
- Hydrologic (HEC-HMS)[1] and hydraulic (HEC-RAS)[2] model development for the Antelope Creek Watershed
- Utilizing U.S. Geological Survey (USGS) Terrain
- Validating/calibrating the hydrologic and hydraulic models at the USGS gage 11379000 – Antelope Creek near Red Bluff, CA (USGS gage)
- Hydraulic model floodplain results based on statistical 2- to 100-year storm events.

[1] HEC-HMS refers to the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's (HEC) Hydrologic Modeling System (HMS)
[2] HEC-RAS refers to USACE's Hydrologic Engineering Center's (HEC) River Analysis System (RAS)

- Developing a Flood Diversion Threshold following the State Water Resources Control Board Code 1242.1 guidelines[3] (State Water Board 2025)
- Estimating potential groundwater recharge volume from diversions under State Water Board Code 1242.1 (2025) guidelines declared flood emergency.

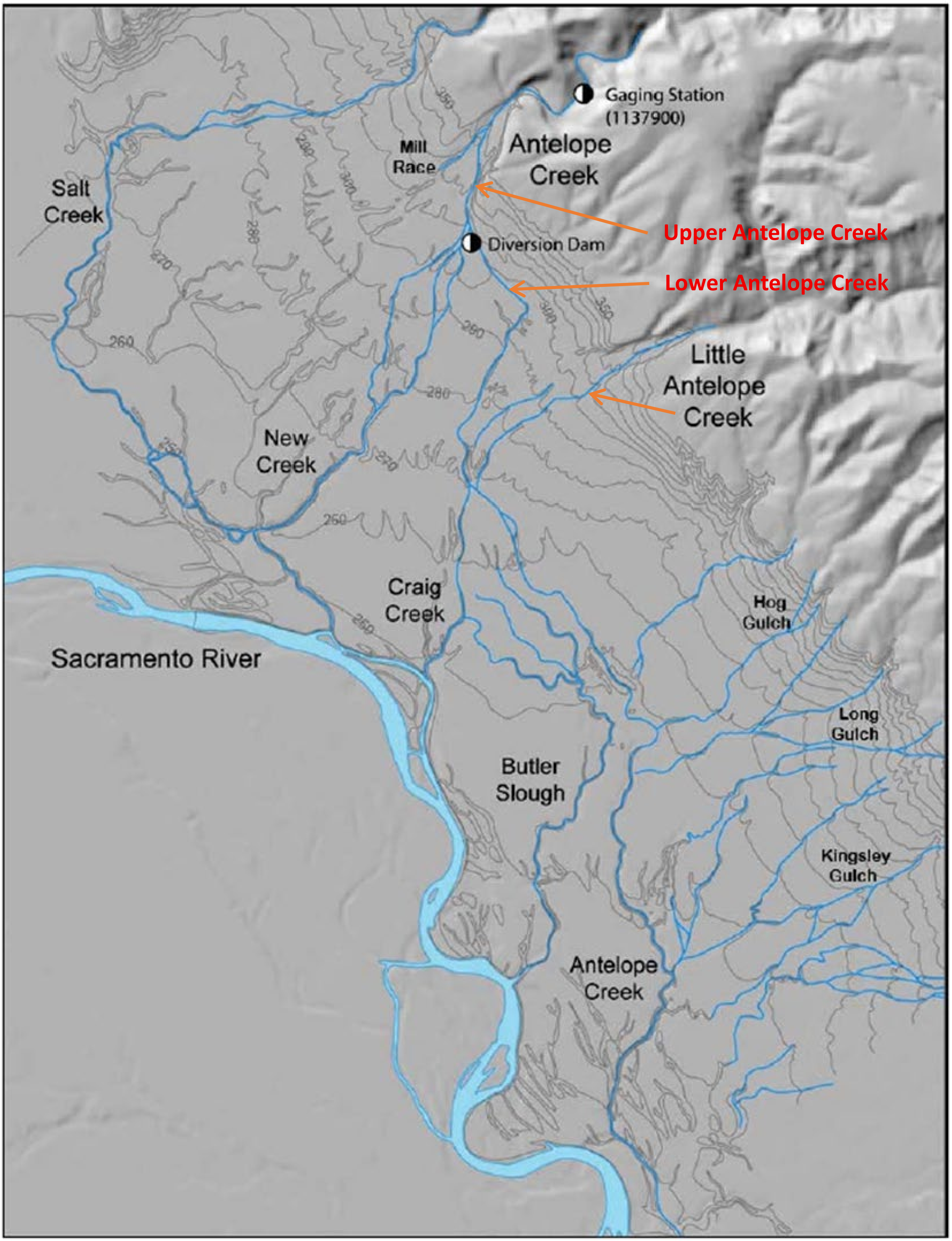
As flood events become more frequent, being prepared for flood diversions and groundwater recharge is crucial for sustainable water management. State Water Board Code §1242.1 (2023) allows parties to divert flood flows for groundwater recharge without a water right if in compliance with certain requirements. The goal of this TM is to describe the steps which developed results to inform and forecast flood flow and flood extents for local emergency managers. Building on the existing conditions model discussed in the previous section, this TM also evaluated project alternatives for diverting flows to recharge groundwater and presents the estimated flood reduction results.

Figure 1. Watershed of Antelope Creek



[3] https://www.waterboards.ca.gov/waterrights/water_issues/programs/groundwater-recharge/recharge-diversions.html

Figure 2. Drainage Network in Antelope Creek Watershed (Gaging Station is USGS gage 11379000)



Source: Stillwater Sciences 2014 (Figure 2-7)

2. Background

The Antelope Creek Watershed, located in Tehama County on the east of the Sacramento River, spans approximately 197 square miles (Figure 3). Elevations in the Watershed range from 6,790 feet in the Lassen National Forest to 200 feet near the Sacramento River. Main tributaries, including North Fork Antelope Creek, South Fork Antelope Creek, and Little Antelope Creek, converge with the main channel before it enters the Sacramento Valley. The Lower Antelope Creek Watershed includes the mainstem of the Antelope Creek and three distributary streams. Below Edwards Diversion Dam, flow from Antelope Creek splits into New Creek, which subsequently branches into Craig Creek and Butler Slough Figure 3).

Antelope Creek has a natural hydrologic pattern characterized by high flows and peak runoff events during the winter months, and low flows in the summer and fall. The largest recorded peak runoff event occurred in December 1969, reaching 17,160 cubic feet per second (cfs). The primary diversion structure on Antelope Creek is the Edwards Diversion Dam, which supplies irrigation water for the Los Molinos Mutual Water Company (LMMWC) and the Edwards Ranch. According to the State Water Board, (Stillwater Sciences, 2014), LMMWC is authorized to divert up to 80 cfs, while Edwards Ranch can divert up to 50 cfs. The watershed does not contain any storage dams or reservoirs. However, significant groundwater resources in the lower elevation areas of Lower Antelope Creek support both domestic and agricultural water supplies. The only long-term streamflow monitoring station on Antelope Creek is USGS gage 11379000, located approximately 1.2 miles upstream of the Edwards Diversion Dam, near the transition from bedrock canyon to the more open Sacramento Valley floor (Figure 4). The peak streamflow data was measured from 1937 to 1981. However, USGS stopped recording after 1981. At present, this gage is being maintained by the California Data Exchange Center (CDEC) under the site name of ATC, Antelope Creek near Red Blugg. However, no high flow records exceeding 440 cfs have been reported so far.

Figure 3. Subbasins in the Antelope Creek Watershed

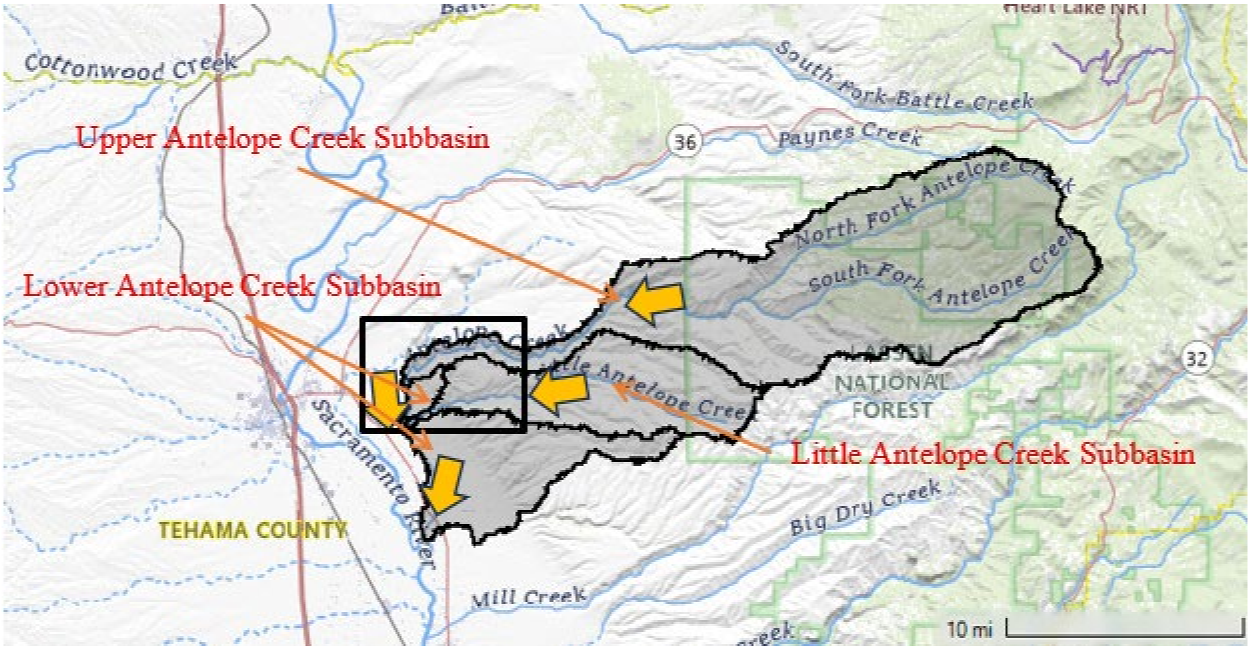
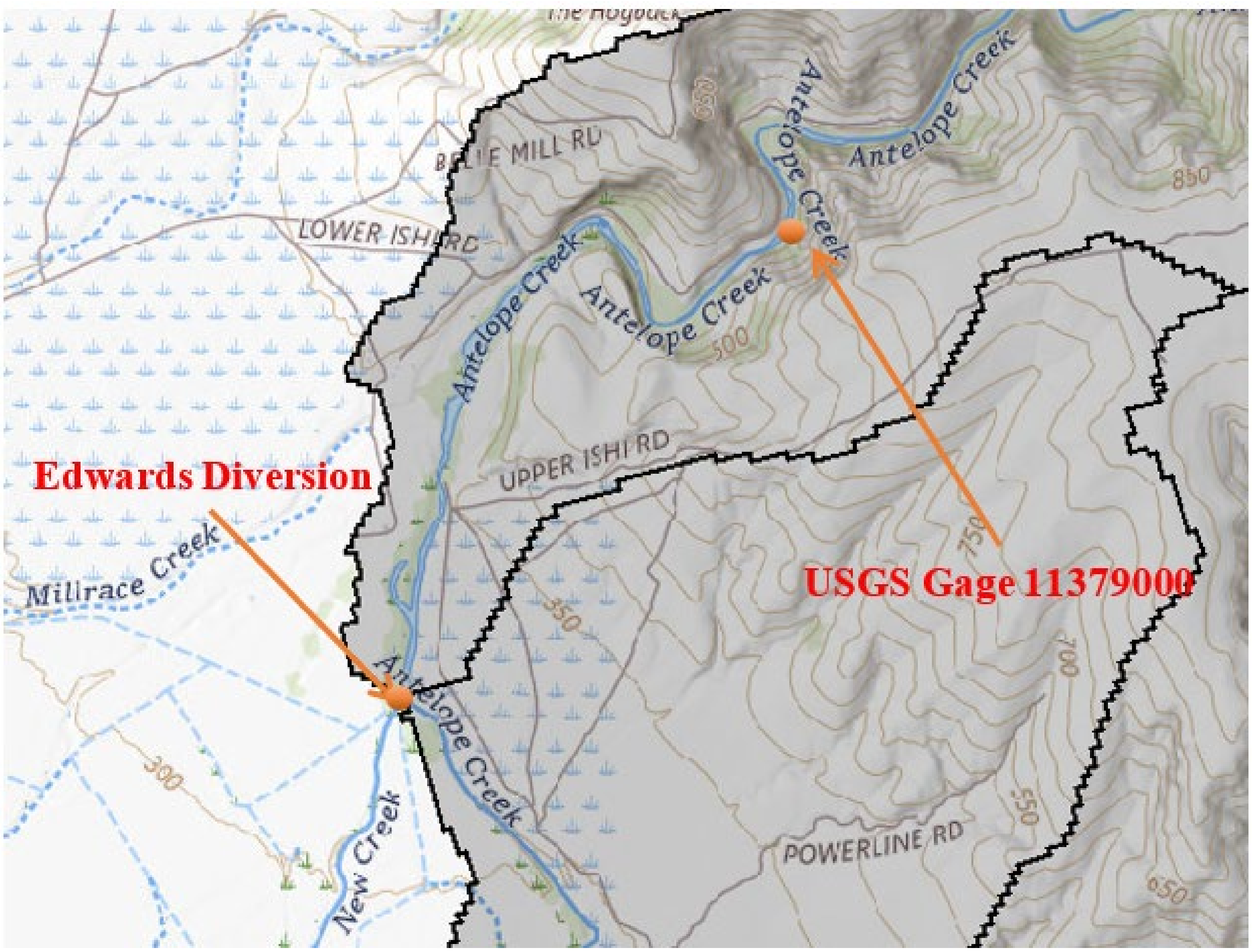


Figure 4. Existing Diversion and Stream Gage in the Antelope Creek Watershed



3. Hydrologic and Hydraulic Models Development

This section discusses the methods used to set up the hydrologic (HEC-HMS) and hydraulic (HEC-RAS) models. These models will then be utilized to evaluate existing conditions, assess various diversion alternatives, and forecast flood flows in Antelope Creek using gage data.

3.1. HEC-HMS Model Development

The hydrologic model (HEC-HMS) was developed using HEC-HMS 4.13 software. For this study, a 10-meter resolution Digital Elevation Model (DEM) was downloaded from USGS's website and applied to the hydrologic model domain. Rainfall information required by the hydrologic model was obtained from National Oceanic and Atmospheric Administration (NOAA) website. Specifically, NOAA Atlas 14 hypothetical design storms for return periods of 2-, 5-, 10-, 50-, and 100-year were downloaded in a gridded format and used as the total rainfall amount for a 24-hour period. The designed 24-hour temporal distribution of the rainfall was also downloaded from NOAA's website.

The Antelope Creek Watershed was divided into four subbasins that can contribute runoff to downstream watersheds. These include the Upper Antelope Creek, the Little Antelope Creek, and two Lower Antelope Creek basins. Figure 3, above, shows the Watershed boundaries along with the surrounding terrain.

- The Upper Antelope Creek Subbasin (124 square miles) captures flows from the glaciated slopes of Mount Lassen down to the confluence with Little Antelope Creek.
- The Little Antelope Creek Subbasin (39 square miles) represents runoff contribution from Little Antelope Creek.
- The two Lower Antelope Creek subbasins (2 and 31 square miles) encompass the area downstream of the confluence of Upper and Little Antelope creeks, extending to the Sacramento River.

Precipitation loss due to infiltration was calculated using the Soil Conservation Service (SCS) Curve Number loss method, developed by the SCS curve number is a widely used empirical method that estimates direct runoff volume from a rainfall event based on land use, soil type, and antecedent moisture conditions. The Curve Number was estimated for each subbasin from the soil drainage characteristics and land use. The SCS Unit Hydrograph approach was used for runoff modeling in this study. This method requires specification of each subbasin's lag time (L) and Peak Rate Factor. The L parameter for each subbasin was estimated as a function of flow path length, maximum potential retention and average watershed land slope. Table 1 presents the parameters the model used to evaluate statistical storms and the parameters to calibrate the model for the historical storms.

Table 1. HEC-HMS Model Parameters

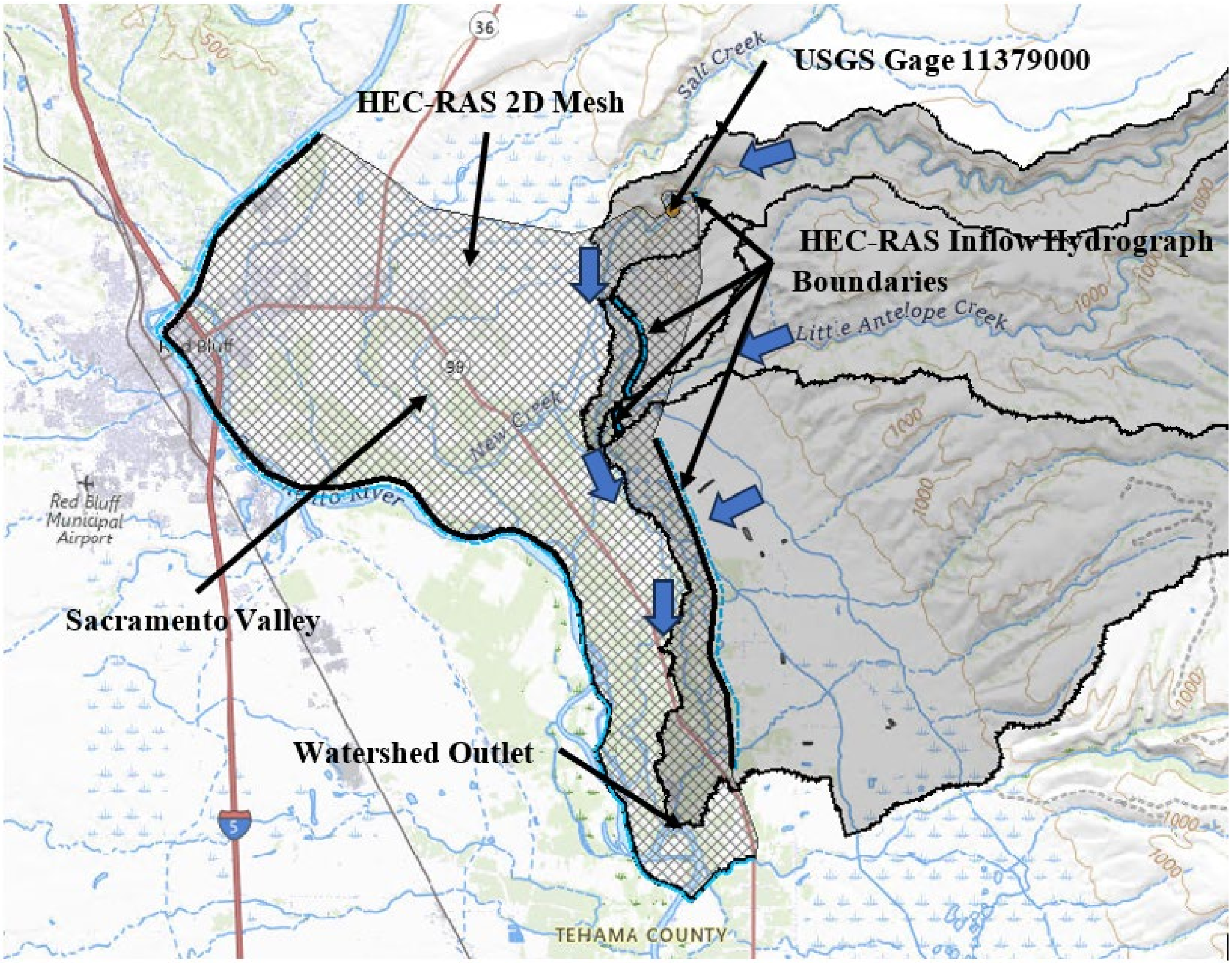
Subbasin	Area (mi^2)	T_{lag} (min)	Peak Rate Factor	Curve Number
Upper Antelope Creek	124.1	470	600	72
Little Antelope Creek	39.2	287	300	87
Lower Antelope Creek 1	2.4	67	250	85
Lower Antelope Creek 2	31.2	192	300	86
Total	**196.9**	N/A		

Notes: min = minute; mi2 = square miles; N/A = not applicable; T_{lag} = minutes

3.2. HEC-RAS Model Development

The hydraulic (HEC-RAS) model for this study was developed using HEC-RAS 6.6 software. A single two-dimensional (2D) mesh was created in HEC-RAS, which not only covers the Watershed outlets, but also part of the downstream Sacramento Valley (Figure 5). For the Upper Antelope Creek Basin, the peaks of the flow hydrographs at the HEC-RAS Upstream Inflow Boundary were calculated from HEC-HMS at its downstream subbasin outlet but then were adjusted using the drainage ratio method. The drainage area ratio method is a way to estimate streamflow at the watershed outlet by multiplying the streamflow at the gage by the ratio of the drainage areas of the outlet and gage sites. For the Little and Lower Antelope Creek basins, their HEC-HMS calculated flow hydrographs were evenly distributed along the entire river segments (Figure 5). The temporal distributions of the flow hydrographs followed the HEC-HMS results at the Watershed outlet.

Figure 5. HEC-RAS Model 2D Mesh and Upstream Boundary Inflows



4. Model Results and Findings:

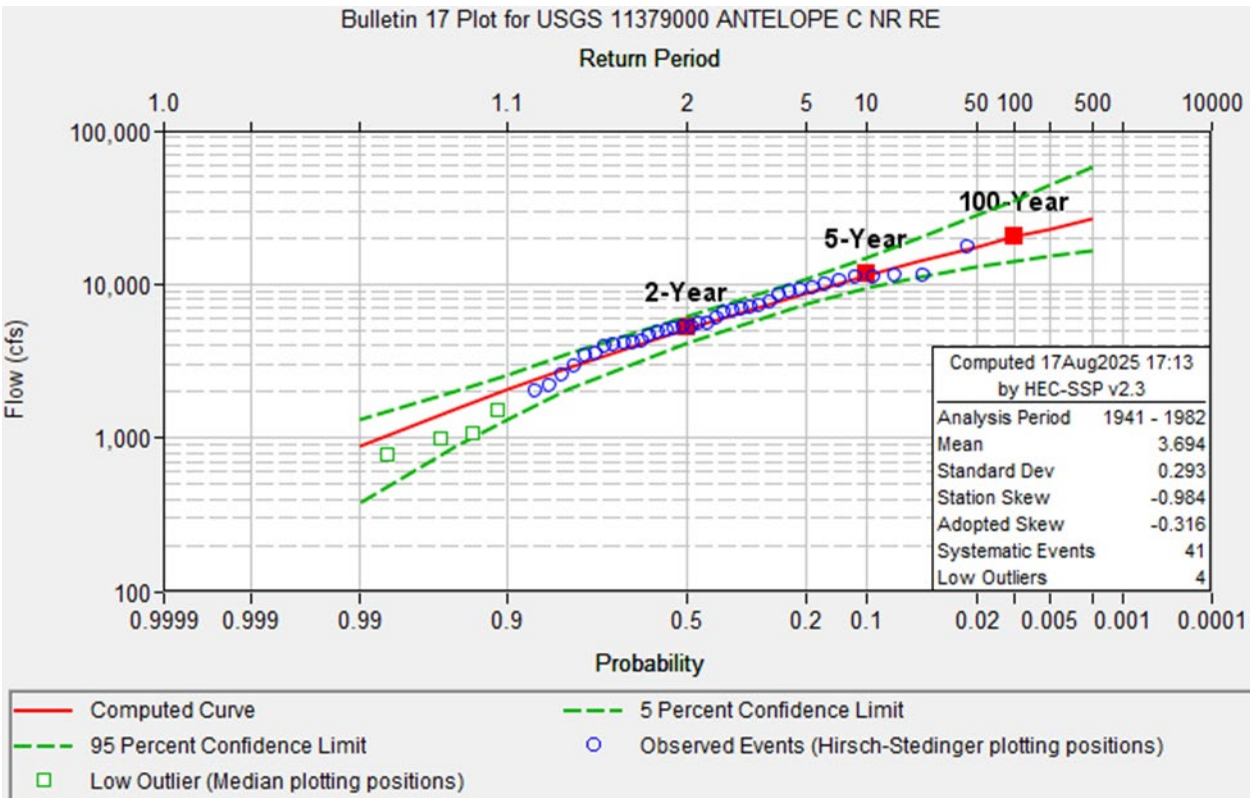
This section discusses the hydrologic (HEC-HMS) and hydraulic (HEC-RAS) model evaluation:

- The hydrologic (HEC-HMS) model flow hydrograph was not calibrated to the USGS gage, because it stopped recording high flow records after 1982. However, the peak flow data from the USGS gage for three statistical storm events: 2-, 10-, and 100-year, was used to calibrate the HEC-HMS model.
- The hydrologic model (HEC-HMS) was used for four statistical design storms for the 2-, 5-, 10-, and 50-year storm frequencies. All the model parameters were calculated using formulas recommended by HEC-HMS user manual.
- The hydraulic model (HEC-RAS) was used to calculate stage hydrographs and flood inundation maps for all the events.

4.1. Gage Frequency Analysis

Figure 6 shows the flood frequency analysis results using the HEC-SSP software at the USGS gage. The computed curve was based on 41 years of annual peak flows from the gage record from 1941 to 1981. The red dots on Figure 6 represent hydrologic (HEC-HMS) model peak flows for the 2-, 10-, and 100-year return periods computed from the HEC-HMS software for comparison. The percent error between the USGS gage frequency results and the hydrologic model for the three peak flow events was less than 10 percent. Table 2 provides the peak flow estimates at the USGS gage from the HEC-SSP analysis.

Figure 6. Flood Frequency Results at USGS Gage



Source: HEC-SSP

Table 2. HEC-SSP Statistical Peak Flow Estimates on Antelope Creek at USGS Gage 11379000

90% Exceedance	2-Year (cfs)	5-Year (cfs)	10-Year (cfs)	50-Year (cfs)	100-Year (cfs)
2,045	5,125	8,789	11,428	17,562	20,249

Notes: % = percent; cfs = cubic feet per second

According to the HEC-HMS model, the peak flow at the USGS gage location at the November 2024 event was calculated at 8,879 cfs, which corresponds approximately to a 5-year (20% annual chance storm) frequency event for the Upper Antelope Creek. Table 3 lists the peak flows at various frequencies, as calculated by the HEC-HMS model at the outlets of the four subbasins (not the USGS gage location).

Table 3. HEC-HMS Statistical Peak Flow Calculated at the Four Subbasins of the Antelope Creek Watershed

Subbasin	2-Year (cfs)	5-Year (cfs)	10-Year (cfs)	50-Year (cfs)	100-Year (cfs)
Upper Antelope	5,062	8,914	11,489	17,398	19,959
Little Antelope	2,019	3,110	3,827	5,473	6,195
Lower Antelope #1	120	186	231	336	383
Lower Antelope #2	1,610	2,470	3,041	4,373	4,966

Note: cfs = cubic feet per second

4.2. HEC-HMS Model Verification for the Design Storms

The hydrologic (HEC-HMS) model verification results are summarized in Table 4. Overall, the model demonstrated strong performance in predicting peak flows, with all modeled errors remaining below 2 percent compared to gage-recorded statistics. However, no calibration or validation was conducted using historical high-flow events, as the gage does not record flows exceeding 440 cfs.

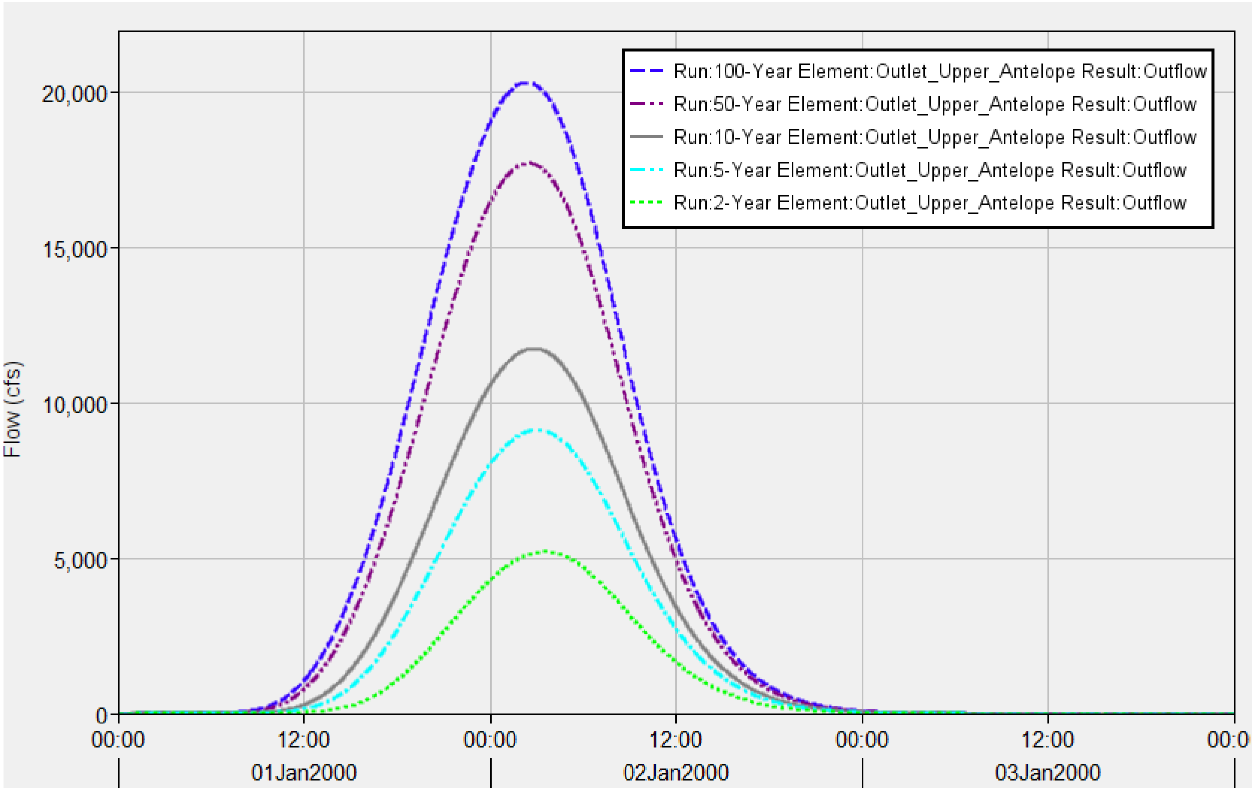
Comparisons of the flow hydrographs at the Upper Antelope Watershed outlet are shown in Figure 7.

Table 4. HEC-HMS Model Calibration Summary at the USGS Gage

	Statistical Design Storm		
	100-Year	10-Year	2-Year
Statistical Peak Flow (cfs)	20,249	11,428	5,125
Modeled Peak Flow (cfs)	20,111	11,625	5,161
Percentage Difference	-0.9%	1.3%	-0.2%

Note: cfs = cubic feet per second

Figure 7. Upper Antelope Creek Flow Hydrographs



Source: HEC-HMS

4.3. November 2024 Flood Event Impact on Lower Antelope Creek Watershed

The HEC-RAS model results predicted the November 2024 flood event could cause road closures, property damage, and restricted access on the west floodplain of Antelope Creek, downstream of the Edwards Diversion Dam. The impacted floodplain also includes the basins of Craig Creek and Butler Slough, which are two distributaries of the Antelope Creek.

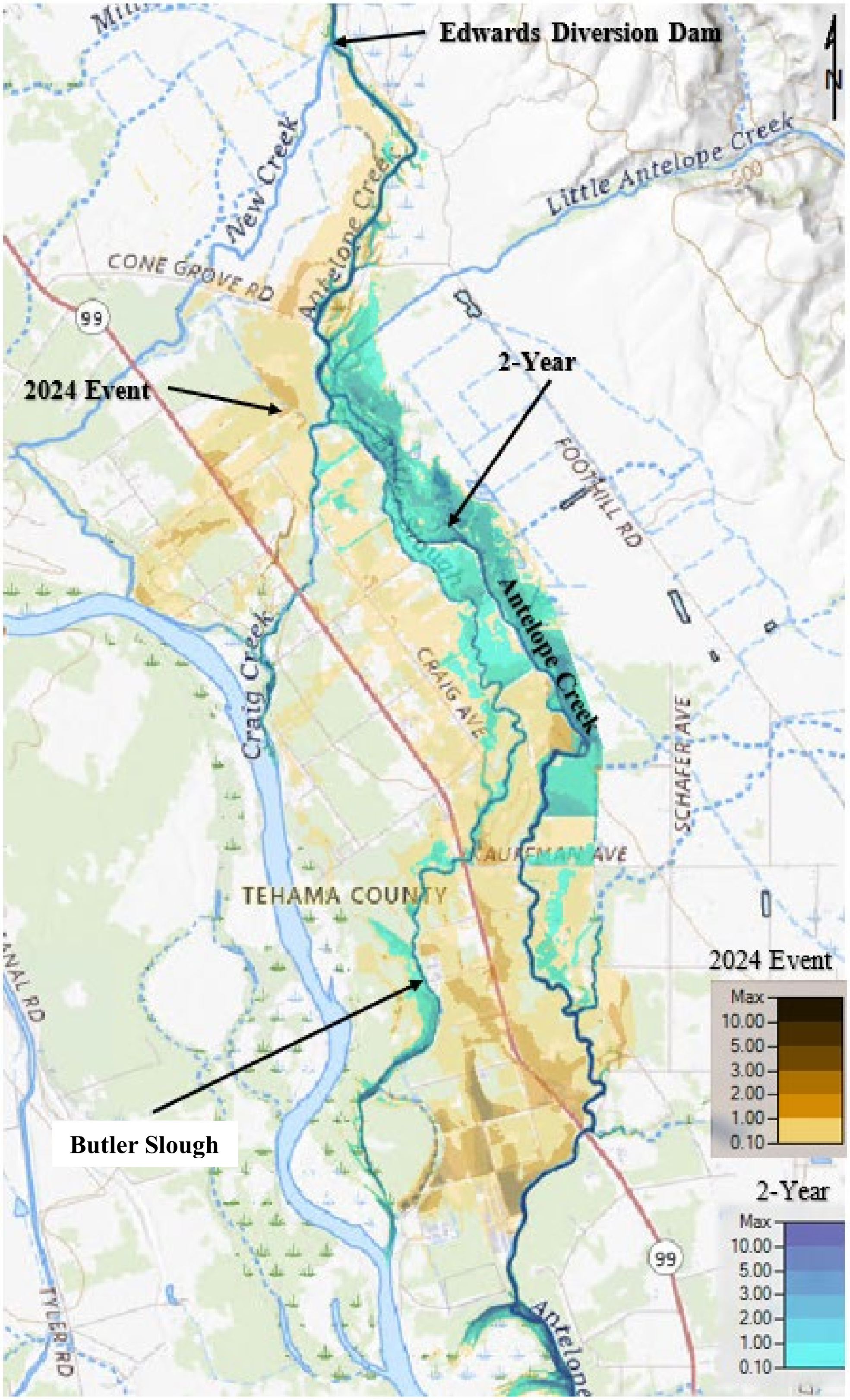
Figure 8 shows the comparison between the 2-year and the 2024 historical storm (an approximate 5-year event for the Upper Antelope Creek). This is because the 2-year flow is the lowest flow that had an imminent thread of flooding on local properties.

According to the State Water Board Code 1242.1 guidelines (2025), flood flows are defined as, “Surface water that has escaped from or is imminently likely to escape from a channel or water body causing or threatening to cause inundation of residential or commercial structures, or roads needed for emergency response.” The 2-year event maximum inundation map shows there is no impact on structures or roads for emergency services, except a few agriculture lands. However, during the November 2024 event, the maximum inundation map indicates many residential properties south of Cone Grove Road, which are located along State Highway 99 (HWY 99) and Craig Avenue would be impacted by floods up to 1 foot of water depth. Possible impacted roads needed for emergency response may include certain segments of HWY 99 near Craig Creek and between Butler Slough and Antelope Creek. Other local roads that were predicted to be flooded during the event included Bray Avenue, Craig Avenue, Kauffman Avenue, Kansas

Avenue, Le Claire Avenue and Conway Avenue between Butler Slough and Lower Antelope Creek (Figure 9).

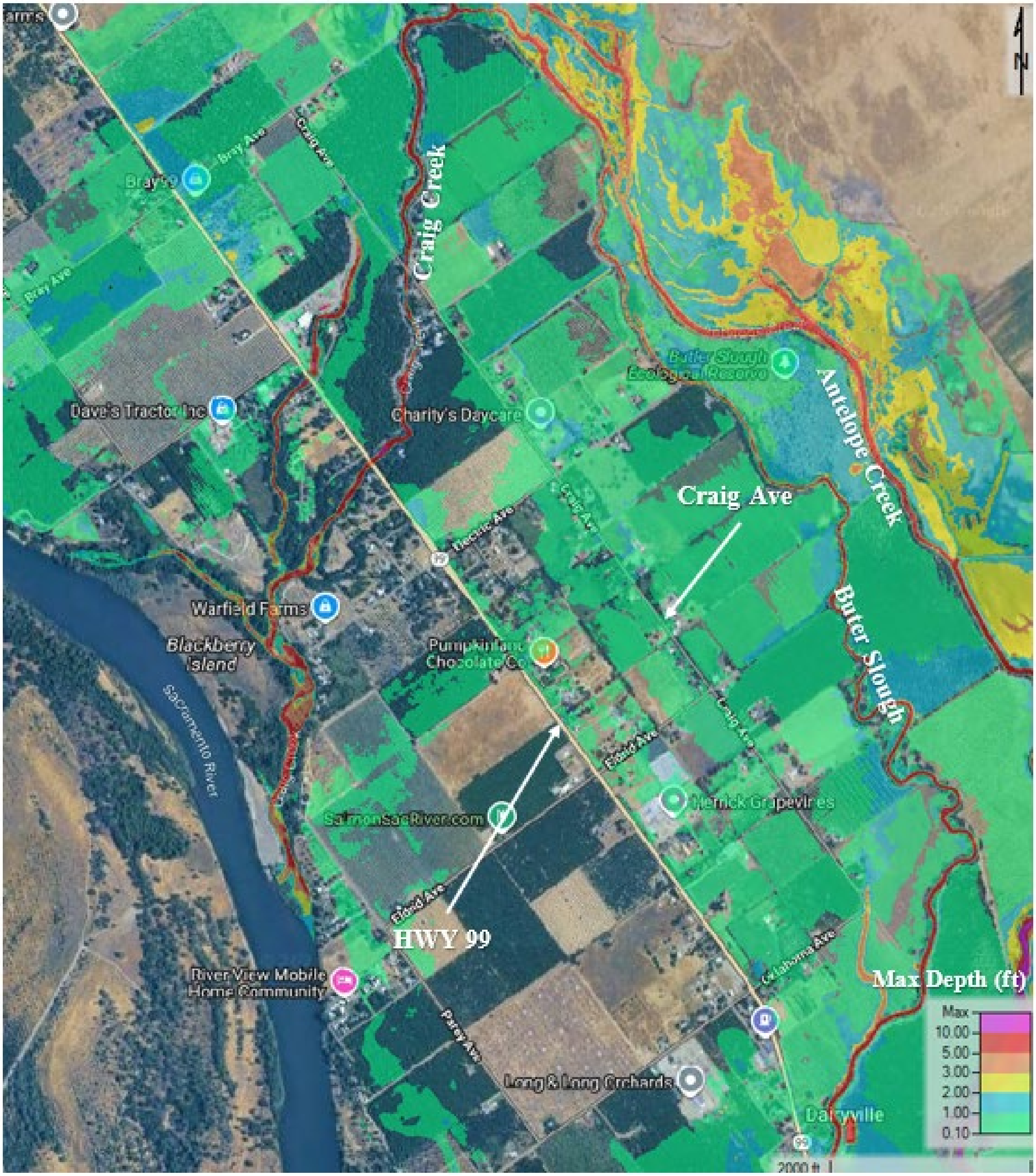
Figures 8 and 9 indicate flooding on the Lower Antelope Creek Watershed would happen at the November 2024 event (5-year), but not at a 2-year event. In other words, any flood with a frequency larger than 2-year period would cause inundation on local properties or road closure. At a flow rate above 5,125 cfs, inundation of residential structures with flood waters from Antelope Creek is imminent (as shown by the November 2024 event). Thus, the flood diversion threshold was set at the peak flow of 5,125 cfs, which is the 2-year event (50% annual chance event) at the USGS gage 11379000.

Figure 8. Max Depth Inundation Comparison Between 2-Year (blue) and the November 2024 Event (brown)



Source: HEC-RAS

Figure 9. Inundation Map on the Residential Area Along HWY 99 and Craig Avenue at the November 2024 Event



Source: HEC-RAS

5. Flood Reduction and Recharge Analysis

To help reduce floodwater levels within the potential inundation area, diversions under State Water Board Code 1242.1 guidelines (2025) would be allowed when flows exceed the flood diversion threshold. Based on a comparison of historical flooding events and results from the Watershed model discussed in the previous section, flooding is expected to begin when the flow at the CDEC gage exceeds approximately 5,125 cfs, corresponding to the 2-year flood event.

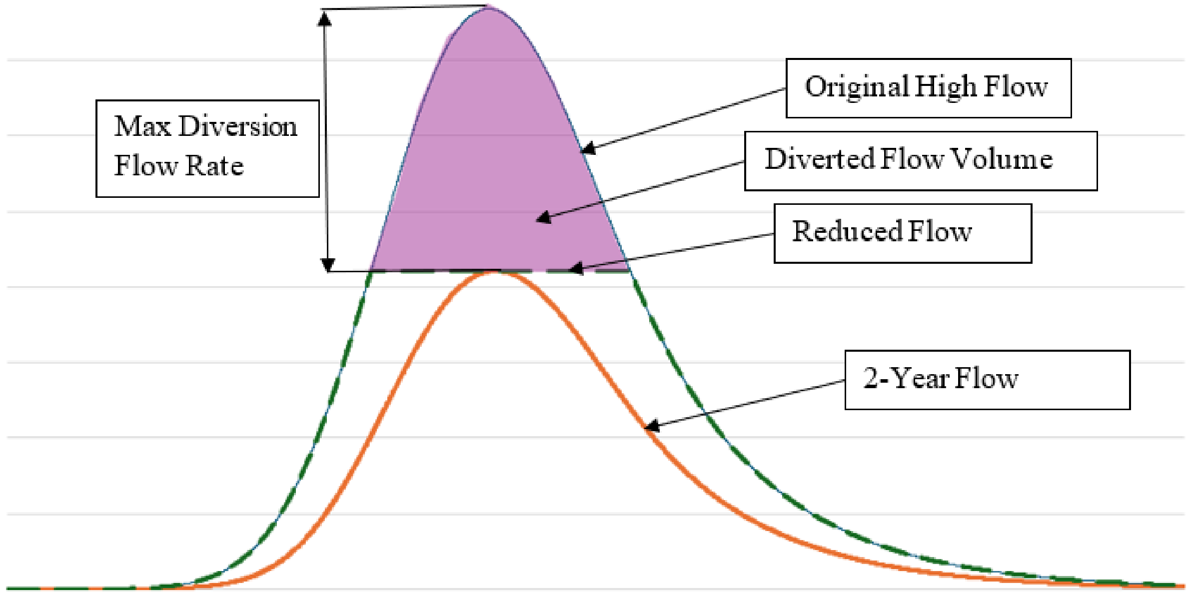
In this study, higher flows with higher return periods were evaluated to estimate the floodwater volume that would need to be diverted from the main channel into a groundwater recharge basin or on to allowable recharge areas. For evaluation purposes, diversions would be made at four proposed water diversion locations (Table 5). The Proposed Diversion #1, upstream of the existing diversion at Edwards Ranch Dam, would prevent floodwater from getting into Millrace Creek and eventually impacting Red Bluff.

Table 5. Proposed Diversion Points at Antelope Creek

Location	Name	Geographic Coordinates	Description
1	Upper Antelope Creek	40.20216, -122.12842	Near Lower Ishi Road, divert flow to prevent flooding in Red Bluff, CA.
2	Edwards Ranch Dam	40.18703, -122.13516	Existing Diversion Structure Edwards Ranch Dam
3	Little Antelope Creek	40.17057, -122.12321	Near Foothill Road, divert flow from Little Antelope Creek
4	Lower Antelope Creek #2	40.15326, -122.12828	0.2 mile downstream of Butler Slough Entrance

- Figure 10 shows the flow hydrograph volume (purple color) that would be removed to reduce the peak flow to the 2-year flow threshold. The purple area represents the total estimated volume required to be diverted out of the channel (Table 6).
- Figure 11 illustrates after the four subbasin drainage peak runoffs and proposed peak diversions at the 5-Year Event.
- Figure 12 presents the flow versus stage rating curve from the HEC-RAS model for a cross section located at Cone Grove Bridge. The result shows the maximum water depth reduction in the channel from a 5-year flow without diversion to a 2-year flow (after diversion).
- Table 6 represents the calculated diverted volume at the Proposed Diversion points. The results show a range of return periods (1st column of the table) to reduce the in-channel water depths to a 2-year water level by diverting flows and volumes (3rd and 4th columns) at the diversion points. The reduced water depths (5th column) after diversion to a 2-year water level were at the Cone Grove Road Bridge. The last column is the estimated area needed to recharge the groundwater within one day assuming a recharge rate of 0.5 foot per day. By spreading water over a larger area or allowing a greater depth of water, water could recharge over a longer period.

Figure 10. Conceptual Flow Reduction Hydrographs



Source: HEC-HMS

Figure 11. Peak Flow Diversions Required for Preventing Flooding from the 5-Year Event on the Antelope Creek

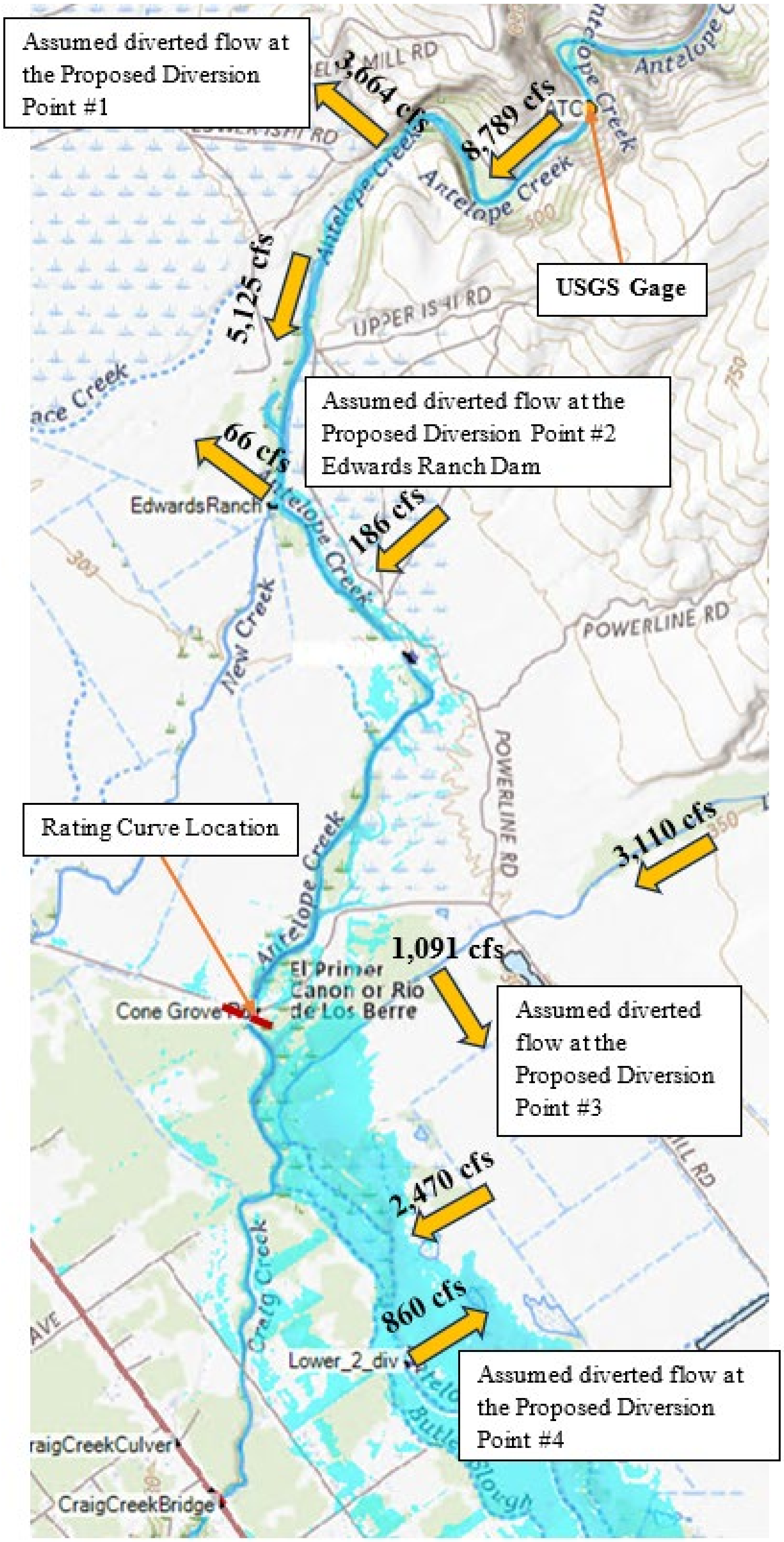
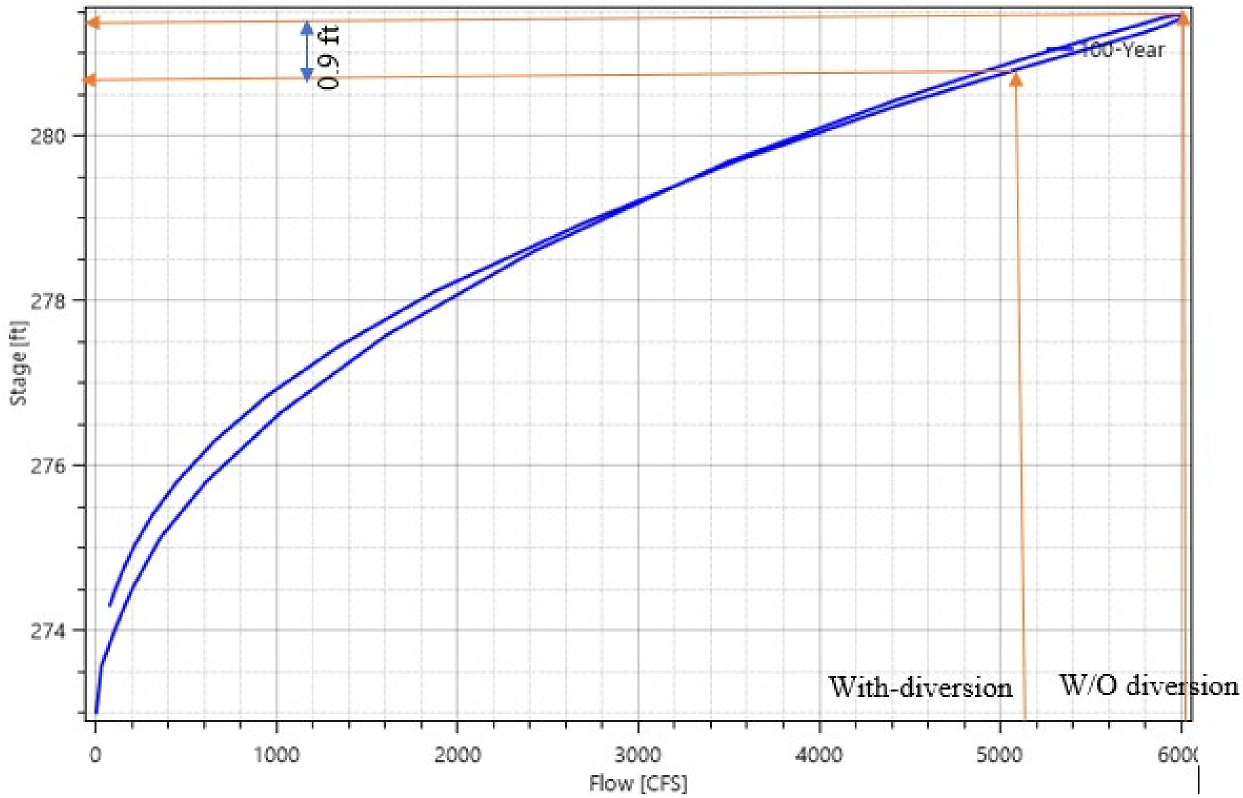


Figure 12. Water Depth Reduction between with- and without-diversion on the Cone Grove Road Bridge on Antelope Creek based on the 5-Year Event



Source: HEC-RAS

Table 6. Estimated Diversions Required to Reduce Flows to Flood Diversion Threshold (2-Year Flood)

Return Period	Peak Flow (cfs) at the USGS gage (from Table 1)	Peak Diversion Flow* (cfs) at the Four Diversion Points				Total Diversion Flow Volume (ac-ft) **	Water Depth Reduction at Cone Grove Road Bridge (ft)	Area Required (acres)
		1	2	3	4			
2	5,125	0	0	0	0	0	0	0
5	8,789	3,790	1,091	66	860	3,587	0.9	7,174***
10	11,428	6,364	1,808	111	1,431	7,182	1.0	14,364
50	17,562	12,273	3,454	216	2,763	16,834	1.1	33,668

Notes: ac-ft = acre-feet / foot; cfs = cubic feet per second

* 1 – Upper Antelope Creek, 2 – Little Antelope Creek, 3 – Lower Antelope Creek #1, 4 – Lower Antelope Creek #2. Due to the size differences among different watersheds, the timing of their peak runoffs and the corresponding required peak diversions are different.

** This is the total water volume required to be diverted out of the river at the diversion points for each event. During a flood event, it is assumed flows in the channel exceeding the peak flow of the 2-year return period threshold would be immediately diverted out of the system into a groundwater recharge basin.

***Assuming a recharge rate of 0.5 foot per day, recharging 3,587 ac-ft of water within 1 day would require 7,174 acres of land. (0.5 foot/day x 7,174 ac =3,587 ac-ft/day)

5.1.1. Example Diversion to Recharge

. In this section, the 2024 event was used as an example to demonstrate the result of diverting flood water from Antelope Creek. The November 2024 event is an approximate 5-year event at the Upper

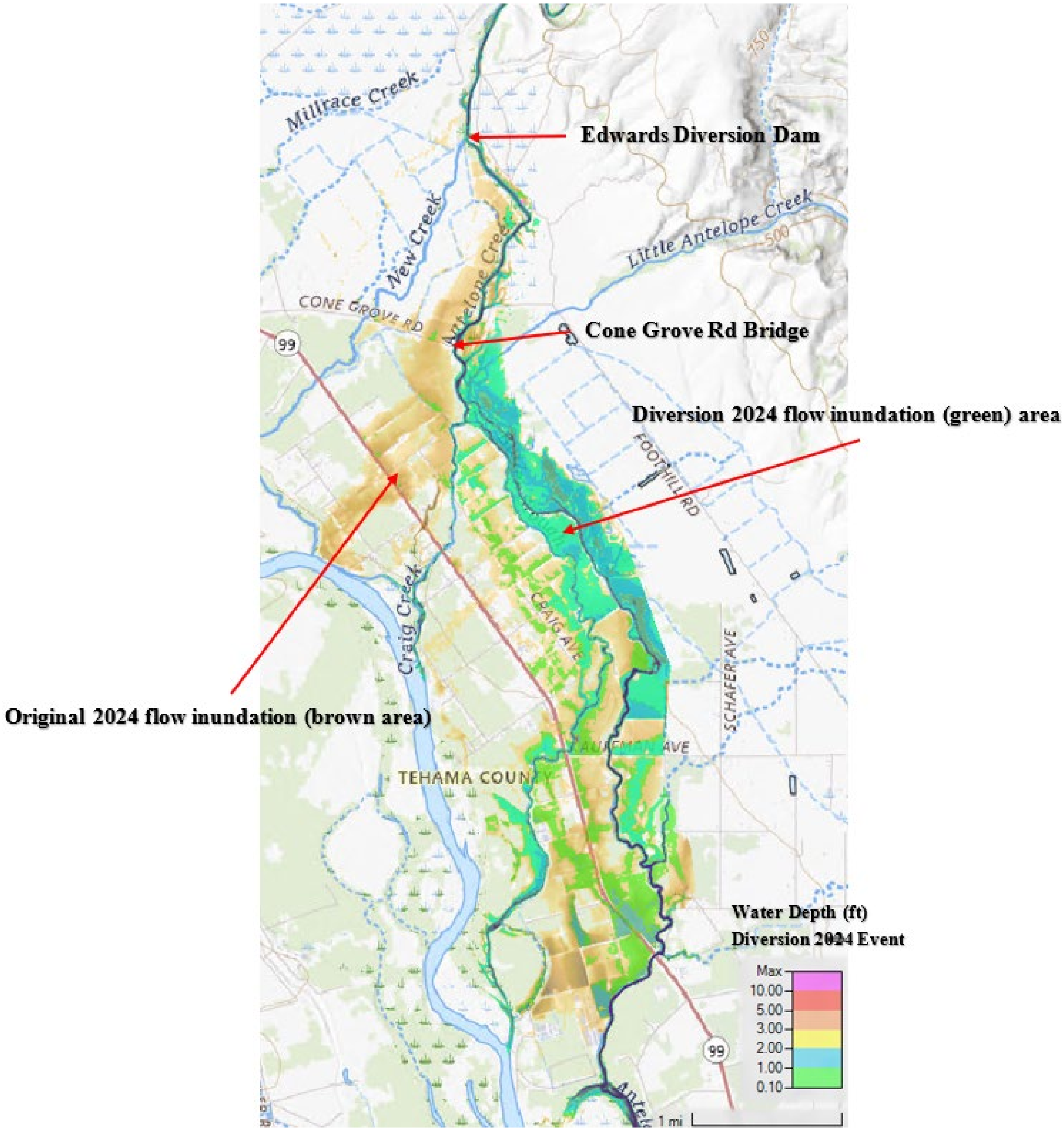
Antelope Creek. The flood frequencies at the Lower and the Little Antelope Creeks are unknown because there is no local gage to calculate their statistics. In this event, all flows above the 2-year peak flow (5,125 cfs) were modeled as being diverted from the Antelope Creek into the groundwater recharge basins during the event period. The maximum peak diversion rate in this event required to prevent flooding was approximately 3,863 cfs at Diversion Point #1; 854 cfs at Diversion Point #2; 25 cfs at Diversion Point #3; and 432 cfs at Diversion Point #4.

As shown in Figure 13, in the Lower Antelope Creek Watershed below the Edwards Diversion Dam, the brown inundation area (no diversions) would be reduced to the green inundation area with the diversion in place. Compared to the 2024 no diversion scenario, no residential or commercial roads or structures would be impacted in the 2024 scenario with diversions. This analysis shows that the diversions would reduce a 5-year event to an approximate 2-year event for the Antelope Creek.

Figure 14 illustrates the proposed flow hydrograph diverted into the groundwater recharge basin during this event period. The total diversion duration is about 19 hours.

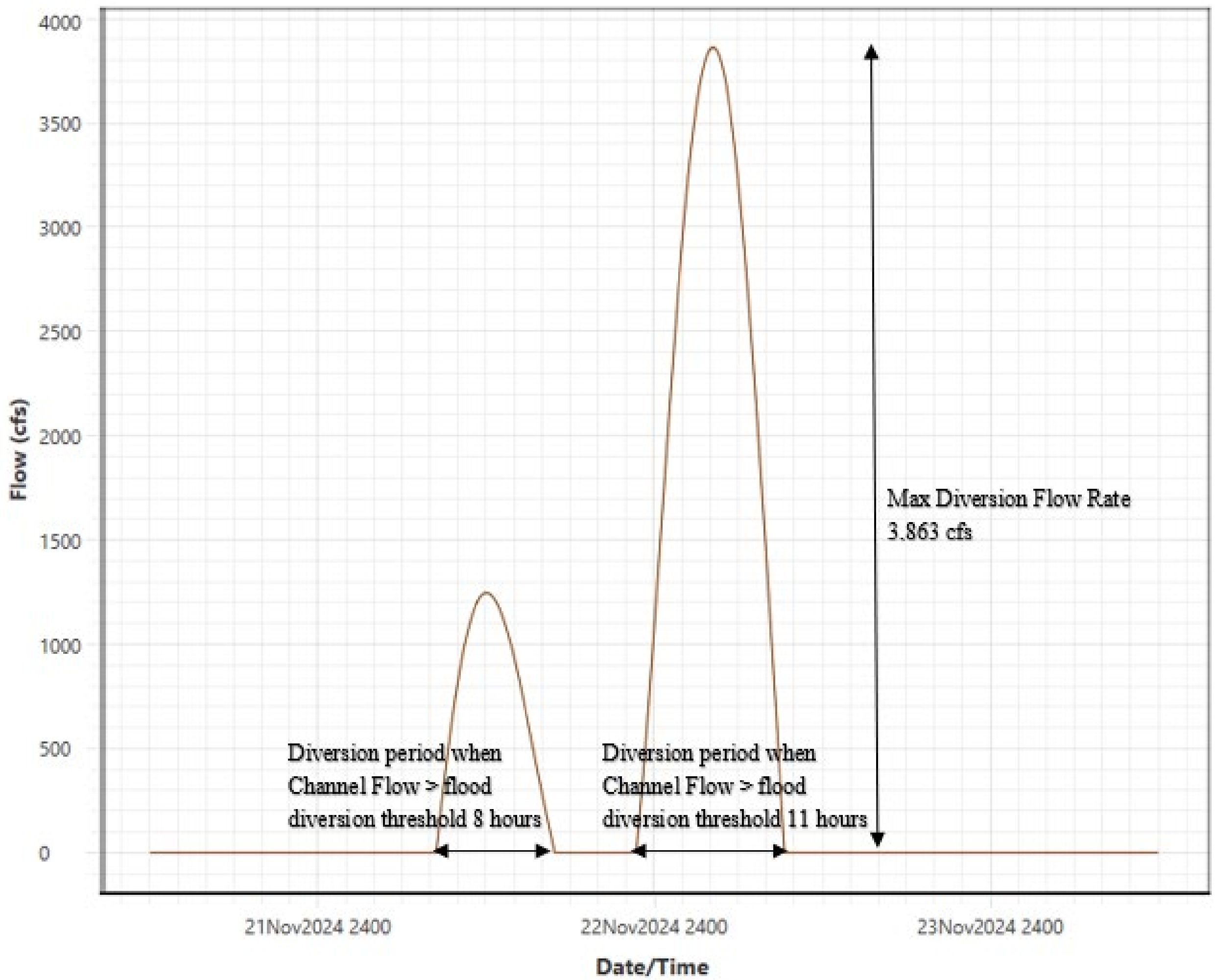
Figure 15 shows the flow hydrograph reduction with and without diversions at the cross section. Figure 16 shows the water level reductions (0.9 ft) at Cone Grove Bridge with diversions.

Figure 13. Maximum Inundation Area Comparison between Original Scenario and Diversion Scenario for the November 2024 Event in the Lower Antelope Creek below Edwards Diversion Dam



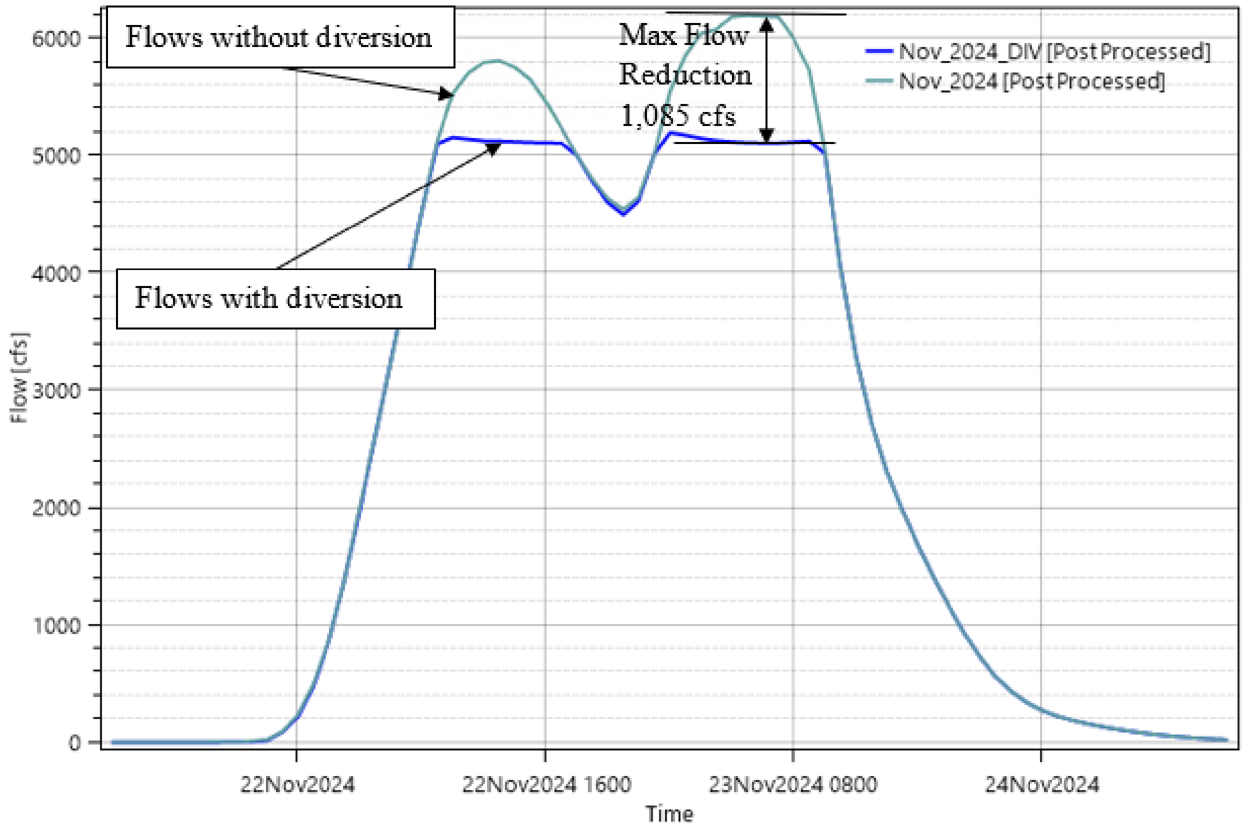
Source: HEC-RAS

Figure 14. Modeled Diverted Flow Hydrograph for Reducing the November 2024 Event Flow to Prevent Flooding at the Proposed Diversion Point #1 Location



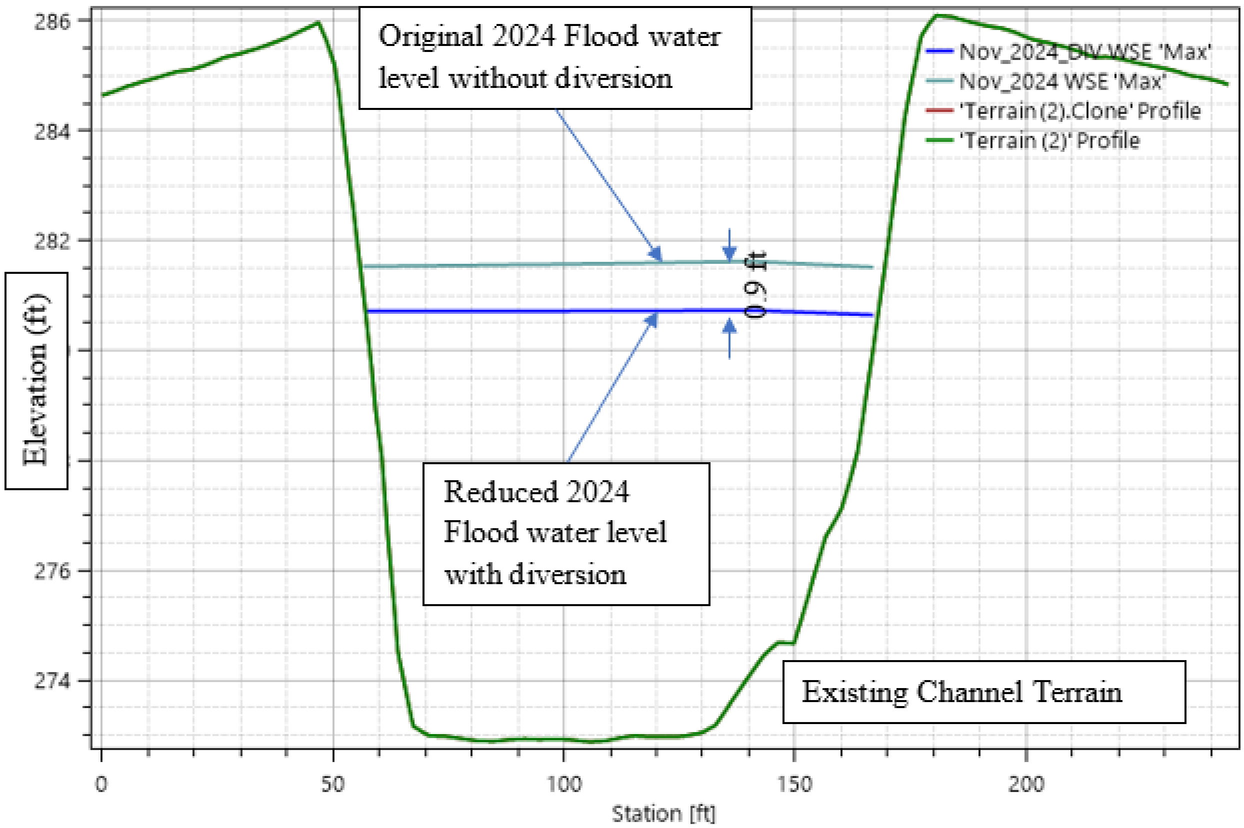
Source: HEC-RAS

Figure 15. Comparison of Flow Hydrograph at the November 2024 Event at the Cone Grove Road Bridge Between the Original Scenario and the Diversion Scenario



Source: HEC-RAS

Figure 16. Comparison of Maximum Stage Hydrographs for the November 2024 Event at the Cone Grove Road Bridge Between the Original Scenario and the Diversion Scenario



5.1.2. Estimated Water Availability

The State Water Board outlines two methods for estimating water availability in its *Water Availability Analysis for Streamlined Recharge Permitting Guidelines* (2017). The analysis in this section focuses on Method 2 - the Threat of Flood Conditions, which defines a flood diversion threshold as the 2-year flood event, based on measurements at the USGS gage. In contrast, Method 1 - The 90th Percentile/ 20 Percent method allows for diversions of up to 20 percent of daily streamflow when flows exceed the 90th percentile of historical daily flow between December 1st and March 31st, assuming sufficient water remains in-stream to satisfy senior water rights.

Table 7 provides estimates of water available for recharge using Method 1, based on the historical daily stream gage record from Upper Antelope Creek. Annual groundwater recharge volumes were not calculated using Method 2 due to data limitations: the USGS gage does not have 15-minute flow records, and the CDEC gage does not record flows exceeding 440 cfs. The last column in the table indicates over the 42 years of records, diversion would be possible for 40 years. Table 8 lists annual statistics of the number of days that water can be diverted out of the channel for all water year types. The definition of water year types was determined by DWR based on Sacramento Valley (WSIHIST).

It is important to note that the November 2024 event is used in this TM solely to demonstrate the flood reduction potential of the proposed diversion plan. However, this event cannot be considered for groundwater recharge under Method 1, as it occurred outside the diversion period limits.

Table 7. Statistics of Annual Groundwater Recharge Volumes Availability by Method 1 based on USGS Gage records at the Antelope Creek.

Method		Min, acre-ft	Max, acre-ft	Average (including years with no volume available), acre-ft	Percent of Years with Volume greater than 0, acre-ft
Method 1	90th Percentile/20 Percent*	369	13,416	4,877	95

Notes: ac-ft = acre-feet / foot; cfs = cubic feet per second; No. = number

*The gage daily flow records are from 1940 to 1982. 90th percentile table was obtained from USGS gage website USGS Surface Water data for USA: USGS Surface-Water Daily Statistics. Statistics were calculated based on water year (from December to March for each year).

Table 8. Annual Days Diversion Available Based on Method 1 at Antelope Creek

Water Year Type	Days Water Available for Diversion			No. Years
	Min	Max	Average*	
Wet	9	27	20	16
Above Normal	4	20	13	6
Below Normal	3	16	6	10
Dry	1	8	4	8
Critical	0	0	0	2
All Years	1	27	12	42

*For each water year type, the Average values calculations include all years with zero diversion. However, the minimum estimates did not include years with zero diversion.

6. Conclusion and Next Steps

This evaluation utilized the latest high-resolution terrain and gage data to demonstrate that higher frequency events could result in road closures property impacts along the Lower Antelope Creek. An evaluation of these events showed that diverting flow from the four proposed diversion points could reduce water levels from the November 2024 event, an approximate 5-year frequency event, to a water level below the flood diversion threshold.

Flood conditions happen infrequently. The unavailability of 15-minute records at this gage prevented calculation of average annual recharge volume for Method 2, the threat of flood conditions The Method 1 90th percentile/20 percent average annual volume and years available likely make pursuing long term flood diversion water right permits a viable option in addition to diverting as much flow as possible under Method 2.

The following information is recommended for inclusion in the flood diversion guidelines:

- Properties south of Cone Grove Road, between HWY 99 and Craig Avenue, may be under imminent threat of flooding due to flows exceeding the flood diversion threshold of 5,125 cfs (refer to Figure 10).
- Diversions under State Water Board Code 1242.1 guidelines (2025) at any location along the entire geographic extent of the stream of Antelope Creek would be expected to reduce an imminent threat.

- The flow threshold (flood diversion threshold) of 5,125 cfs at USGS gage is associated with the imminent threat.

Next steps for Antelope Creek include:

1. Include high flow observations (exceeding 440 cfs) at CDEC gage.
2. Add flow forecast function to the CDEC gage and add it into CNRFC river forecast network.
3. Complete land suitable and available for groundwater recharge analysis for area near the four proposed diversion points and along both sides of the creek between these points.
4. Compile the diversions and capacity of the Antelope Creek upstream of flooding area.
5. Initial screening of possible additional diversions off the Antelope Creek upstream of flooding area
6. The hydrologic (HEC-HMS) and hydraulic (HEC-RAS) models are currently in draft form and undergoing quality control review.
7. Review and develop alternatives to rank potential solutions for flood risk reduction and groundwater recharge locations.

7. References

State Water Resources Control Board (State Water Board). https://www.waterboards.ca.gov/waterrights/water_issues/programs/applications/groundwater_recharge/docs/streamlined_waa_guidance.pdf. Sacramento CA.

State Water Resources Control Board (State Water Board). 2025. (Updated to Reflect Executive Order N-16-25). January 2025

Fish passage in lower Antelope Creek. Prepared by Stillwater Sciences, Arcata, California and Tehama County Resource Conservation District, Red Bluff, California for U.S. Fish and Wildlife Service National Fish Passage Program, Red Bluff, CA.

ATTACHMENT D

GEI Consultants

Technical Memorandum

11010 White Rock Road, Suite 200 ● Rancho Cordova, CA 95670 ● 916.631.4500

Via Email:	pdhaliwal@lsce.com, wanderson@lsce.com
To:	Will Anderson and Pavan Dhaliwal, Luhdorff & Scalmanini Consulting Engineers Tehama County Flood Control and Water Conservation District, California
From:	Bryan Thoreson; Yi Shen (GEI)
cc:	Chris Ferrari (GEI)
Date:	November 19, 2025
Re:	Hydrologic and Hydraulic Model Approach and Evaluation for the Cottonwood Creek Groundwater Recharge and Flood Reduction Project
Project No.	2403778

1. Introduction and Purpose

As flood events become more frequent and existing residential developments are identified in the floodplains area, preparing for flood diversions and implementing groundwater recharge strategies is essential for sustainable water management. The California Water Code §1242.11 allows parties to divert flood flows for groundwater recharge without a water right provided they comply with specified requirements.

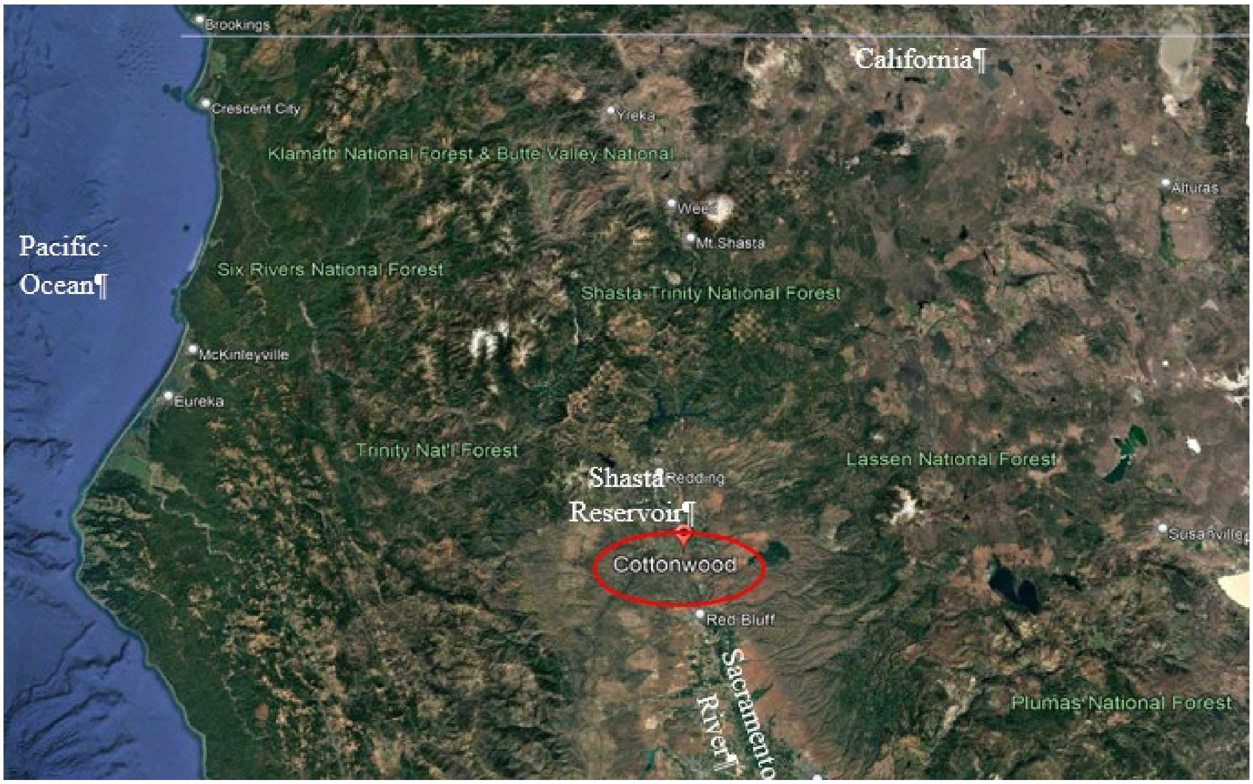
Cottonwood Creek is a major stream and tributary of the Sacramento River near Cottonwood, in Northern California (Figure 1). The purpose of this Technical Memorandum (TM) is to investigate the feasibility of diverting surface water from Cottonwood Creek for groundwater recharge, with the goal of mitigating flood impacts. This TM includes the following items:

- Development of Hydrologic and Hydraulic Models to evaluate existing conditions and proposed alternatives based on statistical 2-, 5-, 10-, 100- and 500-year storm events.
- Calculated rating curves and flood inundation maps for all the events.
- Proposed river diversion locations and determined diversion threshold for Flood Reduction and Groundwater Recharge Analysis.
- Estimated Diversion Volumes Required to Reduce Flows to the Flood Threshold following California Water Code §1242.1 Guidelines.
- Estimated Water Available for groundwater recharge using California State Water Board Guidelines. The existing conditions model information will be used to evaluate various project alternatives for diverting flow to support groundwater recharge and estimate potential flood reduction results.

[1] https://www.waterboards.ca.gov/waterrights/water_issues/programs/groundwater-recharge/recharge-diversions.html

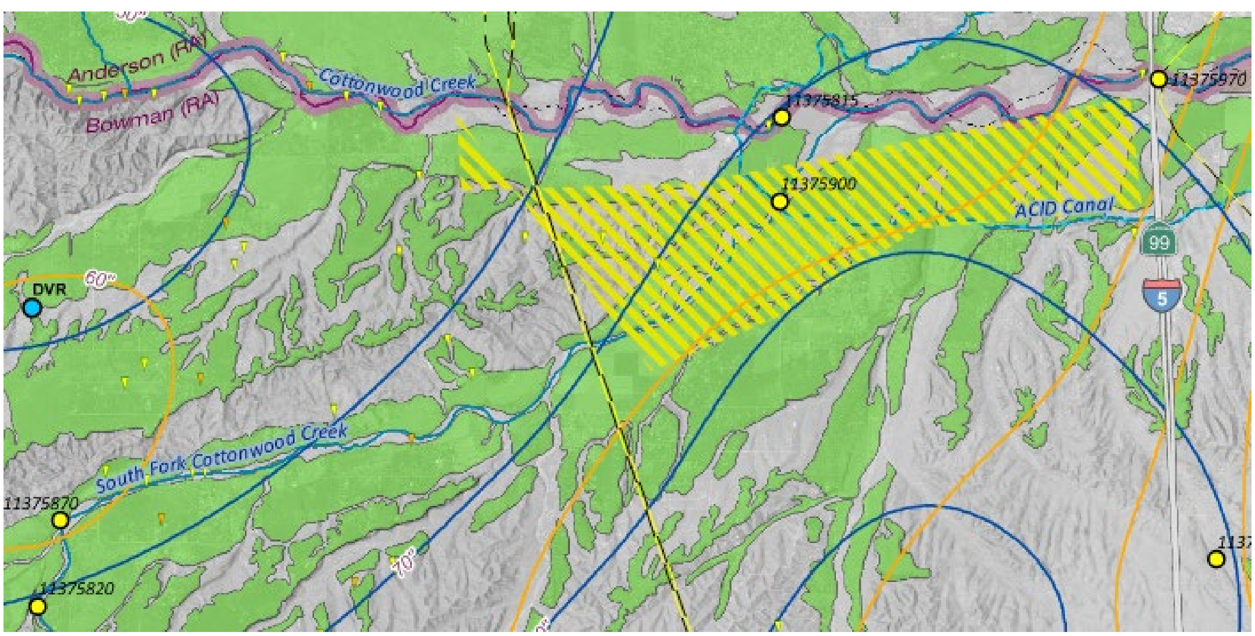
Figure 2 shows a High-Risk Flood-Prone Zone (hatched in yellow) identifies the county's experience near Cottonwood Creek for road closures, property damage, and restricted access to a school during the winter of 2023.

Figure 1. Project Location



Source: Google Maps

Figure 2. Cottonwood Creek High-Risk Flood-Prone Zone



Source: County High Risk Flood Prone Areas

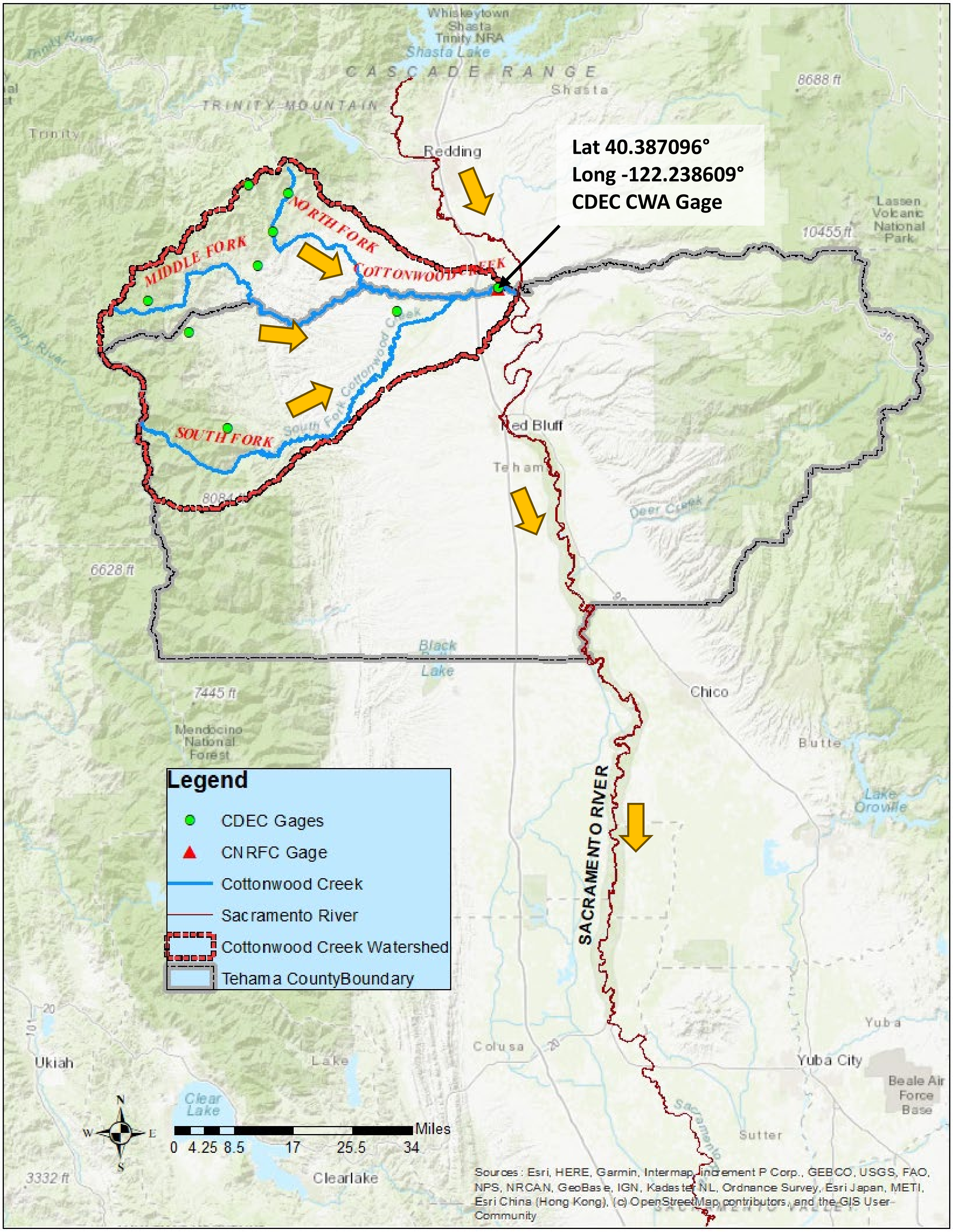
2. Background

The Cottonwood Creek Watershed, located in Shasta and Tehama counties, on the northwest side of the Sacramento Valley, encompasses approximately 938 square miles (Figure 3). Elevations within the watershed range from 7,800 feet in the upper reaches to 150 feet at its confluence with the Sacramento River. The major river draining Cottonwood Creek Watershed is Cottonwood Creek. Its three main tributaries, North Fork, Middle Fork, and South Fork, converge west of I-5 before discharging into the Sacramento River, east of I-5. Cottonwood Creek is a major source of sediment and gravel input to the Sacramento River.

Cottonwood Creek has a natural hydrologic pattern characterized by high flows and peak runoff events during the winter months, and low flows throughout the summer and fall. The average annual flow in the lower reach on Cottonwood Creek, near the Sacramento River, is approximately 860 cubic feet per second (cfs). Summer flows typically average 50 to 100 cfs. The largest 1-day flood flow recorded near the Sacramento River for Cottonwood Creek occurred in January 1974, reaching an estimated peak of approximately 70,000 cfs.

Studies were done in the past to evaluate the feasibility of constructing dams on Cottonwood Creek Watershed for flood control and streamflow characteristics for effect of groundwater contribution (USGS, 1992). This study presents an alternative which will recharge the groundwater basin by diverting extra flood water from Cottonwood Creek.

Figure 3. Cottonwood Creek Watershed and Tehama County Boundary (yellow arrows represent flow directions)



Source: GEI

3. Step by Step Guidance

In support of this investigation, this TM leverages the following models and datasets:

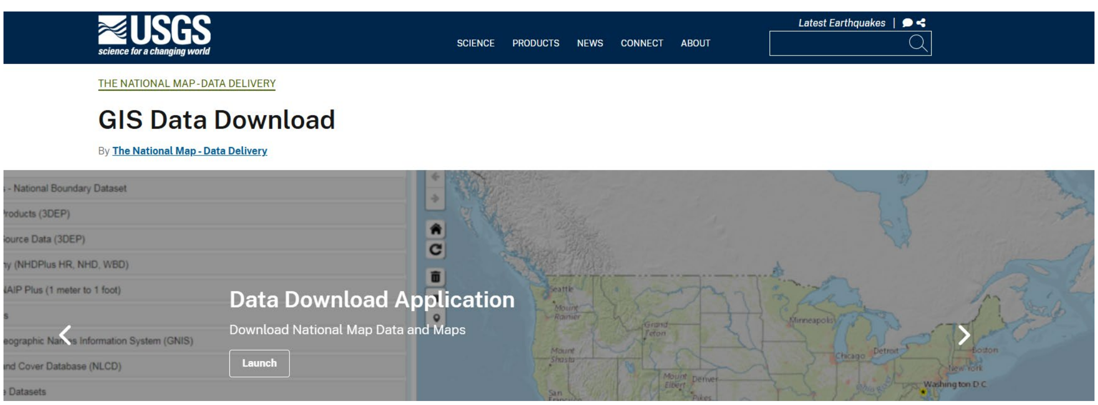
- Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) and Hydraulic Engineering Center River Analysis System (HEC-RAS) model development for Cottonwood Creek
- Utilizing United States Geological (USGS) Survey Terrain
- Validating/calibrating the hydrologic and hydraulic models using USGS gage data
- Hydraulic model floodplain results based on statistical 2-, 5-, 10-, 100- and 500-year storm events.
- Flow and stage forecasting using California Nevada River Forecast Center (CNRFC) tools.
- Hydrologic Engineering Center Statistical System Program (HEC-SSP)

The following steps discuss the approach for setting up hydrologic and hydraulic models to evaluate existing conditions, evaluate various project alternatives, and how to use the CNRFC information to forecast flood flows for Cottonwood Creek.

Step 1 – Data Gathering

Download the best available Light Detection and Ranging (commonly known as LiDAR) terrain for the hydrologic and hydraulic model development. Figure 4 shows the USGS website's home page.[2]

Figure 4. USGS Terrain Website



The rainfall-runoff model computes flow hydrographs based on different precipitation events using the HEC-HMS program. The terrain used in the rainfall-runoff hydrologic model needs to cover the entire modeled watershed. For the Cottonwood Creek watershed, a 10-meter resolution digital elevation model (DEM) was downloaded and applied to the hydrologic model.

[2] https://www.usgs.gov/the-national-map-data-delivery/gis-data-download

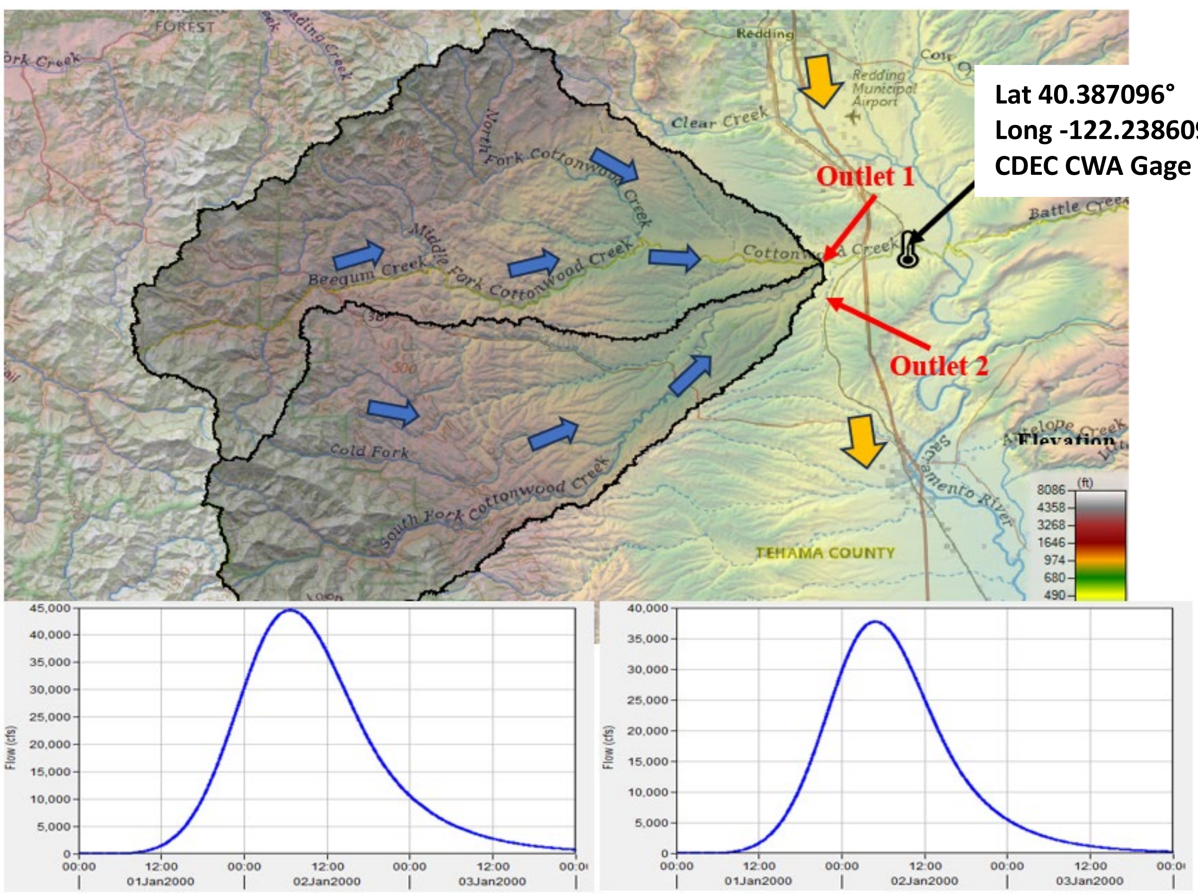
Hydrologic model setup includes obtaining rainfall information from the local agency or use National Oceanic and Atmospheric Administration (NOAA) Atlas 14 hypothetical design storms for return periods of 2-, 5, 10-, 25-, 50-, 100-, 200- and 500-year downloaded from NOAA.[3]

Step 2 – Model Setup

Watersheds can be created in HEC-HMS and be associated with the terrain and precipitation data. The Cottonwood Creek Watershed was divided into northern and southern subbasins to compute flow hydrographs at the watershed outlet. Figure 5 shows the watershed boundaries using the USGS terrain data for the upper Cottonwood Creek Watershed. The north subbasin includes the drainage contribution from the Middle Fork and North Fork tributaries of Cottonwood Creek. The south subbasin represents the flow contribution from South Fork tributary of Cottonwood Creek. Figure 5 shows the computed 100-year flow hydrographs at Outlet 1 and Outlet 2 using the NOAA data and the HMS program.

The subbasin of the North Fork tributary of the Cottonwood Creek covers approximately 477.4 square miles, while the South Fork subbasin covers 396.9 square miles.

Figure 5. HEC-HMS Hydrology Basins with 100-Year Flow Hydrograph Outputs



HMS 100-Year Flow Hydrograph at Outlet 1 HMS 100-Year Flow Hydrograph at Outlet 2

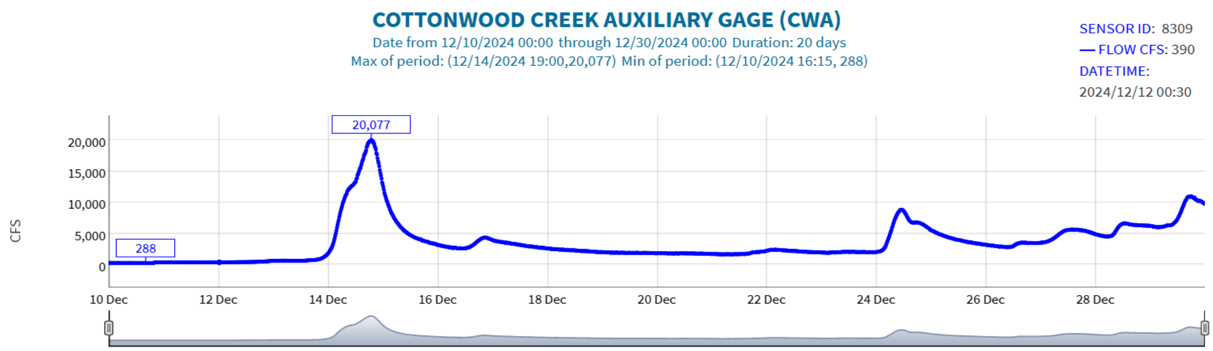
Source: HEC-HMS

[3] https://hdsc.nws.noaa.gov/pfds/pfds_map_cont.html

Step 3 – Calibration and Validation

Calibrating the hydrologic model provides a higher level of confidence for the hydraulic model evaluations. The HEC-HMS hydrologic model was calibrated and validated for selected events using the CDEC Cottonwood Auxiliary (CWA) gage (also known as CNRFC CWAC1, or USGS gage 11376000;). Figure 6 shows the December 2024 flow hydrograph from the CDEC CWA gage that was used for model calibration. The December 2024 storm is the latest major flood event impacting the Cottonwood Creek Watershed. The calculated flow hydrographs of the calibrated hydrologic model were then validated against the observed results from other three historical events including 2005, 2023 and 2017 storms.

Figure 6. December 2024



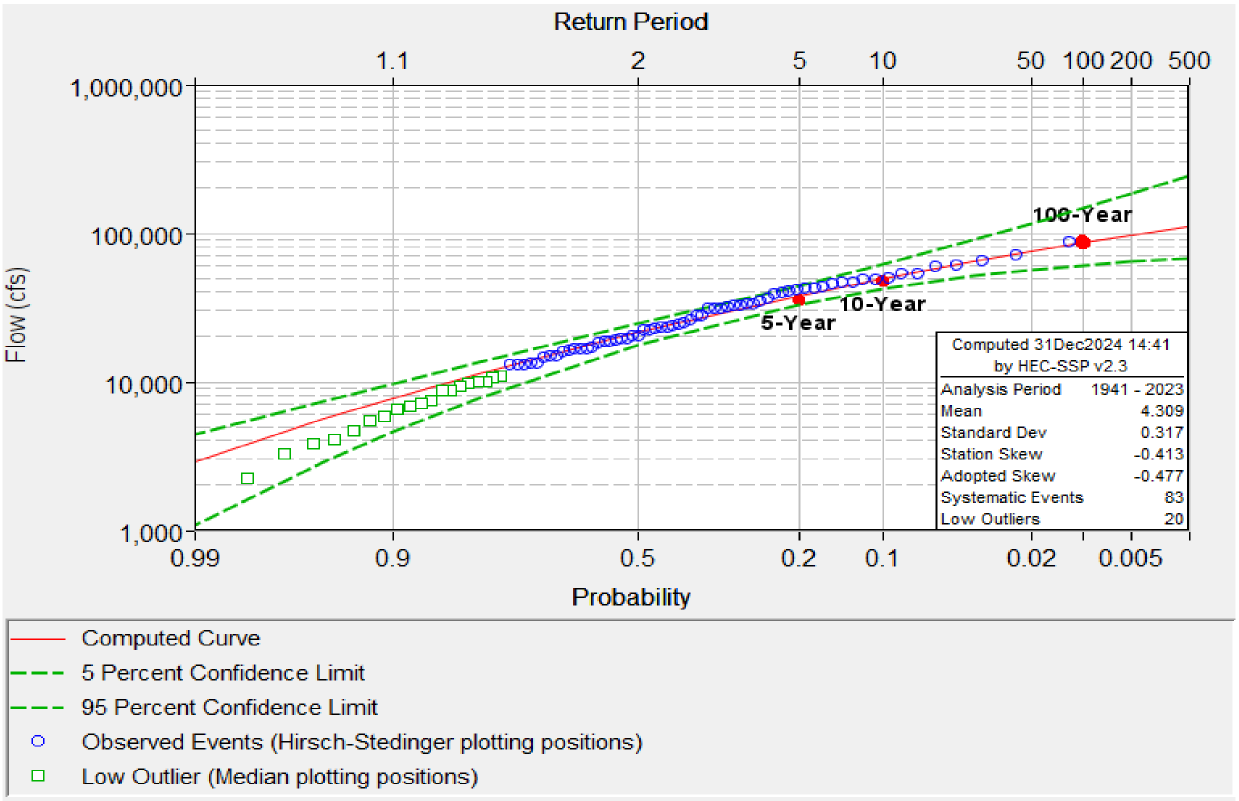
Source: CDEC Website

Step 4 – Flood Frequency Analysis

Figure 7 shows the flood frequency analysis results from HEC-SSP. The computed curve was based on using the available 20 years of annual peak flows from the CDEC CWA gage. The red dots represent the hydrologic model peak flows for the 5-, 10- and 100-year return periods computed from the HMS program from step 3 for comparison. The percent error between the gage and the hydrologic model outputs, using NOAA data for the three peak flow events, was less than 10 percent.

Table 1 provides the peak flow estimates at the CWA gage from the HEC-SSP analysis.

Figure 7. Flood Frequency Results at Gage CWA



Source: HEC-SSP

Table 1. HEC-SSP Statistical Peak Flow Estimates on the Main Channel of Cottonwood Creek

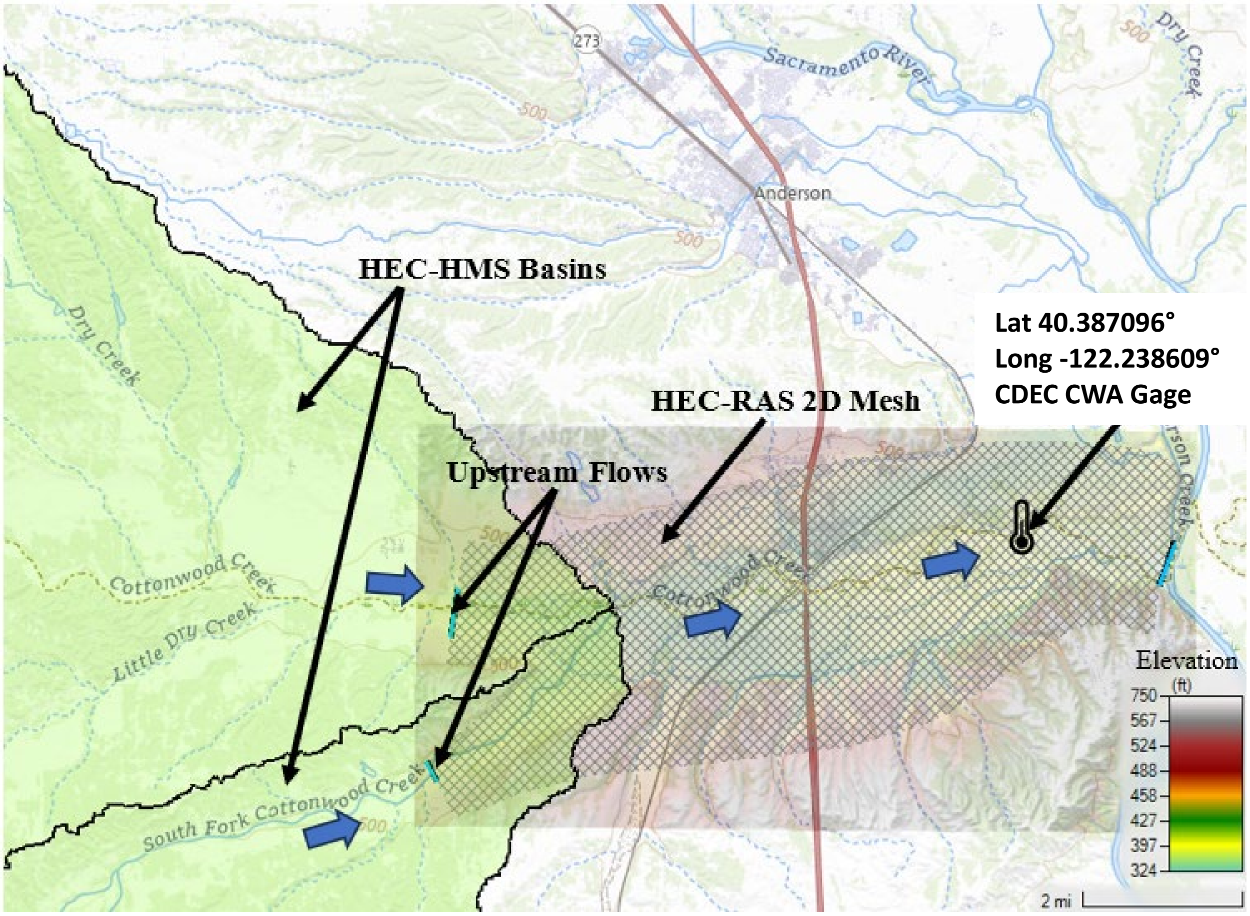
2-Year (cfs)	5-Year (cfs)	10-Year (cfs)	50-Year (cfs)	100-Year (cfs)	500-Year (cfs)
21,690	38,116	49,658	75,328	86,056	110,309

Note: cfs = cubic feet per second

Step 5 – Hydraulic Model Setup

The hydraulic model was developed using HEC-RAS software. The hydraulic model calculates water depths and flows at the study site based on upstream flow contributions from the two subbasins developed from the HEC-HMS model (refer to Steps 2 and 3). A two-dimensional (2D) mesh was created in HEC-RAS along the main channel of Cottonwood Creek which starts near the downstream ends of the three tributaries of Cottonwood Creek including South Fork Cottonwood Creek, Middle Fork Cottonwood Creek and North Fork Cottonwood Creek, and ends at the junction of the Sacramento River (Figure 8). Each grid cell within the 2D mesh calculates water depth and flows for a particular event.

Figure 8. HEC-RAS Model 2D Mesh and Upstream Boundary Inflows

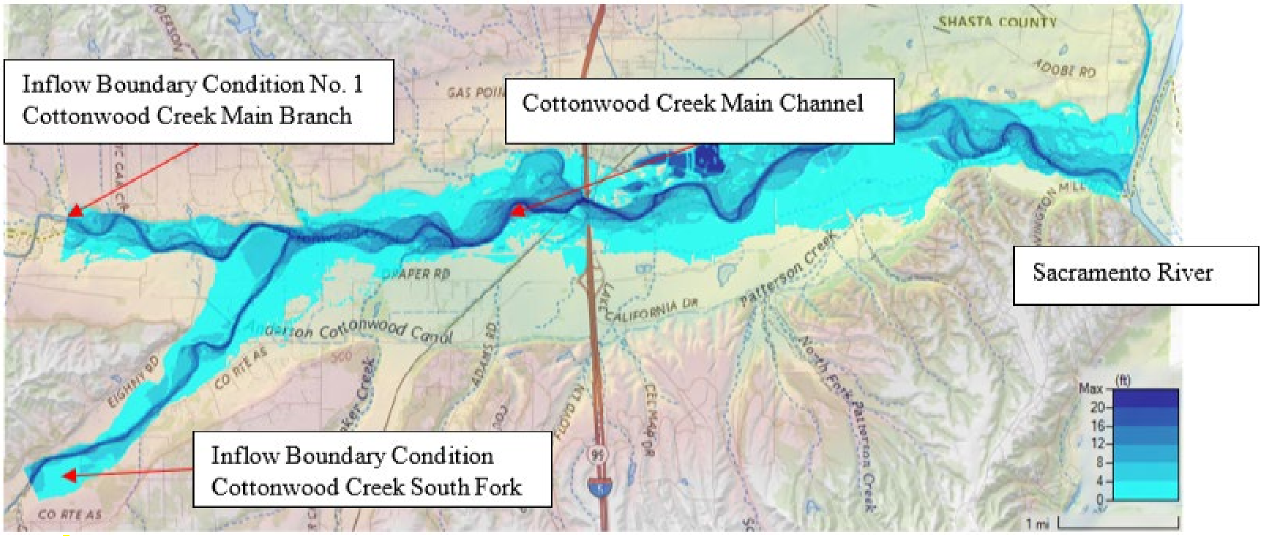


Source: GEI

Step 6- Hydraulic Model Results

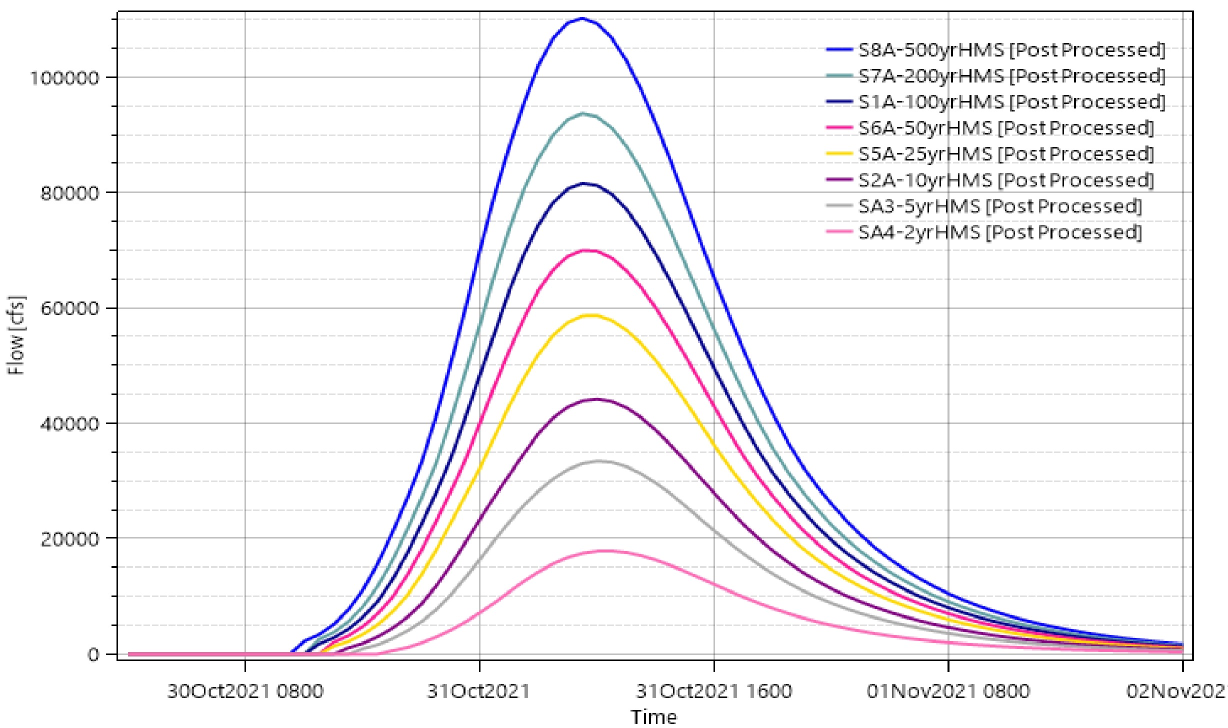
The HEC-RAS program routes the flow hydrographs using unsteady 2D Shallow Water Equations flow analysis method to provide water depth, flood extents and velocities for the different return periods. Figure 9 shows the estimated depths of flow for Cottonwood Creek during a 100-year storm. Figures 10 and 11 provide computed flow hydrographs and water surface elevations respectively at the I-5 bridge for all the eight storm frequencies. Figure 12 provides the flow versus stage rating curve at I-5 from the HEC-RAS 2D model based on the results of the 500-year storm.

Figure 9. 100-year Flow Depths and Extents



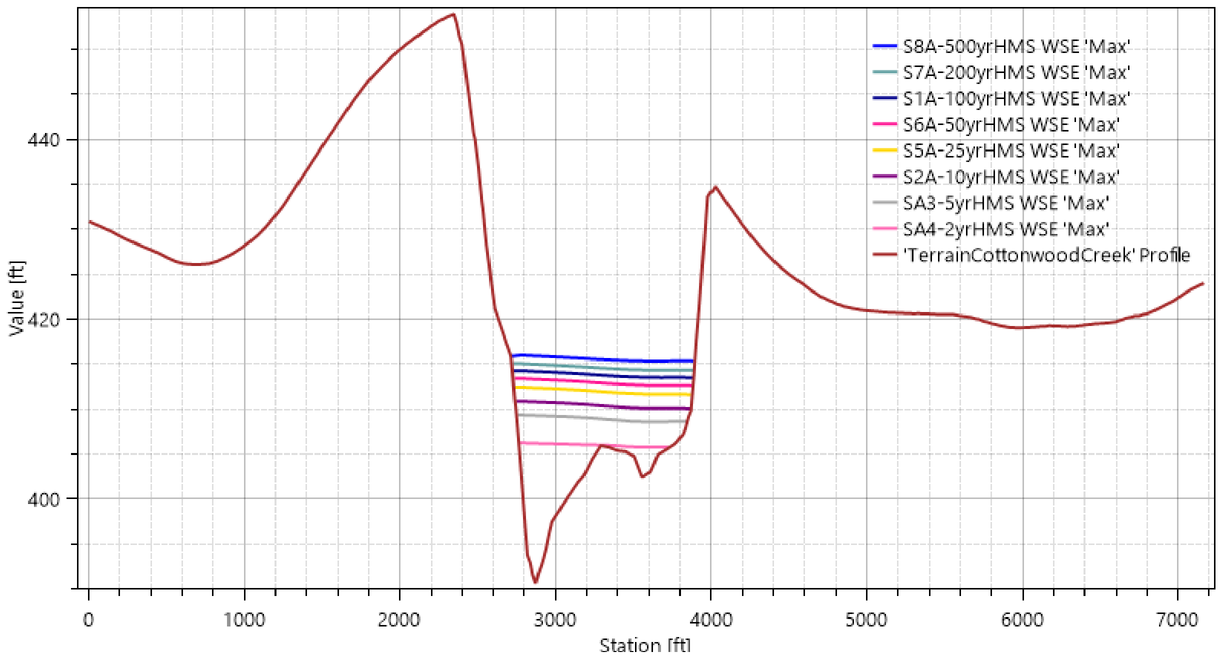
Source: HEC-RAS

Figure 10. Cottonwood Creek Flow Hydrographs



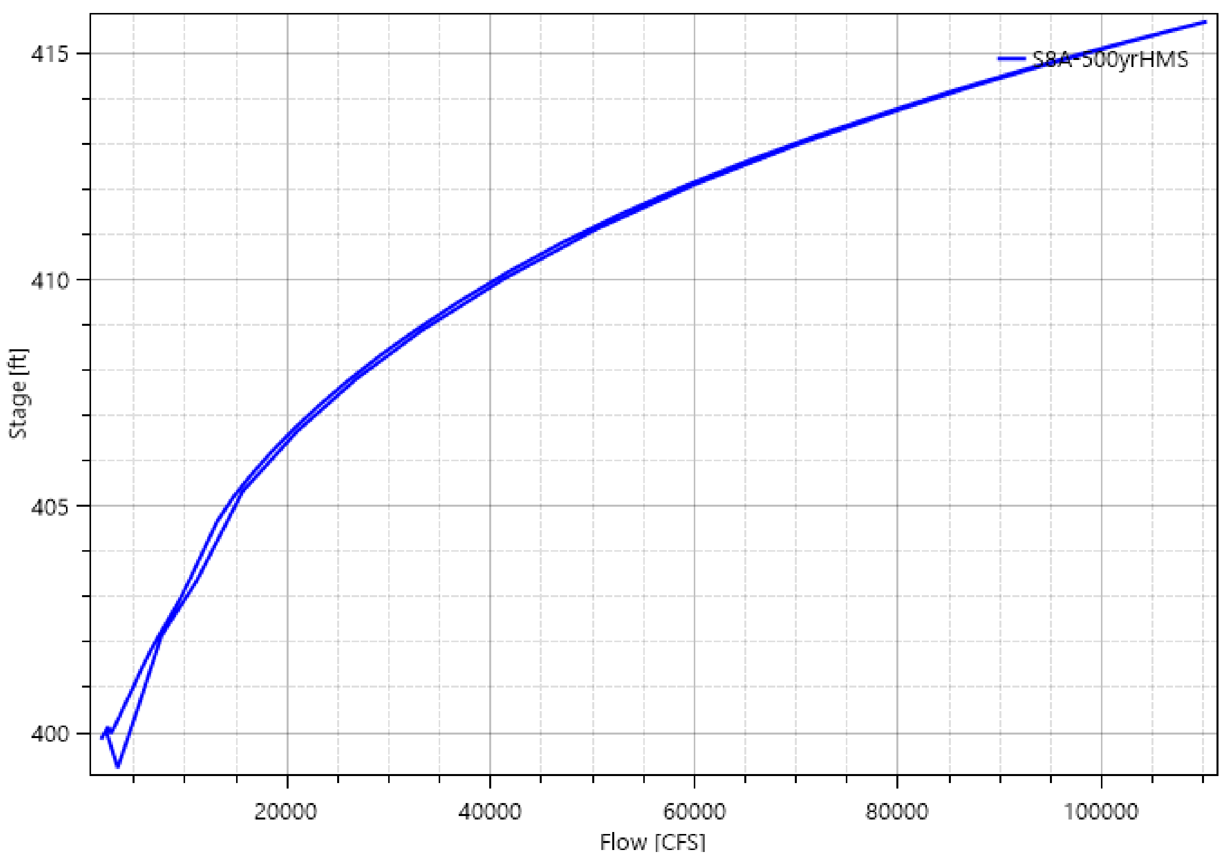
Source: HEC-HMS

Figure 11: Water Surface Elevations at I-5 Bridge



Source: HEC-RAS @ I-5

Figure 12: Flow vs. Stage Rating Curve at I-5



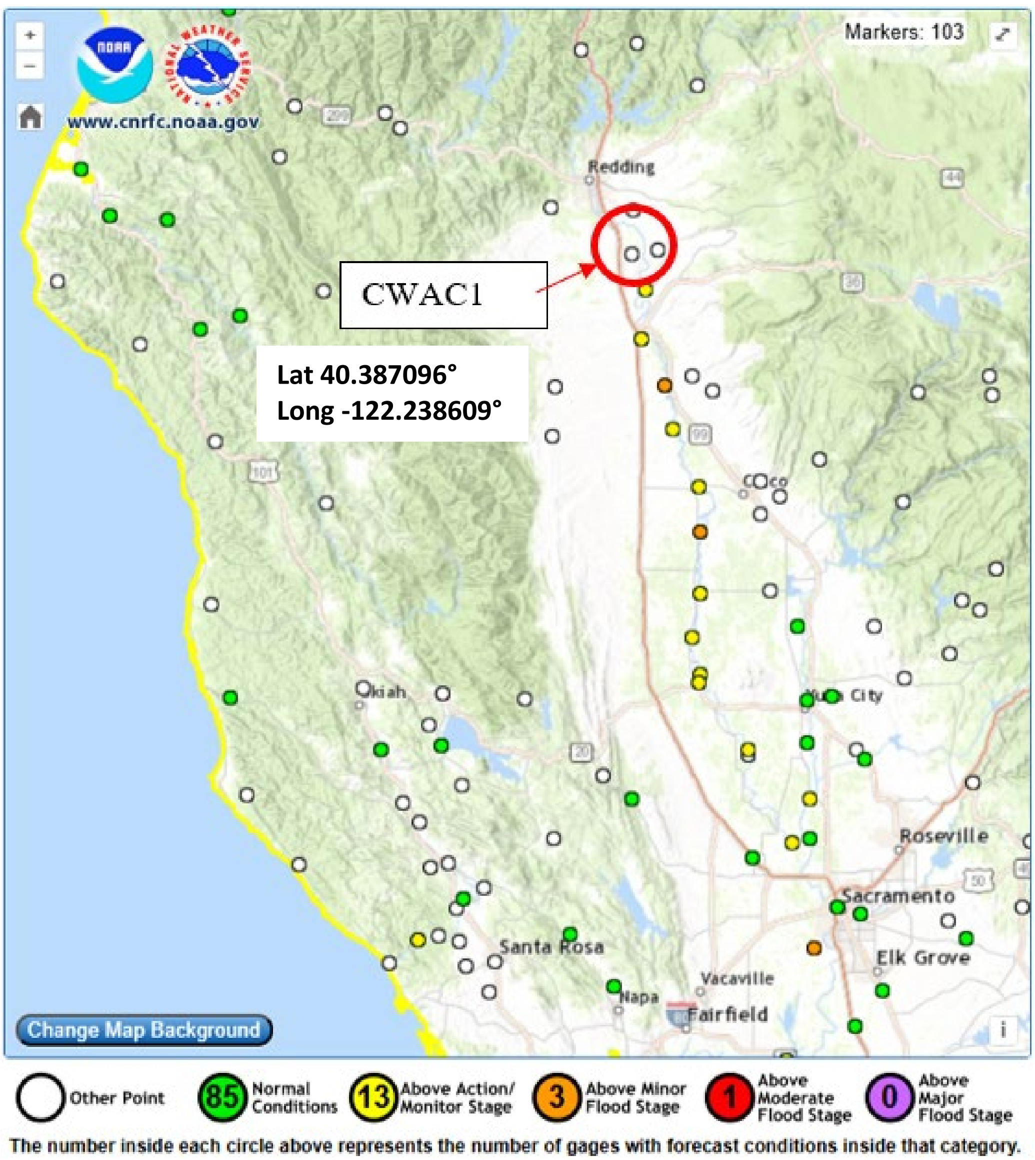
Source: HEC-RAS at I-5

Step 7 – Flood Forecasting

Figures 13 and 14 show the CNRFC gage CWAC1 which provides 5-Day forecast and hindcast hydrology in the main channel for Cottonwood Creek. As mentioned at Section 3, the CNRFC CWAC1 gage is the same as CDEC CWA gage and USGS 11376000 gage with different names. The CNRFC CWAC1 gage is used for forecasting and does not retain historical data. The various colors shown on the nodes predict the flood condition at those locations. The CNRFC flow hydrographs are updated every 6 hours and can be used for comparison in the hydraulic model to estimate flood levels, depths, extents and velocities for potential impacts for emergency managers.

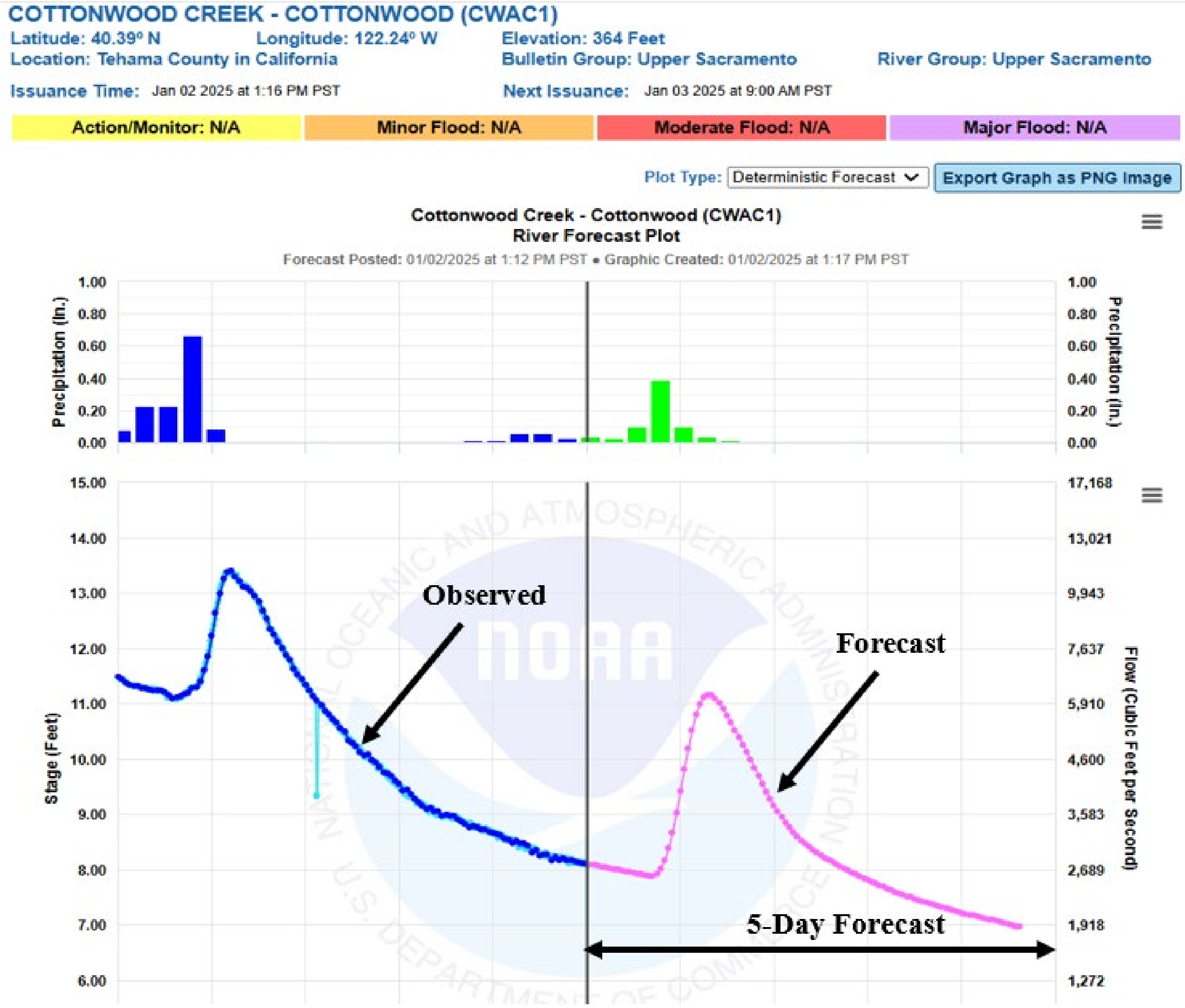
Based on results from the HEC-RAS, Figure 15 presents a flow-versus stage rating curve that can be extracted at any location within the study area. These rating curves, when combined with flood forecast data, can be used to predict water surface elevations and inundation extents at selected locations.

Figure 13: CNRFC Gage CWAC1 (i.e. CDEC CWA)



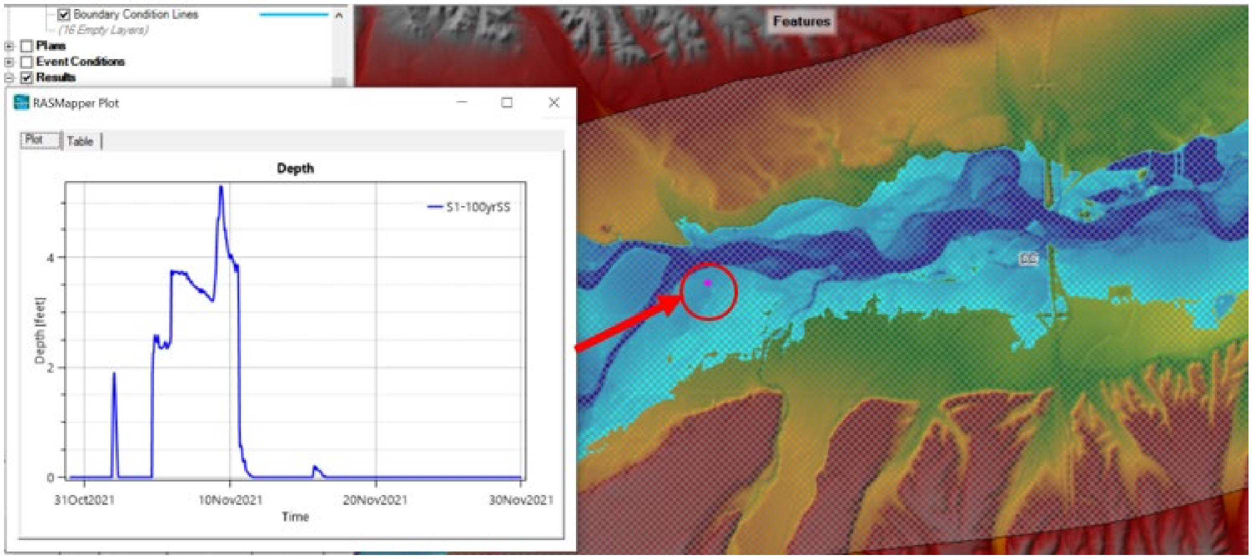
Source: CNRFC

Figure 14. CNRFC Gage CWAC1



Source: CNRFC Website

Figure 15. Depth of Flow (November 2021)



Source: HEC-RAS

4. Model Results and Findings:

This section discusses the hydrologic and hydraulic model evaluation:

- The hydrologic model flow hydrograph was calibrated to the CDEC CWA (i.e., USGS Gage 11376000 – Cottonwood Creek or CNRFC CWAC1) near the unincorporated community of Cottonwood, California for a 2024 event. The calibrated HEC-HMS model was also used to verify the flow hydrographs at the same USGS gage for three additional events (2005, 2017, and 2023 events).
- The Hydrologic Model (HEC-HMS) was used for five statistical design storms for the 5-, 10-, 50-, 100-, and 500-year storm frequency. All the model parameters were calculated using formulas recommended by HEC-HMS user manual.
- The hydraulic model using HEC-RAS was used to calculate stage hydrographs and flood inundation maps for all the events.

Table 2 shows peak flows for the past 10 years (since 2014), and their estimated storm frequency based on the statistical analysis from Step 5 in Section III. The 2024, 2022, 2021, 2020, 2018, 2017, 2016 and 2014 high water events all appear to be less than a 5-year (20% chance storm) frequency, whereas the 2005 peak flow of 46,700 cfs, (not in the table) was estimated as an approximate 9-year (11% chance storm) frequency event. Therefore, no recorded peak flows have exceeded the 10-year frequency within the past decade.

However, as previously noted, flooding was reported in 2023 during a 6-year flood event, but not in 2024 during a 4-year flood event. Based on this, it can be reasonably inferred that flooding may begin to occur between 32,000 and 42,800 cfs, with a likely threshold around 38,000 cfs, corresponding to a 5-year event. Therefore, the flood diversion threshold is set at 38,000 cfs using CDEC CWA gage (see Figure 3 for its location).

Table 2. Historical Flood Events for the Past 10 Years (2014-2024)

Date	2024	2023	2022	2021	2020	2019	2018	2017	2016	2015	2014
Gage Recorded Peak Flow (cfs)	32,000	**42,800**	6,820	1,090	7,370	45,200	9,700	24,700	23,100	41,800	6,500
Estimated Return Period Storm Frequency	4	**6**	< 2	< 2	< 2	8	< 2	2	2	6	< 2

Note: cfs = cubic feet per second

4.1. HEC-HMS Model Verification for the Design Storms

Table 3 presents the model parameters used to evaluate statistical storms and the parameters used to calibrate the model for the historical storms. The hydrologic HEC-HMS model verification and calibration results are summarized in Table 4. The flow hydrograph comparisons between the hydrologic model and the gage records for the four historical flood events are shown in Figures 16, 17, 18, and 19. Overall, the hydrologic and hydraulic models performed well in forecasting peak flows and stages, with all the modeled errors being less than 10 percent when compared to gage records.

Table 3. HEC-HMS Model Parameters

Subbasin	Area (mi2)	T_{lag} (min)		Peak Rate Factor		Curve Number	
		Design Storms	Historical Events	Design Storms	Historical Events	Design Storms	Historical Events
North Fork	477.4	872	484	484	200	80.7	50
South Fork	396.9	755	465	484	200	81.9	50
Downstream	52.7	364	196	484	150	84	70
Total	**927.0**	N/A					

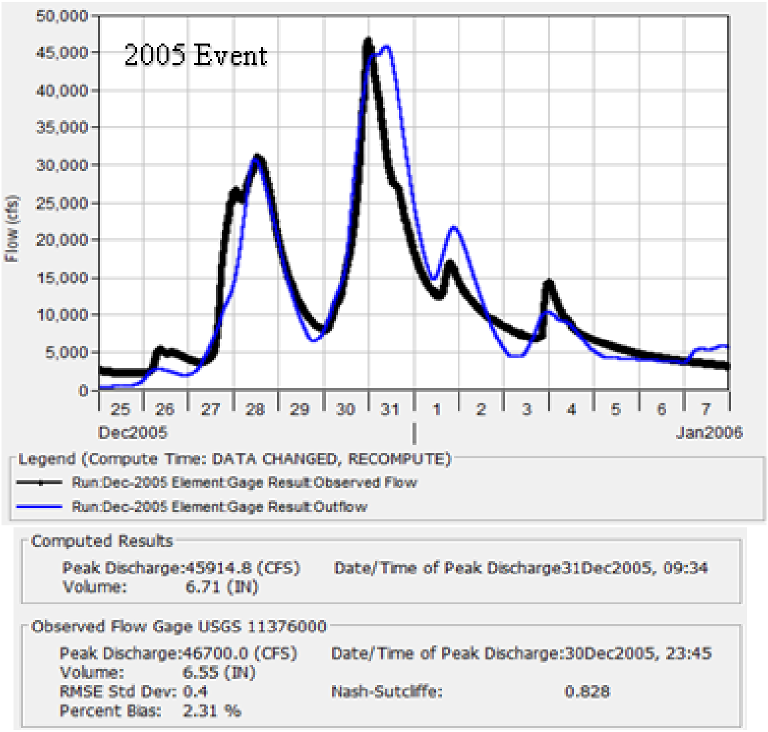
Notes: mi^2 = square mile; N/A = not applicable; T_{lag} (min) = minute

Table 4. HEC-HMS Model Calibration Summary

	Statistical Design Storm			Historical Storm			
	100-Year	10-Year	5-Year	2005	2017	2023	2024
Statistical/Observed Peak Flow (cfs)	86,056	49,658	38,116	46,700	24,700	42,800	20,100
Modeled Peak Flow (cfs)	90,220	52,333	41,319	45,914	26,021	42,875	20,368
Percentage Difference (%)	4.8	5.4	8.4	-1.7	5.3	0.2	1.3

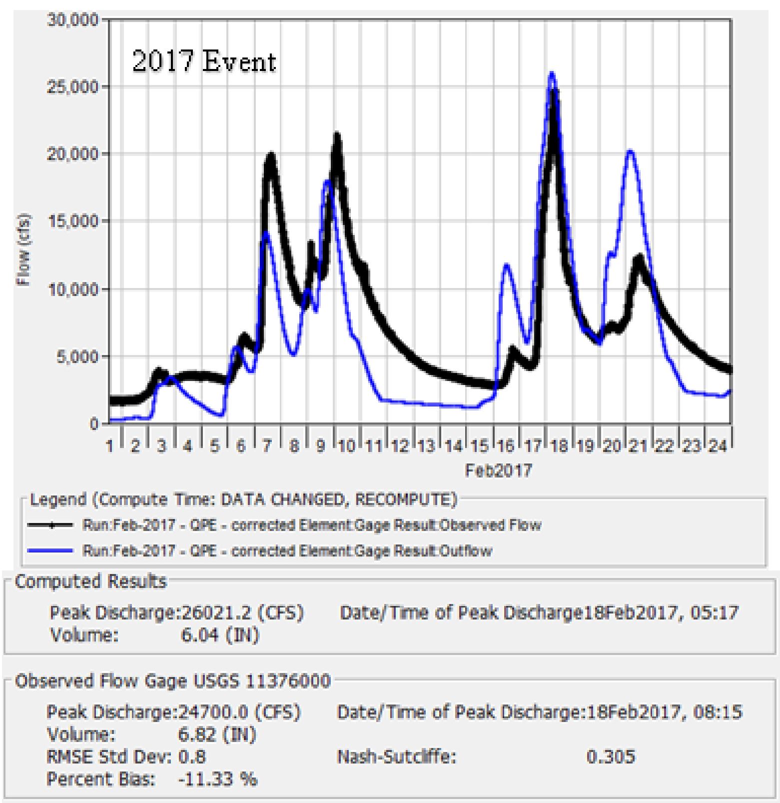
Note: cfs = cubic feet per second

Figure 16. CNRFC Gage CWAC1



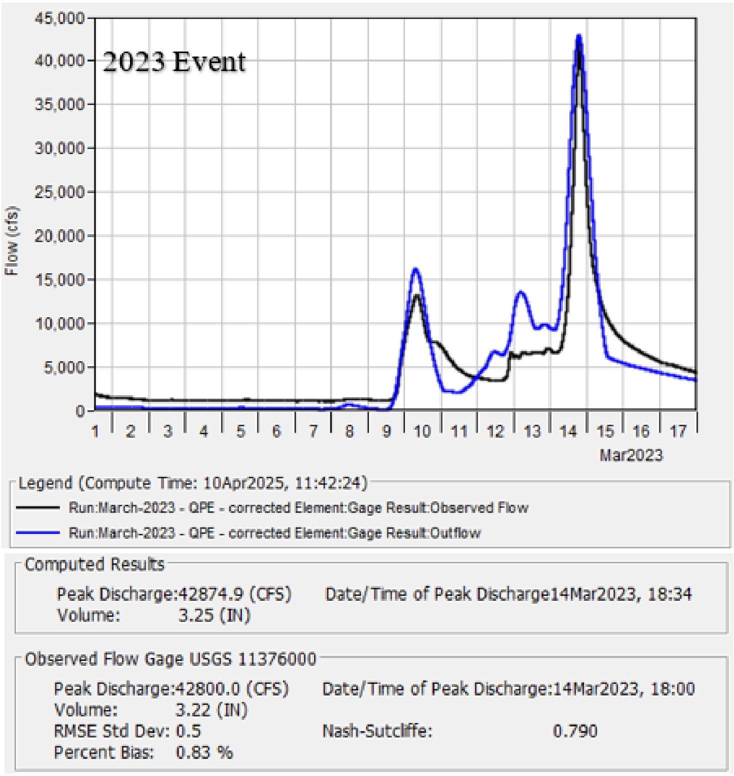
Source: HEC-HMS

Figure 17. Flow Hydrograph Comparison for the February 2017 Event



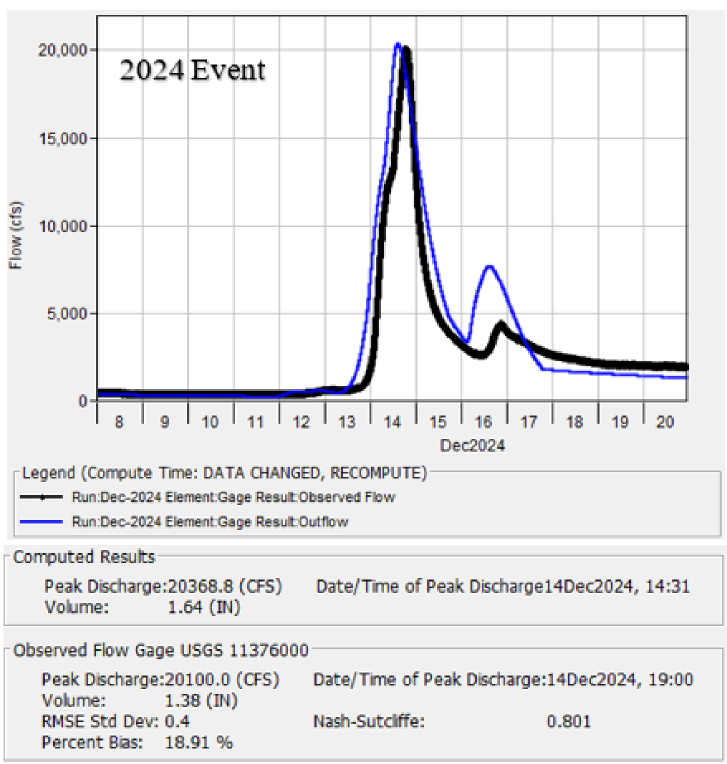
Source: HEC-HMS

Figure 18. Flow Hydrograph Comparison for the March 2023 Event



Source: HEC-HMS

Figure 19. Flow Hydrograph Comparison for the December 2024 Event



Source: HEC-HMS

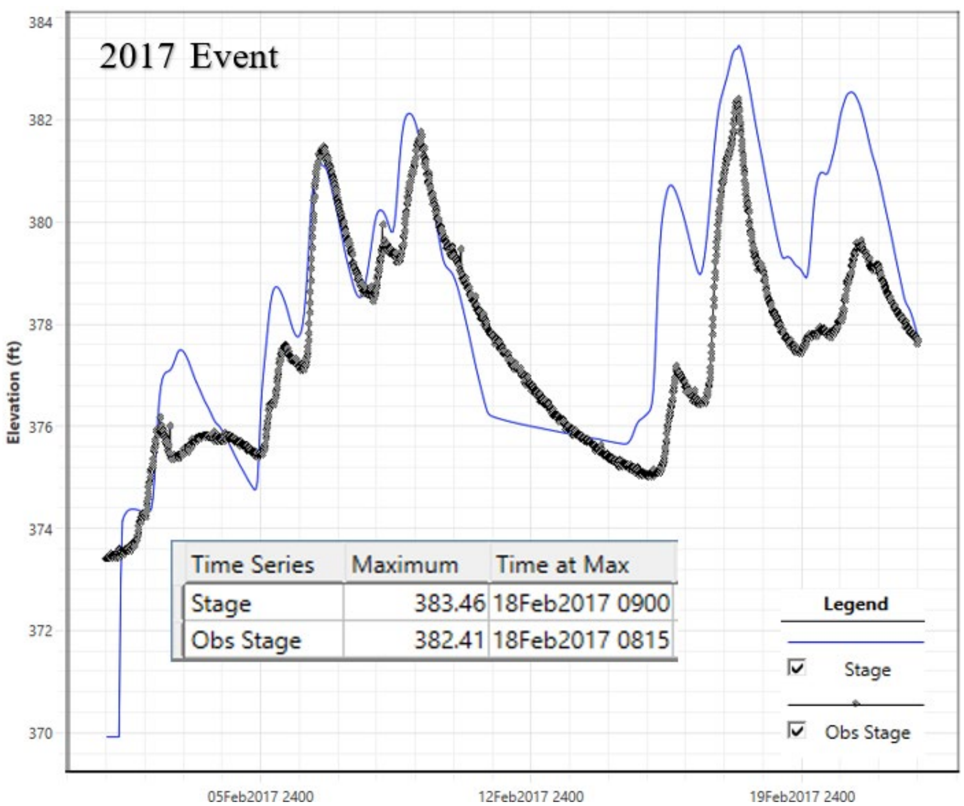
4.2. Hydraulic Model Verification Based on Historical Storms

The 2D hydraulic models utilized flow hydrographs from the hydrologic models previously discussed. Maximum depth inundation maps and stage hydrographs for the different storm events were based on the CDEC CWA gage.

Comparisons of the stage hydrographs at the gage are presented for 2017 (Figure 20), 2023 (Figure 21) and 2024 (Figure 22) events. The 2005 event was not included as the gage had not been installed at that time and thus no data was available for this event.

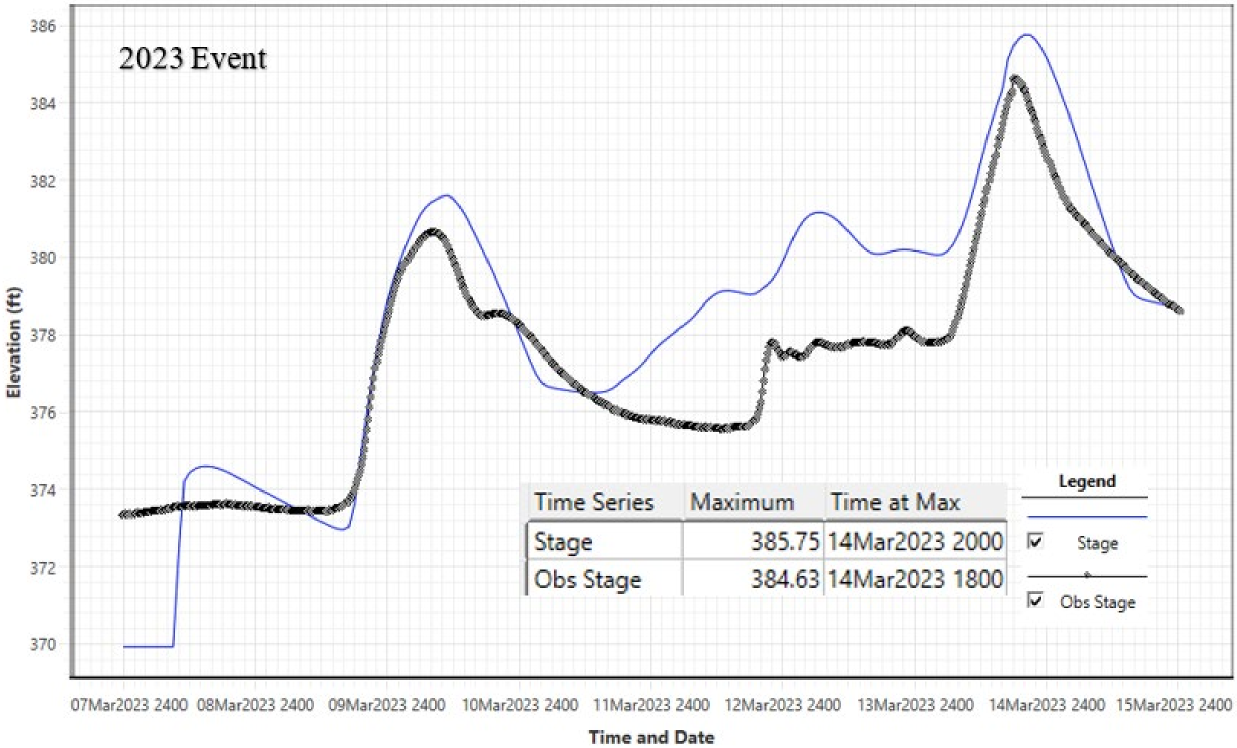
The gage calibration comparisons show reasonable results. The hydrograph shapes of the model generally follow the observed characteristics. The maximum stage differences between the model and the gage records are all less than 10%.

Figure 20. Stage Comparison for February 2017 Event



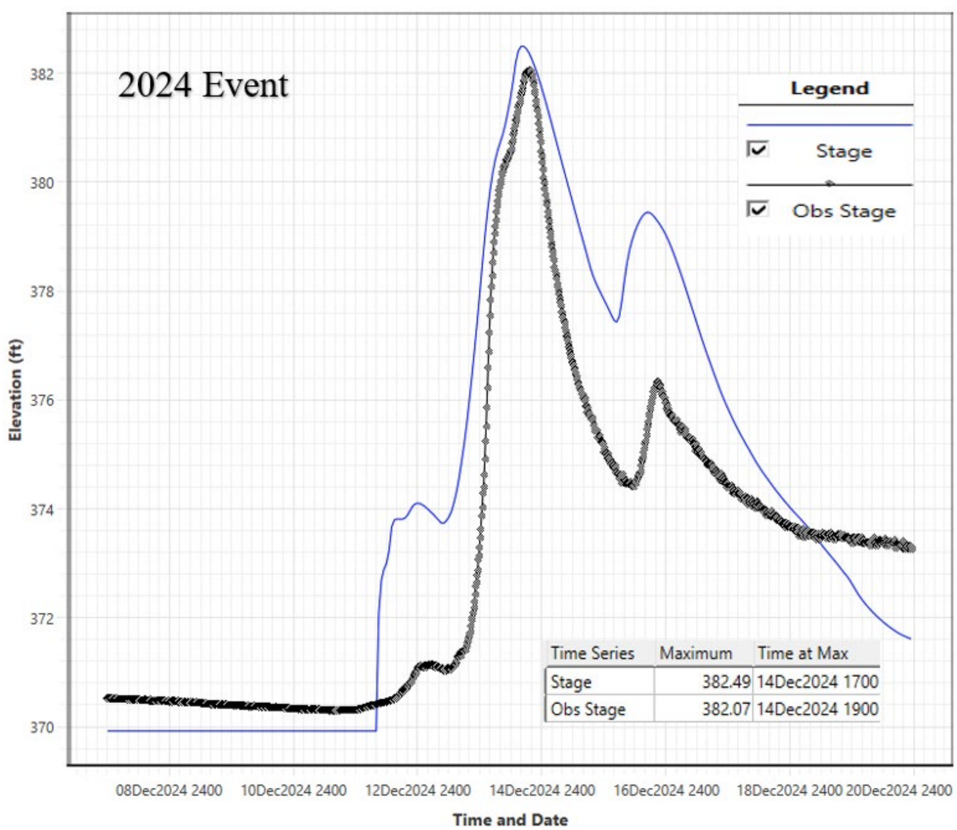
Source: HEC-HMS

Figure 21. Stage Comparison for February 2023 Event



Source: HEC-HMS

Figure 22. Stage Comparison for February 2024 Event

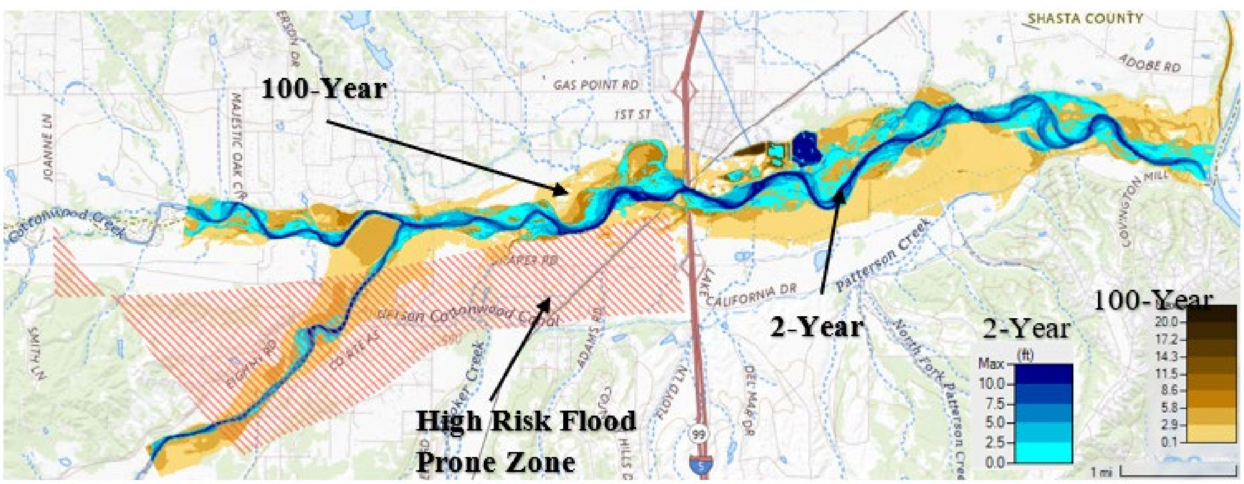


Source: HEC-HMS

Floodplain Comparisons

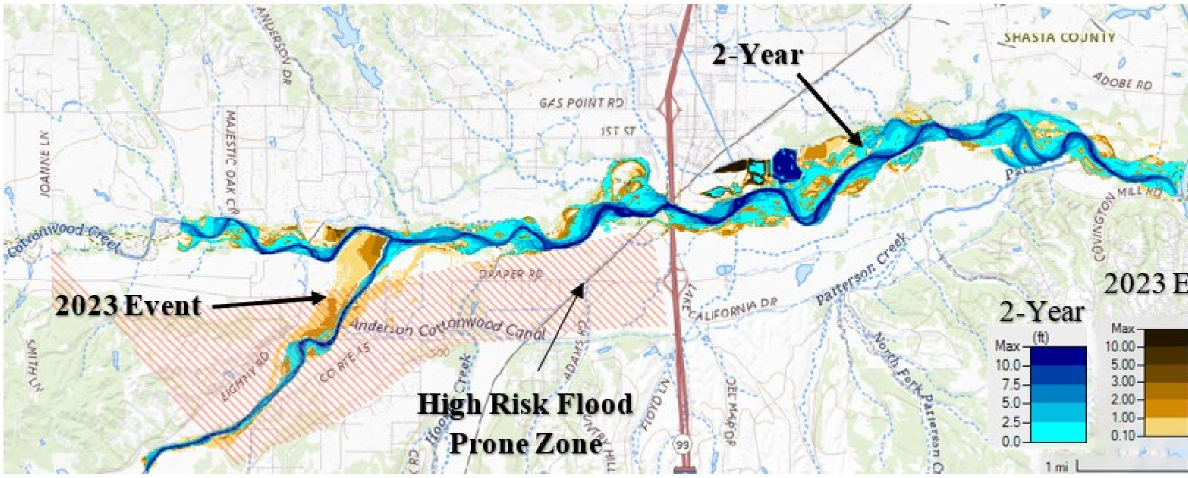
- Figure 23 shows the comparison between the 2-year (21,690 cfs) and 100-year (86,056 cfs) flood extents for the maximum water depth.
- Figure 24 shows the comparison between the 2-year and the 2023 historical storm (6-year event).
- Floodplain inundation is shown within the High-Risk Flood Zone for the South Fork of Cottonwood Creek (Figure 25). Potential road impacts were examined based on the 10-year frequency (49,658 cfs) which is comparable to both the 2005 and 2023 storm events. Figures 25-28 show the hydraulic model results for the 10-year event.
- Main Street, along the north bank of Cottonwood Creek, and the railway, on the south bank of Cottonwood Creek, would be inundated. The portion of the railway located west of I-5 would experience a maximum water depth of 6 feet. The section of Main Street east of I-5 would experience a maximum water depth of 7 feet (Figure 25).
- The north ramp of the Evergreen Road Bridge across the South Fork of Cottonwood Creek would experience a water depth of 2 feet (Figure 26).
- Other roads that could be affected include Longcor Road (maximum water depth = 0.1 foot) on the south bank and Traveled Way (maximum water depth = 1 foot) on the north bank of Cottonwood Creek (Figure 27 and Figure 28).

Figure 23. Max Depth Inundation for 2-year (blue) and 100-year (brown) Design Storms



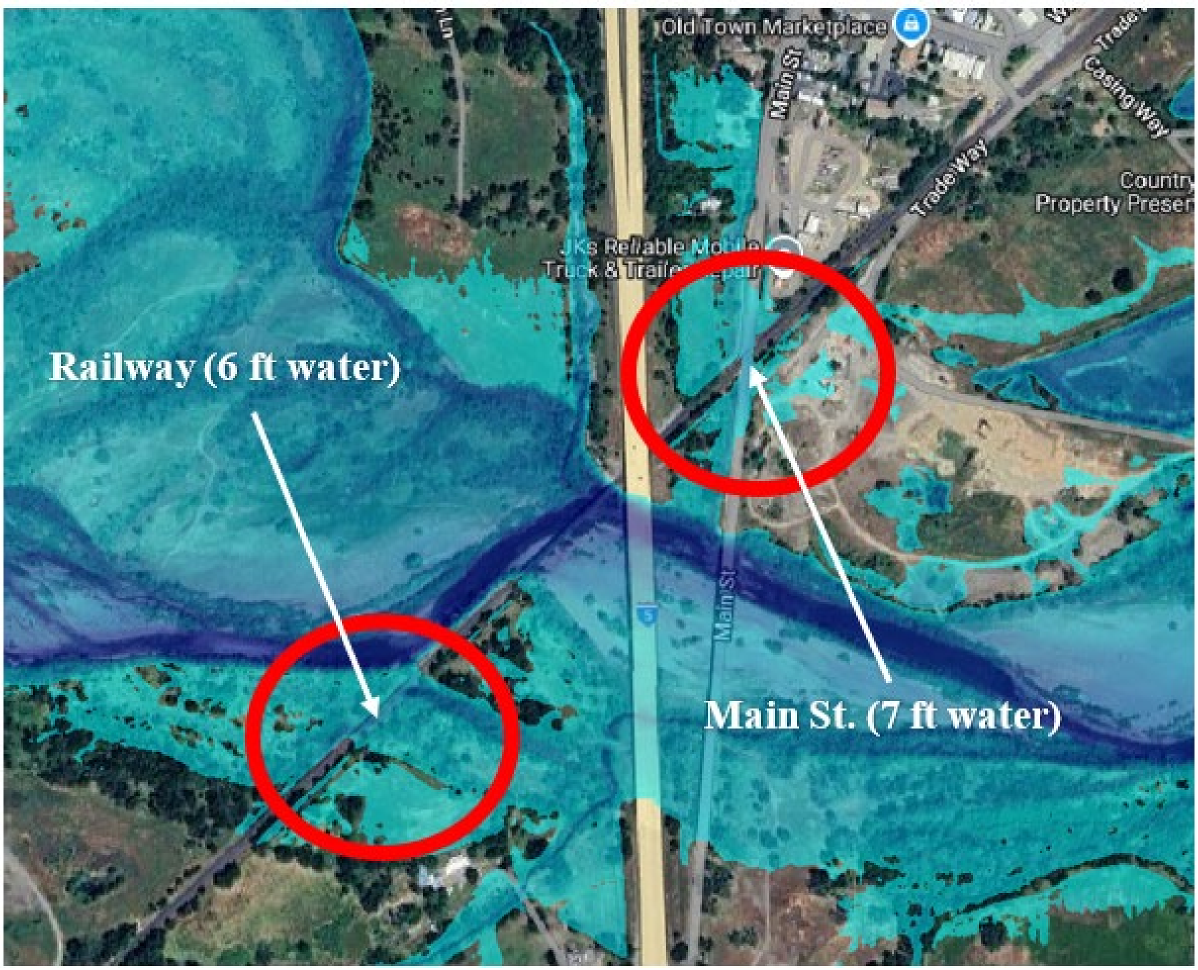
Source: HEC-HMS

Figure 24. Max Depth Inundation Comparison Between 2-year (blue) and 2023 Event (brown)



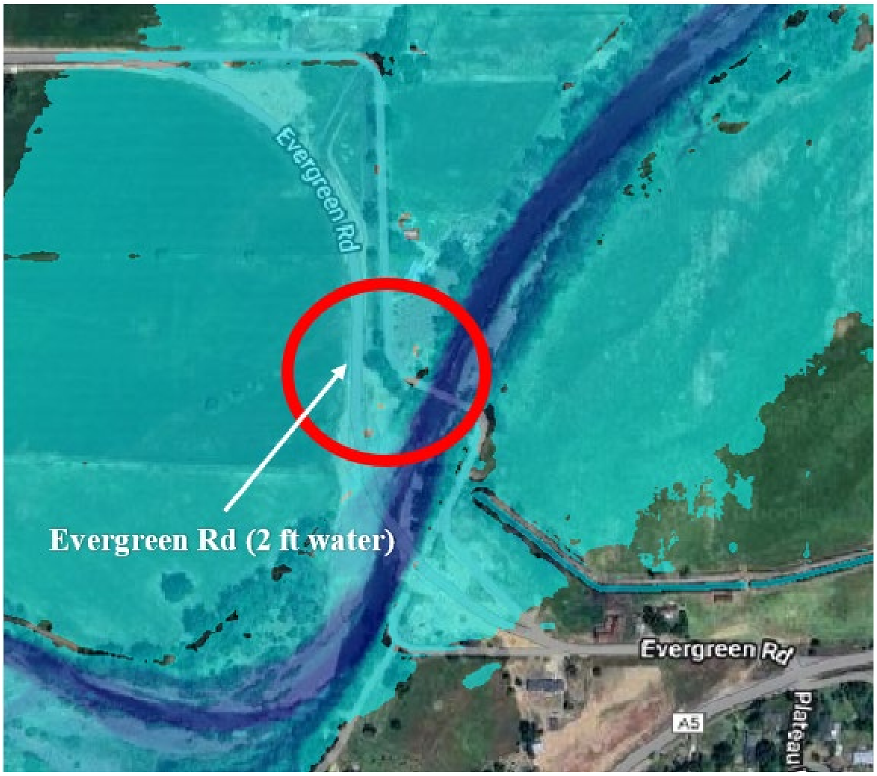
Source: HEC-RAS

Figure 25. Main Street Impacted for a 10-Year Event



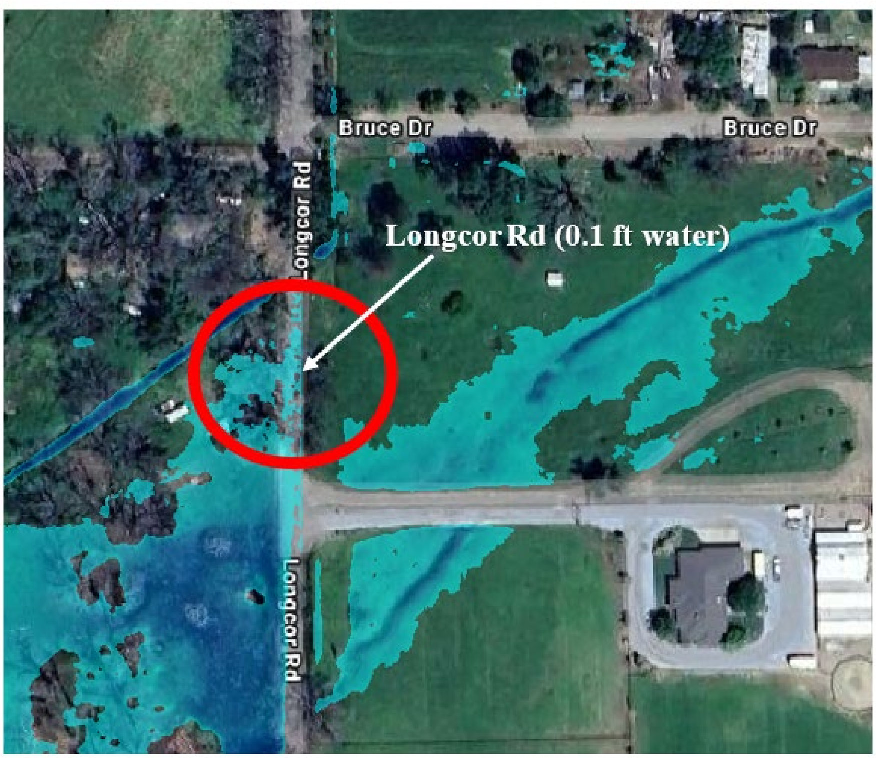
Source: Google Earth™

Figure 26. Evergreen Road Impacted from a 10-Year Event



Source: Google Earth™

Figure 27. Longcor Road Impacted from a 10-Year Event



Source: Google Earth™

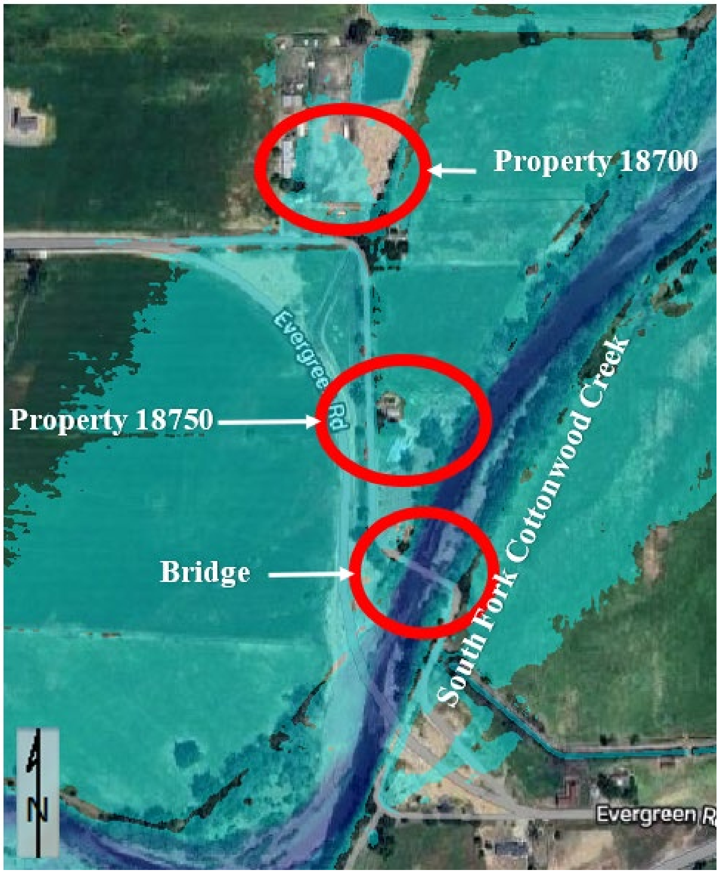
Figure 28. Traveled Way Impacted at a 10-Year event.

Source: Google Earth™

4.3. *2023 Flood Event Impact in the High-Risk Flood-Prone Zone*

The most recent flood event that caused road closures, property damage and restricted access in the High-Risk Flood-Prone Zone occurred in 2023, corresponding to a 6-year event. The flood inundation map shown in Figure 29 indicates that the Evergreen Road Bridge on the South Fork of Cottonwood Creek, along with two properties (18700 and 18750) located on the north bank, could have been impacted by the 2023 event. Figure 30 provides a visual representation of water level relative to Property 18750, with an estimated inundation depth of approximately 1.2 feet, as viewed in Google Earth™.

Figure 29. Evergreen Road Bridge Across the South Fork of Cottonwood Creek for the 2023 Event



Source: Google Earth™

Figure 30. Estimated Water Level on Property 18750 for a 10-year Event (comparable to 2023)

Source: Google Earth™

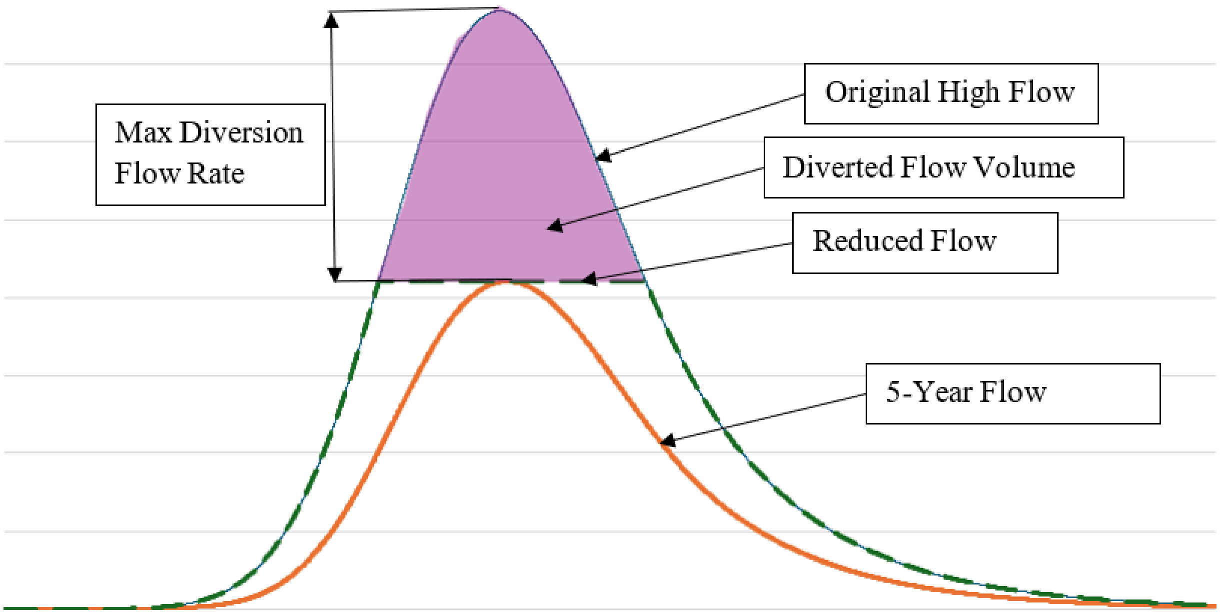
5. Flood Reduction and Recharge Analysis

To mitigate water levels within the High-Risk Flood-Prone Zone, diversions under California Water Code 1242.1 would be allowed when flow exceeds the flood diversion threshold. Based on a comparison of historical flood events and watershed model results, flooding begins when flow at the CNRFC gage CWAC1, located east of the I-5 Bridge, exceeds approximately 38,000 cfs in the main channel of Cottonwood Creek (a flow rate associated with a 5-year flood event). Therefore, a flood diversion threshold of 38,000 cfs, based on measured or forecasted 5-day flow at CWAC1, is proposed to reduce flood risk in the High-Risk Flood-Prone Zone. According to the watershed model, this threshold in the main channel corresponds to a flow of 15,365 cfs in the South Fork Cottonwood Creek at the cross section near Evergreen Bridge, as illustrated in Figure 32.

Higher flows were evaluated in the hydraulic model to estimate the floodwater volume that would need to be diverted from the main channel into a groundwater recharge basin or onto available recharge areas.

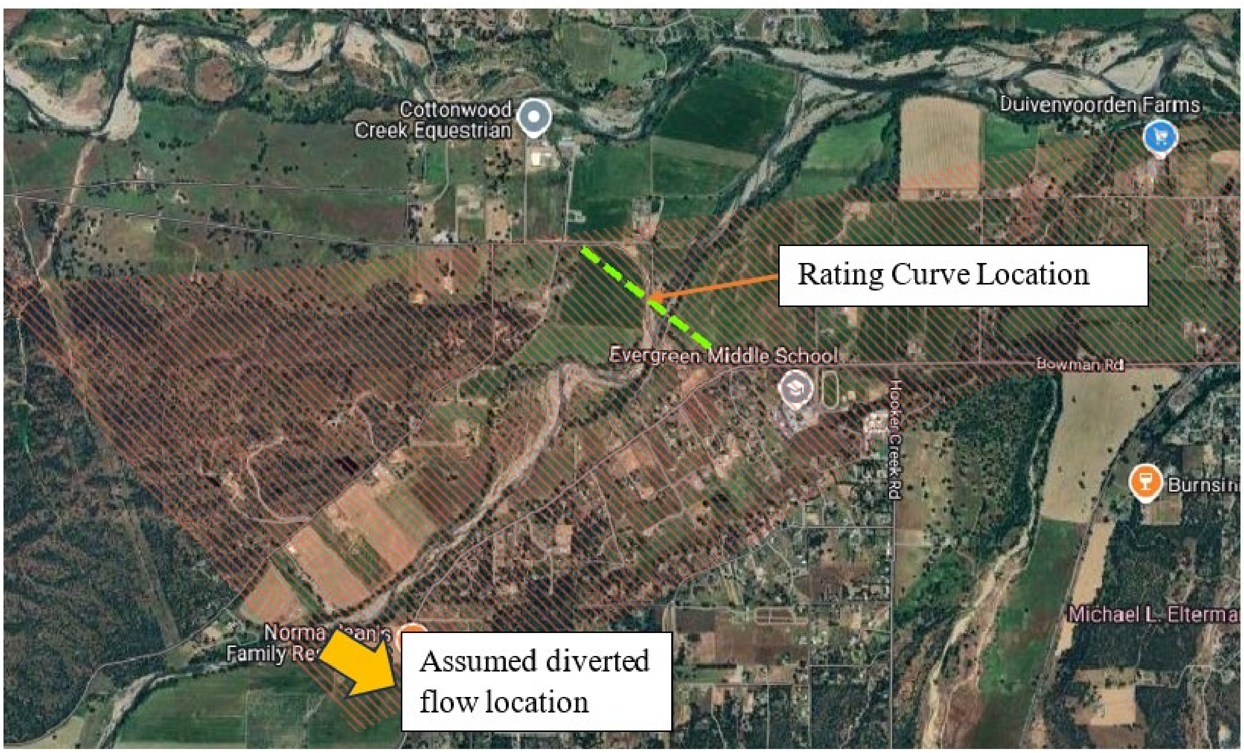
- Figure 31 shows the flow hydrograph volume (purple color) that would be removed to reduce the peak flow to the 5-year flow threshold. The purple area represents the estimated volume required to be diverted out of the channel discussed in Table 5.
- Figures 32 and 33 present the flow-versus stage rating curve from the HEC-RAS model for a cross section located on the South Fork of Cottonwood Creek. The result shows the maximum water depth reduction in the channel from a 10-year flow to a 5-year flow.
- Table 5 represents the proposed diverted volume upstream of the High-Risk Flood-Prone Zone on the South Fork Cottonwood Creek. The results show the range of return periods (1st column in Table 5) to reduce the in-channel water depths to a 5-year water level by diverting flows and volumes (2nd and 4th columns) on the South Fork. The reduced water depths (5th column) after diversion to a 5-year water level were based on the green dashed line shown on Figure 33. The last column is the are required for groundwater recharge. The area has been calculated so that water can be recharged within a day. The assumed recharge rate is 0.5 foot per day.

Figure 31. Conceptual Flow Reduction Hydrographs



Source: HMS

Figure 32. Flow versus Stage Rating Curve Location on the South Fork of Cottonwood Creek



Source: Google Earth™

Figure 33. Rating Curve Located on South Fork Cottonwood Creek

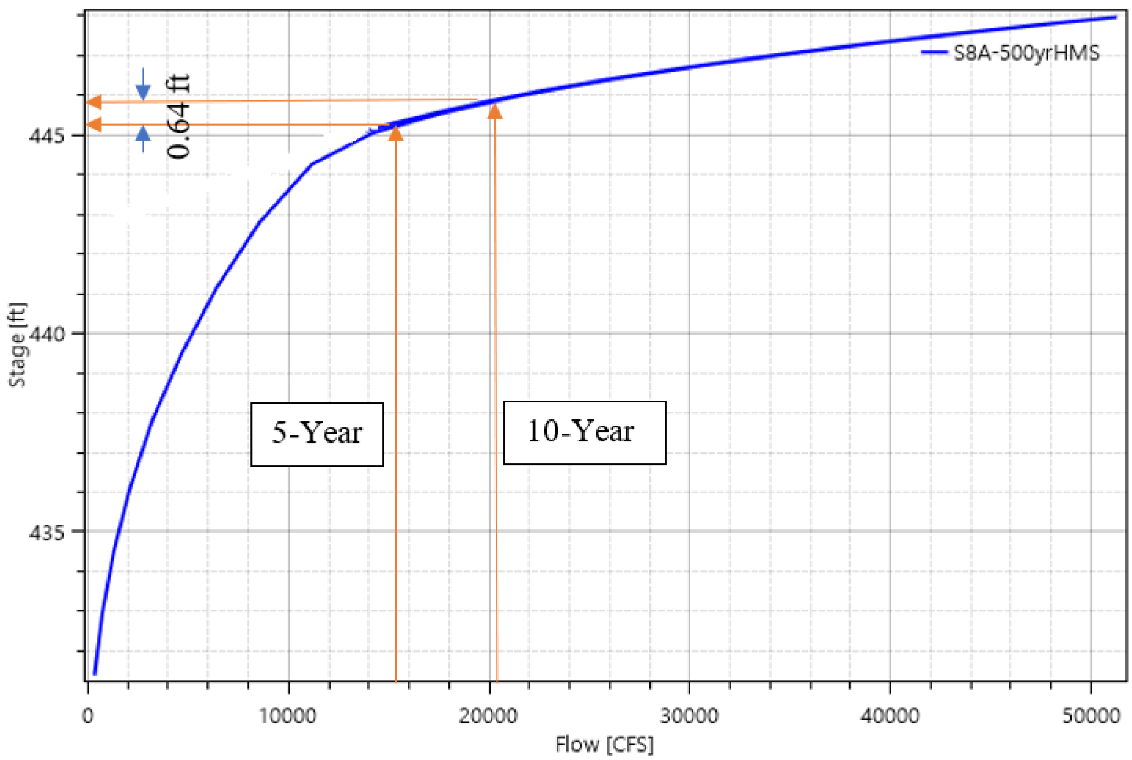


Table 5. Estimated Diversions Required to Reduce Flows to Flood Threshold in the Main Channel (38,000 cfs) or South Fork Cottonwood Creek (15,365 cfs)

Main Channel		South Fork Cottonwood Creek			
Return Period	Main Channel Peak Flow (cfs) w/o Diversion (from Table 1)	Peak Required Diversion Flow (cfs)	Diversion Flow Volume (ac-ft) **	In-Channel South Fork Water Depth Reduction (ft)	Area Required (acres)***
5	38,000	0	0	0	0
6*	42,800	2,516	1,066	0.31	2,132
10	49,700	4,888	2,806	0.57	5,612
50	75,300	16,841	15,447	1.64	30,894

Notes: ac-ft = acre-foot/feet; cfs = cubic feet per second; ft = foot/feet

*the 2023 event was used to represent a 6-year event and calculate diversions in the table.

**during a flood event, it is assumed any portion of flow in the South Fork exceeding the 5-year return period threshold (15,365 cfs) would be immediately diverted out of the system and stored either in a groundwater recharge basin or an off-channel reservoir.

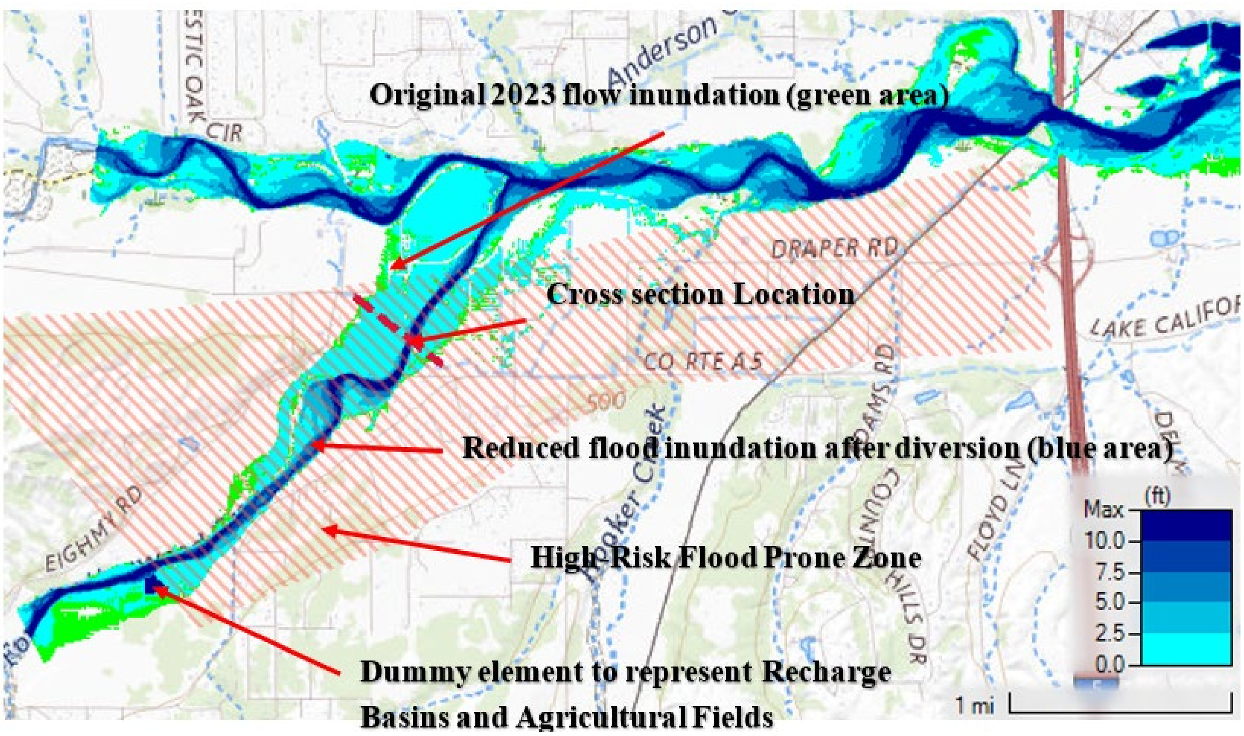
***assumes recharge rate of 0.5 foot per day and depth of 0.5 foot on entire area (example for 1,066 ac-ft over 2,132 acres with recharge rate of 0.5 foot per day, 0.5*2,132=1,066 ac-ft recharges in the single day, leaving 1,066 ac-ft of standing water on 2,132 acres equals a depth of 0.5 foot.)

Example Diversion for Recharge Basin

Using the 2023 flow event as an illustrative example (see Figure 34), proposed diversions to recharge basins and agricultural fields for groundwater recharge were assumed to be available just upstream of the High-Risk Flood-Prone Zone. During the simulation of the 2023 event, all flows exceeding the 5-year peak flow in the South Fork (15,365 cfs) were diverted from the channel into the recharge basins and agricultural fields during the event period. As indicated in Table 5, the maximum diversion rate achieved during this event was 2,516 cfs, which would require approximately 126 pumps, assuming each pump has a maximum capacity of 20 cfs.

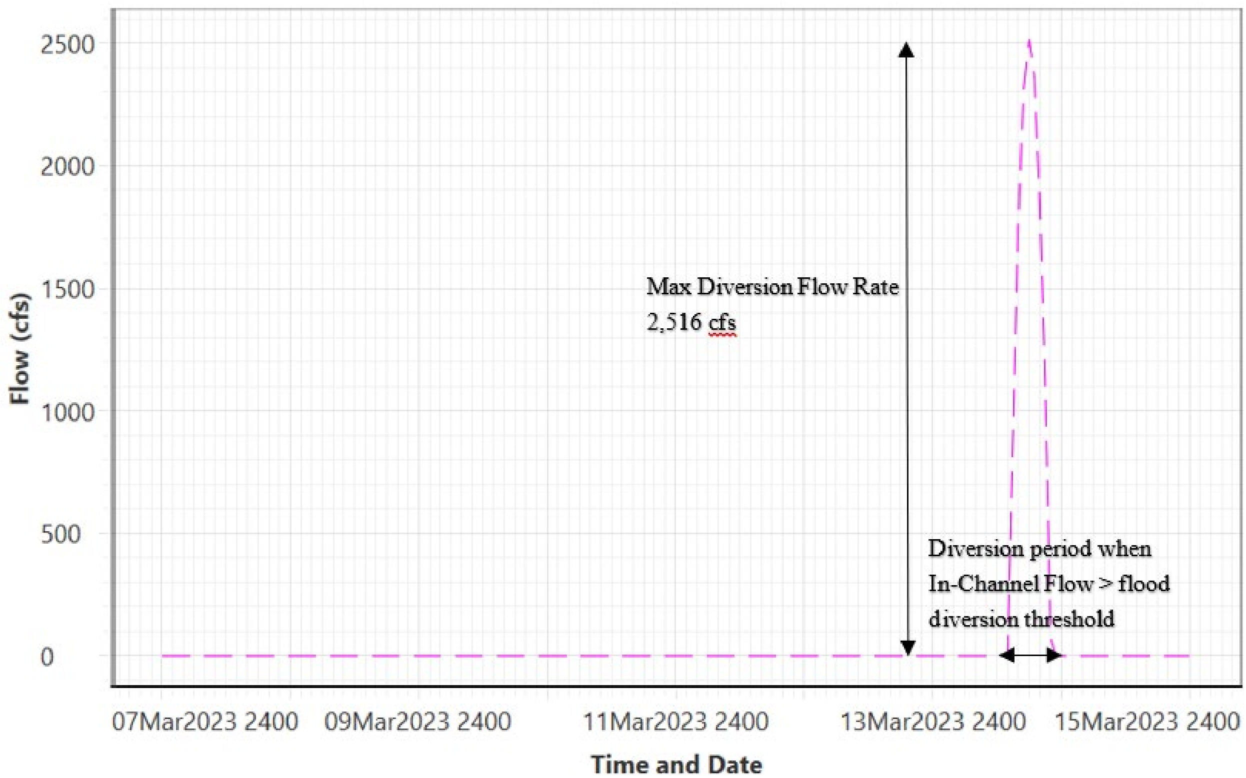
In the High-Risk Flood-Prone Zone, the green inundation area, representing no diversions, would be reduced to the blue inundation area, when diversions are in place. This reduction in the area inundated shows that the diversions would reduce a 6-year event to a 5-year event. Figure 35 illustrates the computed flow hydrograph that was diverted out of the river and into the groundwater recharge basin during this event period. The peak diverted flow is 2,516 cfs. Figure 36 shows the flow hydrograph comparison with and without diversion at the cross section (location is indicated in Figure 34). The maximum reduction on the peak flow values after diversion at the cross section is 2,516 cfs. Figure 37 shows the change in water level with and without diversions, displaying a reduction of 0.3 foot.

Figure 34. Maximum Inundation Area Comparison between Original and Reduced 2023 Events in the High-Risk Flood-Zone at South Fork of Cottonwood Creek



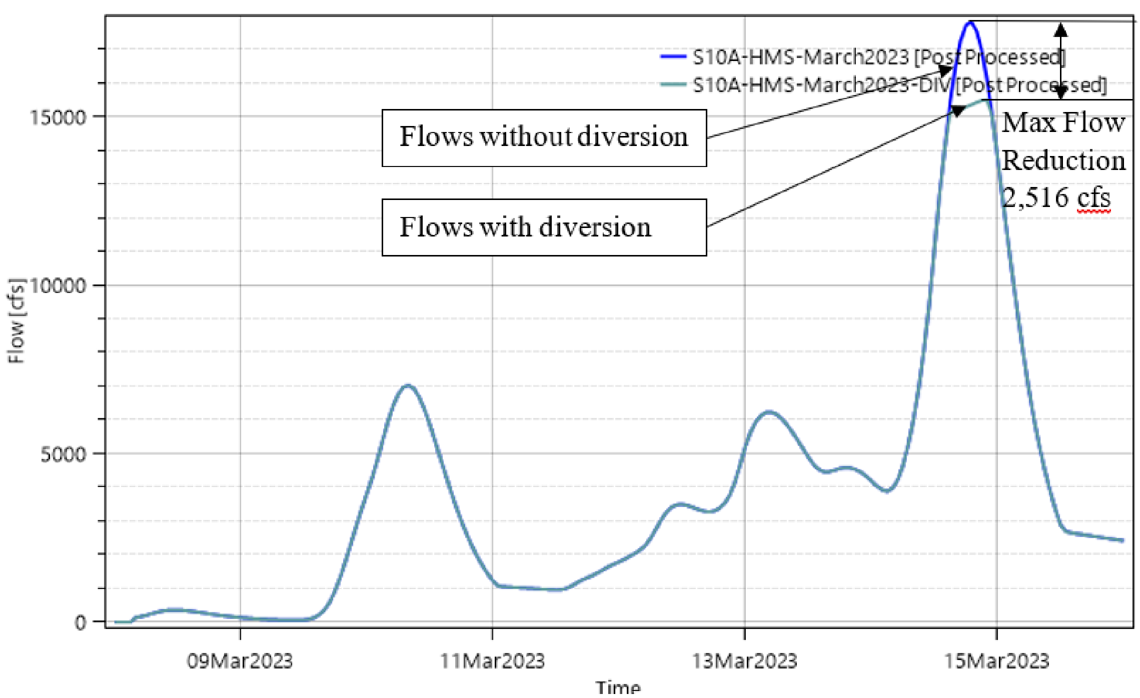
Source: HEC-RAS

Figure 35. Diverted Flow Hydrograph for Reducing the 2023 Event Flow to Prevent Flooding



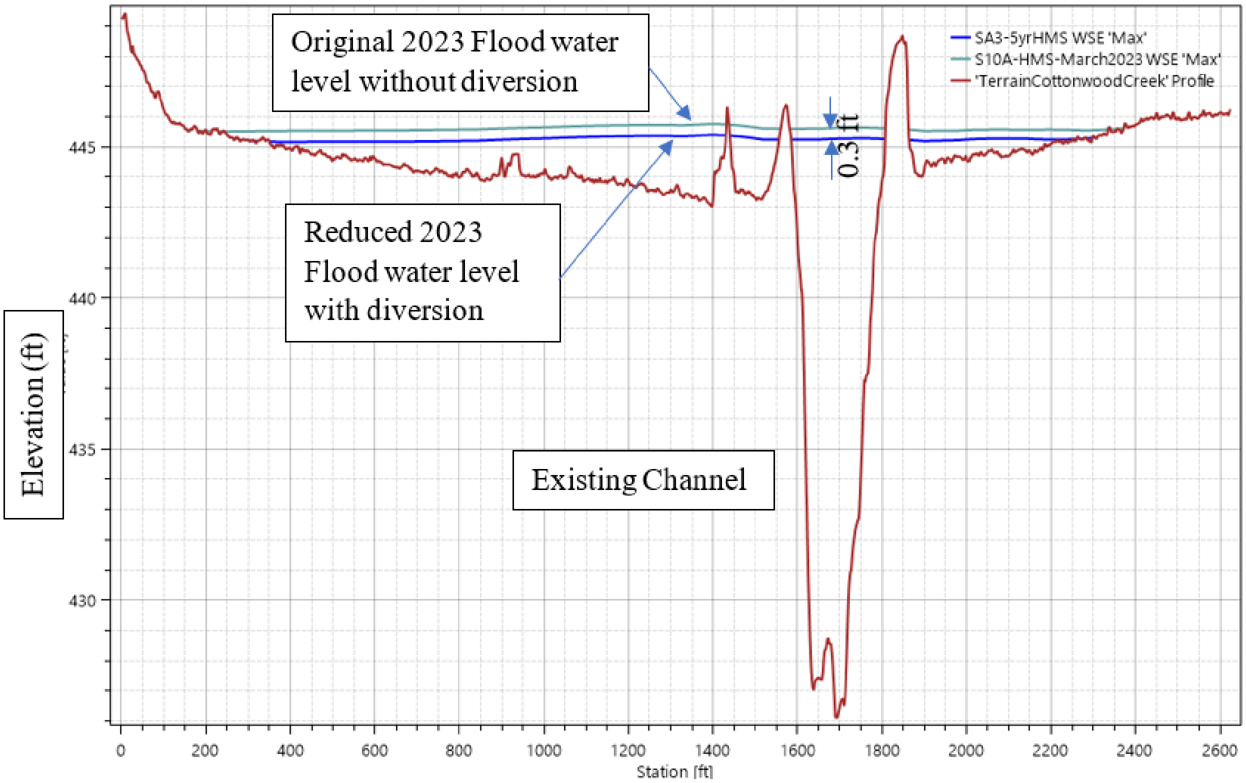
Source: HEC-RAS

Figure 36. Flow Hydrograph Reduction for the 2023 Event at the Cross Section at South Fork Cottonwood Creek



Source: HEC-RAS

Figure 37. Stage Reduction With and Without Diversion for the 2023 Event at the Cross Section in South Fork Cottonwood Creek



Source: HEC-RAS

Estimated Water Availability

The State Water Board's *Water Availability Analysis for Streamlined Recharge Permitting Guidelines* (2017) outlines two methods for estimating water availability. This study primarily focuses on Method 2 - the Threat of Flood Conditions, which defines the flood diversion threshold as the 5-year flood event, based on measurements at CDEC CWA.

In contrast, Method 1 - The 90th Percentile/ 20 Percent Method allows for diversions of up to 20 percent of daily streamflow when flows exceed the 90th percentile of historical daily flow between December 1 and March 31. This method assumes that sufficient water remains in-stream to satisfy senior water rights.

Table 6 provides estimates of water available for recharge using both methods, based on the historical daily stream USGS gage 11376000 records at Cottonwood Creek. It should be noted that because the proposed diversion point is at the South Fork of the Cottonwood Creek, all final diversion volumes in the table were adjusted by 43%, based on the drainage area ratio between the South Fork Subbasin and the entire Cottonwood Creek Watershed (397 mi^2 /927 mi^2 =0.43). Table 7 lists annual statistics of the number of days that water can be diverted out of the channel for all water year types. The definition of water year types was determined by DWR based on Sacramento Valley (WSIHIST).

Table 6. Statistics of Annual Groundwater Recharge Volumes Availability at South Fork Cottonwood Creek based on CDEC CWA at Cottonwood Creek.

Method		Min, acre-ft	Max, acre-ft (max year)	Average (including years with no volume available) (acre-ft)	Percent of Years with Volume greater than 0 (acre-ft)
Method 1	90th Percentile/20 Percent*	17	96,848 (1983)	15,000	87
Method 2	Threat of Flood Conditions**	48	3073 (1995)	224	27

Notes: ac-ft = acre-feet / foot; cfs = cubic feet per second; No. = number; **all diversion volumes were already reduced according to drainage area ratio 0.43**.

*The gage daily flow records are from 1940 to 2025. 90th percentile table was obtained from USGS gage website [USGS Surface Water data for USA: USGS Surface-Water Daily Statistics](). Statistics were calculated based on water year (from December to March for each year).

Table 7. Annual Days Diversion Available Based on Method 1 at Cottonwood Creek

Water Year Type	Days Water Available for Diversion			No. Years
	Min	Max	Average*	
Wet	4	57	22	29
Above Normal	3	26	14	12
Below Normal	1	21	6	15
Dry	1	13	4	17
Critical	1	12	3	12
All Years	1	57	12	85

*For each water year type, the Average values calculations include all years with zero diversion. However, the minimum estimates did not include years with zero diversion.

6. Conclusion and Next Steps

This evaluation utilized the best available terrain and gage data to identify higher-frequency flood events that resulted in road closures and property impacts along Cottonwood Creek. Based on a comparison of historical flood records and watershed model outputs, flooding within the High-Risk Flood-Prone Zone begins when flow at the CNRFC gage CWAC1, exceeds approximately 38,000 cfs – a flow rate associated with a 5-year flood event. Accordingly, a flood diversion threshold of 38,000 cfs, based on measured or forecasted 5-day flow at CWAC1, is proposed to reduce flooding in the High-Risk Flood-Prone Zone. Modeling results indicate that diverting approximately 2,500 cfs from the South Fork Cottonwood Creek over a 24-hour period could have reduced water levels during the 2023 event (an approximate 6-year frequency event) by approximately 0.3 foot within the High-Risk Flood-Prone Zone.

Flood conditions happen infrequently with an average annual recharge volume of 224 af (assuming all the available volume can be used). The Method 1 90th percentile/20 percent average annual volume and years available are much greater and likely enough larger to make pursuing long term flood diversion water right permits a viable option in addition to diverting as much flow as possible under Method 2.

The following information is recommended for inclusion in the flood diversion guidelines:

- Properties at 18700 and 18750 Evergreen Road are under imminent threat of flooding by flows over the flood diversion threshold of 38,000 cfs (Figures 29 and 30).
- Diversions under California Water Code 1242.1 at any location on the South Fork of Cottonwood Creek upstream from the Evergreen Road Bridge would be expected to reduce an imminent flooding threat.
- The flow threshold (flood diversion threshold) of 38,000 cfs at USGS Gage 11376000 -- Cottonwood Creek near Cottonwood, California is associated with the imminent threat.

The following are possible next steps for Cottonwood Creek:

- Evaluate the suitability of installing a stream gage at the Evergreen Bridge to establish local and more refined flood diversion thresholds.
- Conduct suitability analysis for recharge on land upstream of flooding area to identify potential recharge locations.
- Compile existing diversions and capacity off the South Fork of Cottonwood Creek, upstream of flooding area.
- Initial screening of possible additional diversions off the South Fork of Cottonwood Creek upstream of flooding area.
- Confirm reference elevation for the CWAC1 gage.
- Review and develop alternatives solutions for flood risk reduction and groundwater recharge locations.

7. References

State Water Resources Control Board (State Water Board). 2017. *Water Availability Analysis for Streamlined Recharge Permitting*. Sacramento CA. Accessed October 2025: https://www.waterboards.ca.gov/waterrights/water_issues/programs/applications/groundwater_recharge/docs/streamlined_waa_guidance.pdf

State Water Board. 2025. Updated to Reflect Executive Order N-16-25. January 2025.

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