Looking upstream at Harper Canyon Creek reach to be restored for sycamore alluvial woodland habitat (January 14, 2021)

Design Basis Report – Preliminary Design Level

Bourdet Ranch Grading Violation Abatement Project

Santa Clara County PLN20-139

August 2021

walls land+water, llc

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Contents

1. Preliminary Design Level Plan Overview

In association with Geomorph Design Group (GDG), walls land+water (WL+W) completed "Phase 1" preliminary design plans for implementing habitat restoration along parts of Harper Canyon Creek. The restoration work is required for abating grading violations within the Bourdet Property at Harper Canyon Rd in Santa Clara County, California (Santa Clara County PLN20-139). The violations resulted from unpermitted work on a rural ranch property including creek channel realignment, roadway crossings, roadway widening, roadway culverts, and channel grade control structures.

The conceptual design plans include measures for:

- Reservoir Spillway Channel Bed and Bank Erosion Protection – Downstream from the bedrock reservoir spillway, remove the channel-spanning concrete barrier block grade control and weir structure and replace it with an engineered boulder weir. Lay back oversteepened erodible soil slopes within the floodprone area and upland as needed and armor erodible banks below the 100-year water surface elevation with rock slope protection.

- Ford Crossing Improvement – At the existing low-water "ford" crossing, remove the channel-spanning concrete barrier block grade control structure and raised ford crossing and to restore natural channel geometry and uniform flows suitable for riparian revegetation. Rock-and-fill stabilized roadway ramps and at-channel-grade ford crossing.

- Creek Restoration & Bridge Replacement – At the grading violation site in the vicinity of the shop buildings and bridge, restore Harper Canyon Creek to an alignment and channel geometry similar to its pre-violation condition, suitable for restoration of Sycamore Alluvial Woodland (SAW) habitat, and installation of a new clear-channel-spanning replacement bridge.

The preliminary restoration design plans and associated technical advisory and design recommendations were developed based on expert geomorphic, hydrologic, and hydraulic analyses by WL+W and GDG:

- Historical geomorphic analysis for determining pre-violation channel geometry and habitat-supporting geomorphic features using historical air photos and LiDAR elevation data sets.

- A new hydrologic model of the entire Harper Canyon watershed and its individual subwatersheds for computing design peak flows and hydraulic modeling input data at key locations: reservoir inlet, reservoir spillway, downstream creek restoration, ford crossing improvement, and bridge replacement sites, and the seven unpermitted roadway culvert inlets.

- A new detailed two-dimensional hydraulic model of the mainstem Harper Canyon Creek downstream from the reservoir spillway for computing key return interval peak flow water surface elevations and flow velocities needed for restoration design and technical design recommendations.

These geomorphic, hydrologic, and hydraulic analyses and model computations also provided technical basis for design recommendations of associated infrastructure features to be designed by others:

- Roadway Culverts – Hydrologic model computed peak flows tributary to the seven individual roadway culverts for determining which culverts may need to be replaced for meeting Santa Clara County hydraulic design requirements.

- Bridge Replacement – Geomorphic design to restore pre-bridge creek bank geometry, and hydraulic model computed 50-year and 100-year peak flow water surface elevations at the restored bridge section for preliminary replacement bridge type selection and hydraulic design, including span length and soffit elevation.

2. Historical Geomorphic Analysis

Introduction

A historical geomorphic analysis of Harper Canyon Creek was completed using a review of literature, historical aerial photographs, LiDAR elevation datasets, and personal communications to determine channel geometry and habitat-supporting geomorphic features occurring before the grading violations described in Santa Clara County PLN20-139. The primary purpose of this geomorphic analysis is to document pre-violation conditions, but also recommend improvements to enhance stream channel conditions and floodplain habitat

Harper Canyon is an intermittent stream in the Diablo Range of southeastern Santa Clara County, CA. Harper Canyon is tributary to Pacheco Creek, which is a tributary of the Pajaro River (Figure 1). At the confluence with Pacheco Creek, Harper Canyon drains 6.3 mi^2 of primarily Franciscan geology and a small portion of volcanic geology in the northeastern portion (Figure 2). Various historic and active landslides are mapped throughout the watershed (Feltman 2020).

The poorly sorted nature of Franciscan mélange in the San Francisco Bay Area combined with steep topography, tectonic activity, and prevalent landslides lead to high sediment loads in stream channels (Elder 2013). Streams with high sediment loads relative to discharge tend to be dynamic, i.e. they change configuration frequently after large winter storms cause erosion and landslides deliver large quantities of coarse sediment to stream channels, and high-flows rework the streambanks and channel alignment. Often the channels become braided in low-gradient valley reaches as coarse sediment deposits when the slope decreases. Intermittent, braided stream reaches with relatively stable groundwater levels and periodic flooding are optimal conditions for Sycamore Alluvial Woodland (SAW) habitat (Keeler-Wolf et al. 1996). Historically, Harper Canyon supported SAW habitat along the valley floor in the project reach as it does in the confined reach, between the ford crossing and creek restoration reach (Figure 3).

A permitted in-channel earthen dam was constructed along the mainstem of Harper Canyon Creek in the 1970's (pers. comm. Lacy Bourdet, 2021), effectively disrupting coarse sediment (bedload) transport from upstream of the dam to the Project Area, which contributes 4.6 mi^2, or 74%, of the total drainage area. Typically, when the coarse sediment transport continuum is disrupted in alluvial streams by manmade constrictions or dams, there are geomorphic effects downstream. These effects include bed incision, bank erosion, and bed coarsening until bedrock or a new equilibrium is reached (Williams and Wolman 1984, Kondolf 1997, Vericat and Batalla 2006). The channel incision in particular can result in a decoupling of the floodplain (Williams and Wolman 1984). Although pre-dam construction topography is limited in resolution in the Project Area, it is likely that construction of the earthen dam led to channel bed incision in the unconfined alluvial reaches. This subsequently decreased floodplain inundation frequency and lowered groundwater levels, decreasing the opportunity and viability for sycamore floodplain recruitment and potentially increasing mortality of established sycamore trees. Therefore, SAW habitat was likely already degraded in the unconfined alluvial reaches before the violations occurred.

Although the impoundment on Harper Canyon Creek disrupts the transport of coarse sediment downstream, it generally is at or near capacity year-round and if not quickly fills during any significant rainfall event. Therefore, it essentially acts as a "run-of-the-river" reservoir and does not measurably reduce peak flows during storms the way that a larger impoundment would. Several large storms in January and February 2017 produced high-flows in the region (Figures 4a, 4b). The stream gages in Pacheco Creek (USGS gage 11153000) and Upper Coyote Creek (USGS gage 11169800) recorded the January 10-11, 2017 event which was estimated to be a 25-year storm (SFEI & H.T. Harvey, 2017). This storm and the February 7 storm, the latter of which caused Anderson Dam to spill and the flooding of Coyote Creek in San Jose, also caused significant erosion along Harper Canyon Creek which precipitated some of the unpermitted in-channel grading.

Methods

Historical aerial photographs from 1939 and 1956 were georeferenced in GIS and reviewed for geomorphic changes in the last century. More recently, satellite photography in Google Earth and aerial photography orthomosaics from Santa Clara County (years 2006, 2010, 2014, and 2016-2020).

High-resolution topographic information prior to the grading violations is limited to LiDAR flown for the County of Santa Clara in 2006. However, elevation contours generated from the LiDAR are limited to 5 ft intervals in the upland areas of the County, which is too coarse for measuring stream channel geometry.

To improve upon the 2006 LiDAR contours made available by the County, the raw LiDAR point cloud was downloaded, ground points were classified, and a digital terrain model and 1-ft contours were created in LiDAR processing software (LP360). It should be noted that the point cloud from 2006 was relatively sparse (average of 0.5m ground point spacing) compared to contemporary LiDAR point clouds, and was even more sparse within the channel, likely due to water being present at the time of survey (LiDAR pulses reflect off water surfaces and do not return ground elevation information), and beneath tree canopy (Figure 5). However, the 2006 LiDAR was useful in determining top of bank limits and elevations, and the occasional cross-section within the channel.

Analysis of post-violation / existing conditions relied heavily on aerial photography and photogrammetrically-derived topography (Towill 2020), and was supplemented with site visits in and topographic surveys using a total station, RTK-GPS, and unmanned aerial system (UAS / drone) surveys in January and April 2021. We also used 2018 LiDAR and the recently released 2020 LiDAR in our analyses.

Results

Detailed results of the geomorphic analysis are provided for the three main project components in Chapter 5 (Reservoir Spillway Channel Bed Bank Erosion Protection, Chapter 6 (Ford Crossing Improvements), and Chapter 7 (Creek Restoration & Bridge Replacement).

3. Hydrologic Modeling

The Harper Canyon watershed drains more than 4,000 acres of remote forest and grassland foothill ranchland terrain underlain by primarily permeable soils. Per Santa Clara County Drainage Manual (SCCDM) recommendations, the Army Corps of Engineers HEC-HMS hydrology model was selected for computing the peak stream flows at key locations in the watershed for engineering design. HEC-HMS simulates the hydrologic process of rainfall, runoff, and stream flow routing for computing the peak flows at locations in the watershed. Per SCCDM requirements for drainage areas larger than 200 acres, the unit hydrograph method was used for hydrologic model simulations.

The HEC-HMS model was run with 6 design hyetographs to simulate peak flow discharges at key locations in the watershed for engineering design (Table 1). These values are the design peak discharges determined by methods required by the SCCDM for the evaluation and design of existing and new hydraulic structures within the Harper Canyon watershed.

Table 1
Harper Canyon Watershed
HEC-HMS Simulated Peak Flow at Selected Locations

#	Location	Drainage Area (mi^2)	HEC-HMS Simulated Peak Flow (cfs)					
			2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
1	Culvert 1	0.039	0.6	2.6	4.9	10.0	14.1	18.5
2	Culvert 2	0.011	0.1	0.6	1.3	2.7	3.9	5.2
3	Culvert 3	0.190	5.3	28.5	56	85	106	128
4	Culvert 4	0.003	0.1	0.5	1.0	1.5	1.9	2.3
5	Culvert 5	0.012	0.7	3.3	5.7	8.2	10.1	12.0
6	Culvert 6	0.667	15.5	52	118	187	242	298
7	Culvert 7	0.016	0.4	3.3	6.4	9.5	11.9	14.3
8	At Basin Outlet	6.274	116	318	737	1,205	1,585	1,976
9	At Bridge	6.159	117	320	745	1,217	1,600	1,995
10	Reservoir Inflow	4.586	92	257	592	965	1,264	1,573
11	Reservoir Outflow	4.586	88	249	568	938	1,235	1,541

Please see Technical Memorandum #1 (Appendix A-1) for complete documentation of the HEC-HMS modeled watershed hydrology.

4. Hydraulic Modeling

HEC-RAS hydraulic modeling of the main channel from the reservoir to the watershed outlet to Pacheco Creek for both existing and proposed project conditions, and the evaluation of the existing culvert outlet flow conditions to ensure culverts meet the 100-year design criteria under existing conditions. The Harper Canyon HEC-RAS model was developed as a 2D unsteady state model using the latest release HEC-RAS 6.0. The terrain elevation surface was derived from the LiDAR data flown in 2020.

Six scenario runs were performed: combinations of three flow events (Q2yr, Q10yr, and Q100yr) and two geometry conditions (existing channel geometry and the proposed project channel geometry).

The modeling results at the key locations are highlighted as follows:

1. In the spillway channel reach, the 10-year peak flow velocities vary from 7 ft/s to 22 ft/s for both existing and project conditions. 100-year water surface elevations were computed and plotted on channel cross-sections to show where flows would exceed the limits of exposed bedrock and wet channel banks with exposed native soils susceptible to erosion;

2. At the ford crossing, the 100-year peak flow WSE is about 397.7 ft NAVD88 with a typical depth of 1.6 ft, and the velocity is about 15 ft/s for existing condition[1];

3. On restored floodplain surfaces in the Creek Restoration & Bridge Replacement Reach, the typical 2-year inundation depth is approximately 0.2 ft;

4. At the bridge replacement crossing, the 100-year peak WSE is about 335.3 ft NAVD88 with a typical flow depth of 5.3 ft and velocity of 9 ft/s for the "restored pre-bridge channel geometry with replacement clear span bridge" condition.

[1] The proposed at-channel grade shallow ford crossing will reduce water surface elevations compared to the modeled flows, produce more uniform flow conditions suitable for stable channel conditions and reestablishment of riparian vegetation.

Please see Figures (6a-b) for sample output from the hydraulic model and Technical Memorandum #3 (Appendix A-3) for complete documentation of the HEC-RAS hydraulic model development and computations.

5. Spillway Channel Erosion Protection Design

Photo 1. Drone photo looking upstream at spillway channel site and reservoir (January 14, 2021)

Geomorphic History

The reservoir and instream impoundment were constructed in the 1970s. The spillway channel for the reservoir was historically along the northwestern shore of the reservoir. In about 2010 the spillway channel was moved from the northwestern shore to the northern shore. Shortly after the spillway channel was moved it began to incise. The winter 2017 storms caused significant erosion in the spillway channel and it incised down to bedrock and widened significantly and exposed a continuous bedrock channel bed with nearly continuous bedrock channel banks (Figure 7). There are also two soil horizons overlying the bedrock, forming the exposed channel banks in places, and the slopes above the channel banks:

1. Lower: orange-brown colored older, partly consolidated colluvium consisting or angular rock fragments similar to the underlying native bedrock within a coarse sandy debris-flow matrix.
2. Upper: gray-brown colored younger, loose, fine-grained colluvial soil consisting of sandy-silt and silty-sand, with minor fine gravel sized angular clasts.

The lower older consolidated colluvium exhibits substantial erosion resistance and soil strength. However, recent creek flows eroded the material to produce an oversized channel, and continuing bank erosion should be anticipated in this material where it is exposed on the channel banks within reach of future high flows. The upper looser material has lesser bank erosion resistance and strength (Photos 2-3).

Photo 2. Looking downstream to the reservoir spillway channel from the left bank at the concrete barrier block weir. The over-widened channel has primarily bedrock banks on the right bank with the loose upper alluvium forming steep slopes above the 100-year water surface elevation in background of view (April 6, 2021).

Photo 3. Looking downstream from the right bank showing the exposed bedrock and steep erodible soil layers to be laid back (April 14, 2021).

Close to the reservoir spillway, there is a 50-foot-wide channel-spanning weir constructed from 18 large (approximate unit dimensions 2′x4′x3′) concrete barrier blocks, weighing about 2 tons each (Photo 3). The concrete barrier block weir was apparently installed after winter 2017 to plunge reservoir spillway channel flows into the center of the channel, and/or to prevent potential headcut advance into the spillway channel segment immediately downstream from the reservoir spillway.

The Preliminary Design Plans are included as Attachment A and specific sheets are referred to by sheet number. The process and rationale for the preliminary design components are provided below.

Photo 4. Looking from left bank to right bank at concrete barrier block weir at upstream end of the reservoir spillway channel. A shallow layer of coarse alluvium covers part of the bedrock spillway channel upstream from the weir (to the right of view) (April 6, 2021)

Design Considerations

The existing spillway channel configuration is preferable to the former location due to the bedrock underlying the spillway.[2] The proposed measures for stabilizing the channel downstream from the spillway are: (1) remove illegally placed concrete weir and replace it with an engineered boulder weir; (2) stabilize exposed soil channel banks within reach of the 100-year peak flood waters with heavy rock slope protection; and (3) lay back oversteepened slopes of the upper loose

[2] Based on visual observations of the ground surface near the spillway. We did not confirm extents, depth, and stability characteristics of the bedrock occurring near the spillway.

unconsolidated soil horizon to a stable slope suitable for simple erosion protection (Figure 8, Sheets R11-12).

Hydrologic and hydraulic model-computed 100-year water surface elevations were used to determine limits of new bank erosion protection, and velocities were computed to preliminarily size rock materials and design an engineered boulder weir to replace the removed concrete barrier block weir. Oversteepened slopes would be laid back to maximum 2H:1V finished grade slope, and provided erosion control such as seeded biodegradable erosion control fabric and/or seeded straw-mulch cover.

The conceptual design plans include measures for:

1. Removing all of the concrete barrier blocks forming the channel spanning weir and replacing the weir and grade control function with an engineered boulder weir consisting of 2-ton to 4-ton boulders closely spaced together with adjacent boulders fastened together with epoxy-set galvanized wire rope. The boulder weir will have a low point in the middle of the channel for steering high velocity spillway flows entering the steep portion of the reservoir spillway channel toward the center of the channel and away from the channel banks.

2. Installing new 2-ton to 4-ton boulder rock slope protection along the left bank with top elevation minimum 1 foot above the hydraulic model computed 100-year flood water surface elevation.

3. Lay back exposed oversteepened upper horizon loose unconsolidated soil slopes to maximum 2H:1V finished slope and protect from surface erosion with CA native grass seed, straw mulch, and 100% biodegradable erosion control fabric, if needed.

6. Channel Ford Improvements Design

Photo 5. Drone photo looking downstream at ford crossing site (January 14, 2021)

The Ford Improvement Design project consists of the removal of a raised ford crossing and concrete barrier block grade control structure to restore natural channel geometry and uniform flows suitable for riparian revegetation. Rock-and-fill stabilized roadway ramps and at-channel-grade ford crossing would be provided to maintain infrequent vehicle access to the south side of the channel. The ford crossing site is located approximately 500 feet downstream of the impoundment (Sta 52+00).

The Preliminary Design Plans are included as Attachment A and specific sheets are referred to by sheet number. The process and rationale for the preliminary design components are provided below.

Geomorphic History

Prior to construction of the existing raised ford crossing, there was a ford crossing at-grade with the channel bed at this location connecting to a small road leading to the south side of the reservoir. Construction of the existing ford crossing began circa 2016, and to build it the channel was realigned approximately 80 ft to the northeast and a portion of the road was extended across the former creekbed (Figure 7). The ford crossing was constructed raised above the channel bed grade to match with the adjacent floodplain surfaces and connecting to a newly constructed road on the left bank towards the impoundment. Concrete barrier blocks were placed on the downstream side to act as grade control (Photo 6). This effectively created a coarse sediment trap behind the crossing. As the spillway channel upstream eroded, coarse sediment deposited behind the crossing, creating an artificially widened channel. There was a large pulse of coarse sediment from the winter 2017 storms and subsequent spillway channel erosion. Vegetation has established on the channel banks and parts of the channel bed immediately downstream of the crossing and the crossing itself appears to be stable.

Photo 6. Looking from left bank to right bank at concrete barrier block grade control structure spanning the channel immediately downstream from the existing ford crossing (April 6, 2021)

Design Considerations

The proposed design for the ford crossing is to remove the concrete barrier blocks and restore the ford crossing to an "at-channel-bed-grade" shallow ford crossing design (Figure 8, Sheets R8-R9). Ten percent-sloped rock-and-fill reinforced ramps will lead into the channel crossing on both banks. A natural rock-and-fill reinforced crossing will prevent erosion, water quality impacts, and minimize future repair work. The proposed design would allow sediment to pass during high flows and restore uniform flow conditions with lower high-flow water surface elevations and velocities, similar pre-violation conditions.

It is proposed that the stream channel maintains its current alignment to minimize disturbance of dense established vegetation downstream of the crossing and reduce the need for grading disturbances within the channel.

7. Creek Restoration Design & Bridge Replacement

Photo 7. Drone photo looking downstream at the Creek Restoration Design & Bridge Replacement site (January 14, 2021)

The Creek Restoration & Bridge Replacement reach consists of 1,600 feet of valley length, beginning where the creek exits the confined reach (Sta 27+00) to approximately 800 ft downstream of the current bridge crossing (Sta 9+00).

Geomorphic History

Prior to the grading violations, Harper Canyon Creek was in a relatively stable planform configuration since at least 1956 (Figure 9). This is likely due in part to the earthen dam construction in the 1970s that disrupts most of the coarse sediment supply and decreases lateral channel dynamism, as discussed in Chapter 2. There was an unimproved ford crossing at the existing bridge location.

Satellite and aerial imagery shows that the channel alignment remained relatively stable in this reach between 2006 and 2016. Therefore, the 2006 LiDAR serves as a pre-violation baseline for this reach. In 2016, before the grading violations occurred in this reach, aerial imagery shows that the creek was eroding the existing dirt road upstream of the existing concrete bridge along the right bank. The creek then made a near-90-degree left turn towards the valley wall, and then another 90-degree right turn towards the former ford crossing.

The large winter storms in January and February 2017 caused significant erosion in the creek and along the roadway approximately 300 ft up-valley from the former ford crossing. The channel was realigned to reduce erosional pressure on the existing access road. Between 2017 and 2018, the channel was further realigned to the west side (river left) of the valley floor, a concrete bridge was constructed at the site of the ford crossing, additional in-channel grading was conducted approximately 400-800 ft downstream of the bridge, and road improvements (widening, addition of recycled asphalt roadway surface stabilization) were completed.

Design Considerations

The Preliminary Design Plans for the Creek Restoration Design and Bridge Replacement Area show a restoration plan developed from historical information. The recommended designs for the grading violation abatements incorporate the results of the geomorphic analysis and are supported by the two-dimensional hydraulic model developed to fine-tune the conceptual design to meet geomorphic objectives relevant for SAW restoration. Simply restoring the channel bed alignment and topography to strictly pre-violation conditions would be a missed opportunity to restore valuable SAW floodplain habitat in the grading violation areas.

SFEI and H.T. Harvey (2017) completed Sycamore Alluvial Woodland Habitat Mapping and Regeneration study in the region, with study sites on upper Coyote Creek and Pacheco Creek. The latter study site is directly adjacent to the Bourdet property. They mapped geomorphic zones in relation to the establishment of mature sycamore trees. The geomorphic zones mapped included primary channels, secondary channels, gravel bars, floodplains, and terraces. Although Coyote Creek and Pacheco Creek have much larger drainage areas, the work provides a template for restoration of SAW in the project reach, and a similar approach was used in the design process for this project. Existing primary and secondary channels, gravel bars, 2-year floodplains were mapped, and these geormorphic units were created where possible in the preliminary design plans.

The Preliminary Design Plans are included as Attachment A and specific sheets are referred to by sheet number. The process and rationale for the preliminary design components are provided below.

Channel Alignment

Prior to grading violations, the creek was eroding the access road along the right bank and valley wall along the left bank upstream of the then-existing ford crossing. There was and still is additional erosion occurring along the left bank downstream of the ford crossing.

The proposed preliminary design for the reach (Figure 10, Sheets R3-R7) calls for excavation of a new primary channel near the location of the pre-violation channel. The new primary channel alignment eases the severity of the pre-violation channel meanders upstream of the bridge. The more sustainable configuration allows for some dynamism but provides a buffer from the existing road to reduce the risk and frequency of road repairs related to bank erosion. It also provides space for the construction of a lowered, more frequently inundated floodplain surface along both banks to allow recruitment of SAW species. (See *Floodplain Enhancements* below.) The proposed alignment also joins with the eastern tributary at a suitable distance from the roadway to minimize excessive grading and disturbance. The alignment of the channel downstream of the bridge was not altered by the grading violations, so it will not be changed by the restoration project.

Secondary / Overflow Channel

The 2006 LiDAR shows the existence of a swale or former secondary or overflow in the location of the existing creek channel (Sheet R5). The proposed design is to fill the constructed channel along the west edge of the valley floor but maintain the existing channel as a smaller secondary channel. The secondary channel begins at the upstream end of the project reach and is activated during a typical average winter high flows and would provide additional riparian habitat potential. The secondary channel's location adjacent to the terrace provides potential for groundwater from the hillslope to feed the channel and provide sustained winter aquatic habitat between high-flow events.

Channel Geometry and Slope

A typical design channel width (top of bank to top of bank) of 28 ft was determined using the average width for the reach approximated from the 2006 LiDAR and aerial photo analysis in the project reach (Sheet R5). A constructed 2:1 side slope to the channel toe leaves a toe of bank width of 25 ft. An average channel slope of 1.2% was determined by connecting natural bed elevations from upstream and downstream from the grading violation impacted reach and calculating slope along the approximately 1,800 ft length of the proposed thalweg alignment. The proposed design includes the construction of riffles at the upstream extent of the floodplain areas to increase channel complexity and further enhance floodplain inundation potential (see *Floodplain Enhancements*) below.

Floodplain Enhancements for Sycamore Alluvial Woodland Restoration

Opportunities to create floodplain surfaces and increase inundation frequency to support Sycamore Alluvial Woodland habitat were identified in the project reach upstream and downstream of the bridge in the areas where grading violations occurred. We located four locations for floodplain creation totaling approximately 54,000 sq ft. Floodplain enhancement sites were selected to maximize restored area potential, avoid conflicts with the road, and avoid disturbing mature trees.

Optimal floodplain elevations were estimated at first using the existing natural floodplain surface with two large sycamores occurring along the right bank just downstream of the bridge as a reference (Photo 7-8). The floodplain elevation here is approximately 1.5-2 ft above the thalweg.

Photo 7. Looking downstream to Harper Canyon Creek channel downstream from the existing roadway bridge crossing to be removed. The sycamore trees on the right bank are rooted on a floodplain with a surface only 1.5-2 feet above the adjacent channel bed (April 6, 2021)

Photo 8. Drone photo looking downstream to Harper Canyon Creek channel downstream and the sycamore trees on the right bank are rooted on a floodplain (January 14, 2021)

The 2D hydraulic model was then used to test the proposed restored stream channel with various iterations of floodplain elevations. Results demonstrate that a flood elevation of 1.5 ft corresponded to greater than a 2-year flood. The target floodplain elevation that would be inundated by the 2-year flood was determined to be approximately 1 ft above the thalweg elevation. Therefore, the design floodplains were set at 1 ft above the thalweg elevation for each floodplain restoration site. Even at 1 ft height, the typical threshold of accuracy for construction in stream channel work, floodplain inundation was not complete according to the hydraulic model. Thus, constructed riffles are proposed at key locations for promoting floodplain inundation for immediate post-construction conditions. Hydraulic modeling shows the 2-year flood produces an average depth of 0.2 ft across the restored floodplain surfaces (Figure 11a).

New vegetated rock slope protection is proposed along the left bank downstream from the bridge where the eroding natural bank is close to the roadway edge (Sta 12+80 to 15+00). The vegetated rock slope protection is adjacent to the downstream-most constructed riffle. Slope protection here will avoid conflicts of the creek restoration hydraulics with the roadway (Sheet R4).

Bridge Replacement

The existing unpermitted concrete bridge and wingwalls will be removed and the channel bed and bank geometry will be restored to pre-bridge conditions. A clear-spanning bridge will be designed by a structural engineer to meet County of Santa Clara building and hydraulic design requirements, including roadway width, design loading, abutment foundation scour protection, and hydraulic freeboard. See Chapter 9 for details.

Design Conclusions & Caveats

This is a preliminary design plan to be submitted for agency approval, grading volume estimates, and cost estimation. Actual bed elevations and additional topographic detail will be developed in later design phases. Bed elevations and contours anticipated to be established by natural winter flows during the first 1-2 winters post-construction.

The proposed channel length in the project reach is approximately 1,800 feet (Sta 9+00 to 27+00), compared to a 1,950 foot pre-violation length. However, with the creation of 54,000 sq ft of Sycamore Alluvial Woodland floodplain habitat and an additional 500 feet of frequently activated secondary channel there is a vast improvement in ecological value compared to pre-violation conditions.

The restored floodplain provides the hydraulic and geomorphic setting for SAW recruitment, however success of recruitment is not guaranteed. Cattle and grazing animals should be excluded from the restoration area. Groundwater levels should be monitored to predict success of SAW recruitment and establishment. SFEI and H.T. Harvey (2018) provide an in-depth Planting Guide for Sycamore Alluvial Woodland that should be referenced for any restoration plantings.

The restored channel geometry at the bridge crossing and a clear span replacement bridge would lower water surface elevations, particularly for the 100-year flood (Figure 11c). See Chapter 9 for details.

8. Technical Recommendations 1: Roadway Culverts

Photo 9. Photo of downstream end of culvert 6 and concrete tailwall (January 14, 2021)

The HEC-HMS computed peak flows were determined for the inlet of each of the seven roadway culverts (Table 2). Under existing conditions, culverts #3 and #6 do not pass the 10-year design flow under free outfall conditions when the headwater surface is at the top of the culvert inlet (H/W=1) (Table 3). Culverts #3, #5, and #6 do not pass the 25-year design flow under free outfall conditions when H/W=1.

Table 2
Harper Canyon Watershed
HEC-HMS Simulated Peak Flows at Culvert Inlet Locations

#	Location	Drainage Area (mi^2)	HEC-HMS Simulated Peak Flow (cfs)					
			2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
1	Culvert 1	0.039	0.6	2.6	4.9	10.0	14.1	18.5
2	Culvert 2	0.011	0.1	0.6	1.3	2.7	3.9	5.2
3	Culvert 3	0.190	5.3	28.5	56	85	106	128
4	Culvert 4	0.003	0.1	0.5	1.0	1.5	1.9	2.3
5	Culvert 5	0.012	0.7	3.3	5.7	8.2	10.1	12.0
6	Culvert 6	0.667	15.5	52	118	187	242	298
7	Culvert 7	0.016	0.4	3.3	6.4	9.5	11.9	14.3

Table 3
Harper Canyon Watershed
Existing Inlet Control Culvert Flow Capacities

Culvert	Flow Capacity with Headwater at Top of Culvert (H/W = 1) (cfs)	Able to Pass Q10yr Flow with Headwater at Top of Culvert (H/W=1)?	Able to Pass Q25yr Flow with Headwater at Top of Culvert (H/W=1)?
Culvert 1	13.9	Yes	Yes
Culvert 2	6.8	Yes	Yes
Culvert 3	13.9	No	No
Culvert 4	6.8	Yes	Yes
Culvert 5	6.8	Yes	No
Culvert 6	48	No	No
Culvert 7	13.9	Yes	Yes

Recommendations:

- The number and/or size of the culverts should be increased at culvert sites #3 and #6 for meeting both the 10-year and 100-year design criteria per Santa Clara County requirements.
- The length of culvert #3 should be extended to conform with the creek restoration grading plan in that vicinity.
- The replacement/extended culverts at sites #3 and #6 should be provided poured concrete collar headwalls and tailwalls.
- The replacement/extended culverts at sites #3 and #6 should be provided rock energy dissipators with minimum length equal to 4.5 times the culvert diameter and minimum width equal to 4 times the culvert diameter. Fifty-percent of the rock shall be larger than 12-inch-diameter, and the dissipator shall be underlain with filter fabric or a minimum 6-inch-thick bedding layer of 6” minus rock.

Please see Technical Memorandum #2 (Appendix A-2) for complete documentation of the roadway culvert capacity evaluation.

Photo 10. Looking upstream to the subwatershed and poorly formed channel tributary to Culvert #3 (April 6, 2021)

9. Technical Recommendations 2: Bridge Replacement

Photo 11. Looking downstream from the left bank to the existing roadway bridge to be removed and replaced with a new bridge conforming with restored pre-violation creek bank slopes and meeting hydraulic design criteria (April 6, 2021)

The existing roadway bridge crossing (Photo 10) will be removed and replaced by a new bridge engineered to meet Santa Clara County hydraulic design standards and support findings substantiating a Federal Emergency Management Agency (FEMA) "No Net Rise Certificate".

Recommendations:

- The replacement bridge should clear-span between abutments set near the finished restored top of bank lines shown in the Creek Restoration site area grading plans. The finished restored top of banks lines (at Elevation +/- 337 feet NAVD88) are 56 feet apart at the bridge section.

- The 100-year flood water surface elevation for without bridge conditions is about 334.4 feet NAVD88, producing a flow top width of 45 feet at the bridge section.
- The bridge abutments for a clear-span replacement bridge may be set, preliminarily, as close as 45 feet apart, inside-to-inside diameter.
- Preliminarily, the bridge deck may have a soffit as low as 335.0 feet for passing the 100-year flood flow without freeboard, which should be acceptable for meeting Santa Clara County hydraulic design requirements and producing conditions for findings supporting a FEMA "No-Net Rise" Certificate.
- Should the abutments be spaced, preliminarily, at least 45 feet apart (requiring a replacement bridge deck spanning 45 feet), then approach wing walls and tail walls or detailing of abutment wall corners should not be required.
- Design iteration may be required between hydraulic model simulations and civil-structural engineering design for bridge type selection, especially if a bridge type requiring a deck soffit below elevation 335.0 feet be required, such as for avoiding roadway grading or sight-line impacts both sides of the bridge deck surface. Having the soffit protrude below the 100-year peak water surface elevation may be acceptable, so long as the pressure flow conditions created during the 100-year peak flow are minor, and do not substantially impact hydraulic performance for supporting a FEMA "No-Net Rise" Certificate, or require an unreasonably depth of scour countermeasures for protecting abutment foundations.
- The subsurface conditions are not known. Should bedrock or scour resistant alluvial soil materials occur at shallow depths below the channel bed elevation, required scour countermeasures may be minimal.

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Williams GP, Wolman MG. 1984. Downstream effects of dams on alluvial rivers. Reston (VA): US Geological Survey. Professional Paper nr 1286.

FIGURE 1. Watershed Vicinity Map

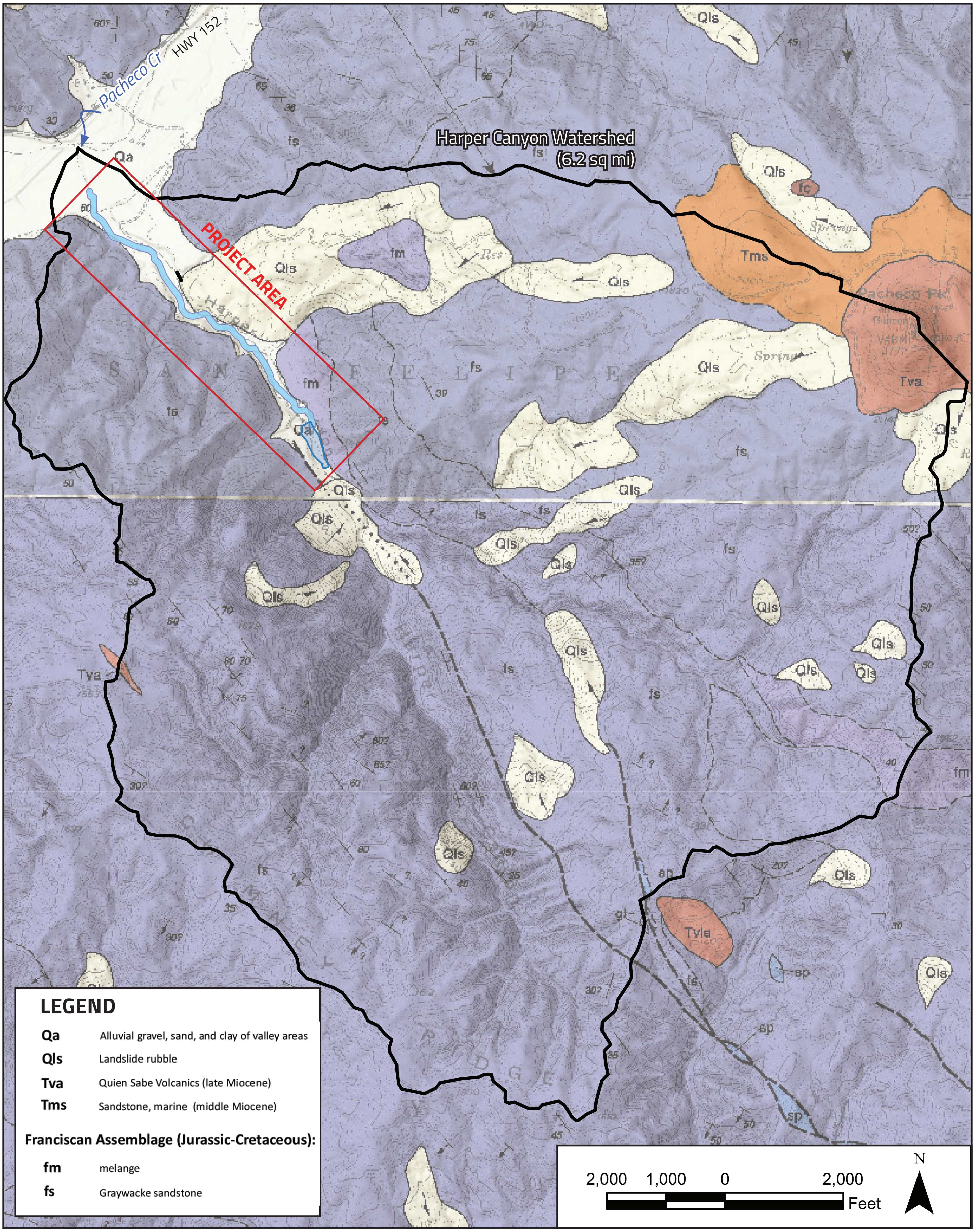
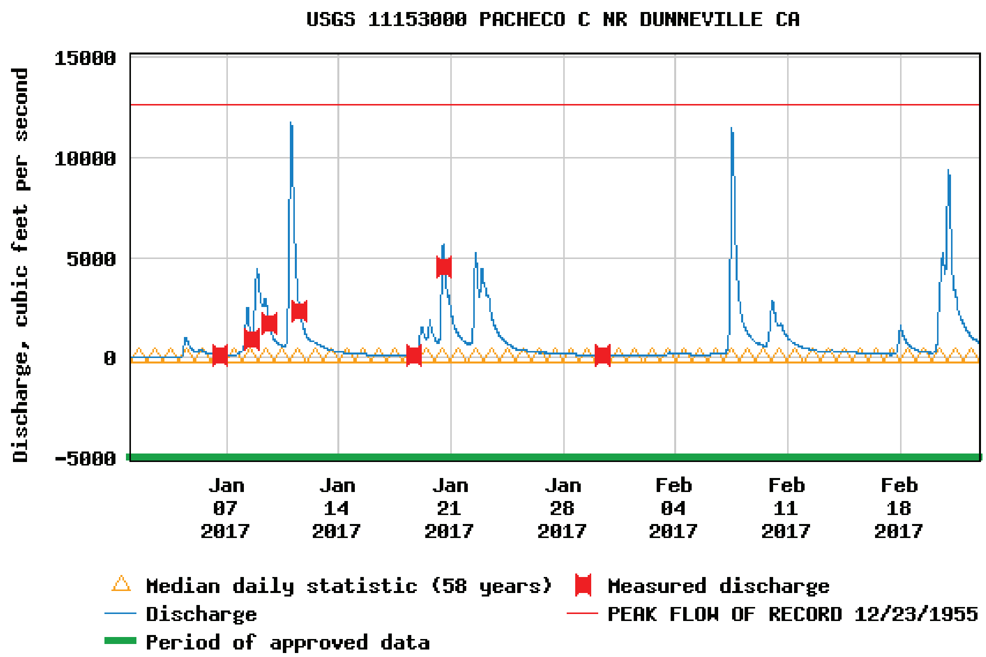
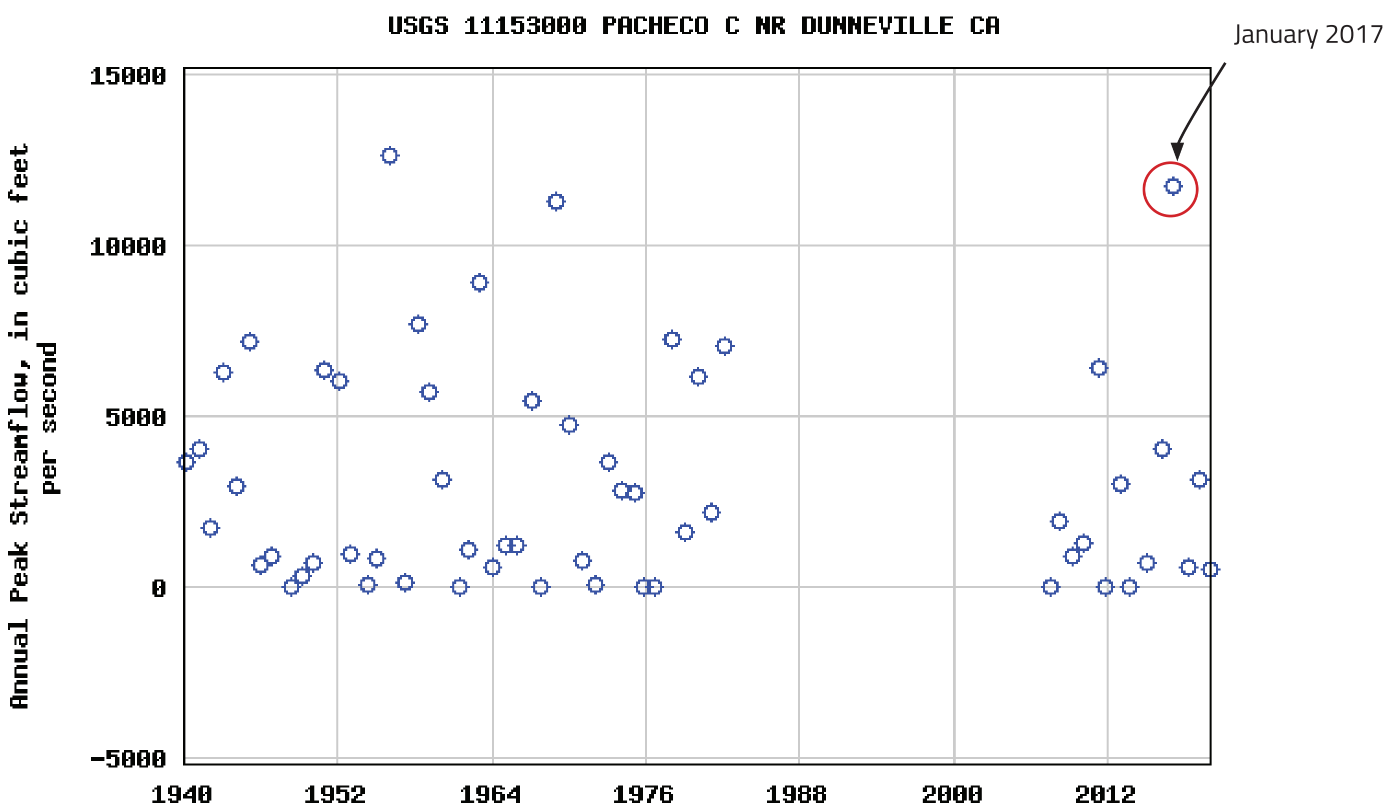


FIGURE 2. Watershed Geology Map

FIGURE 3. Project Area Map

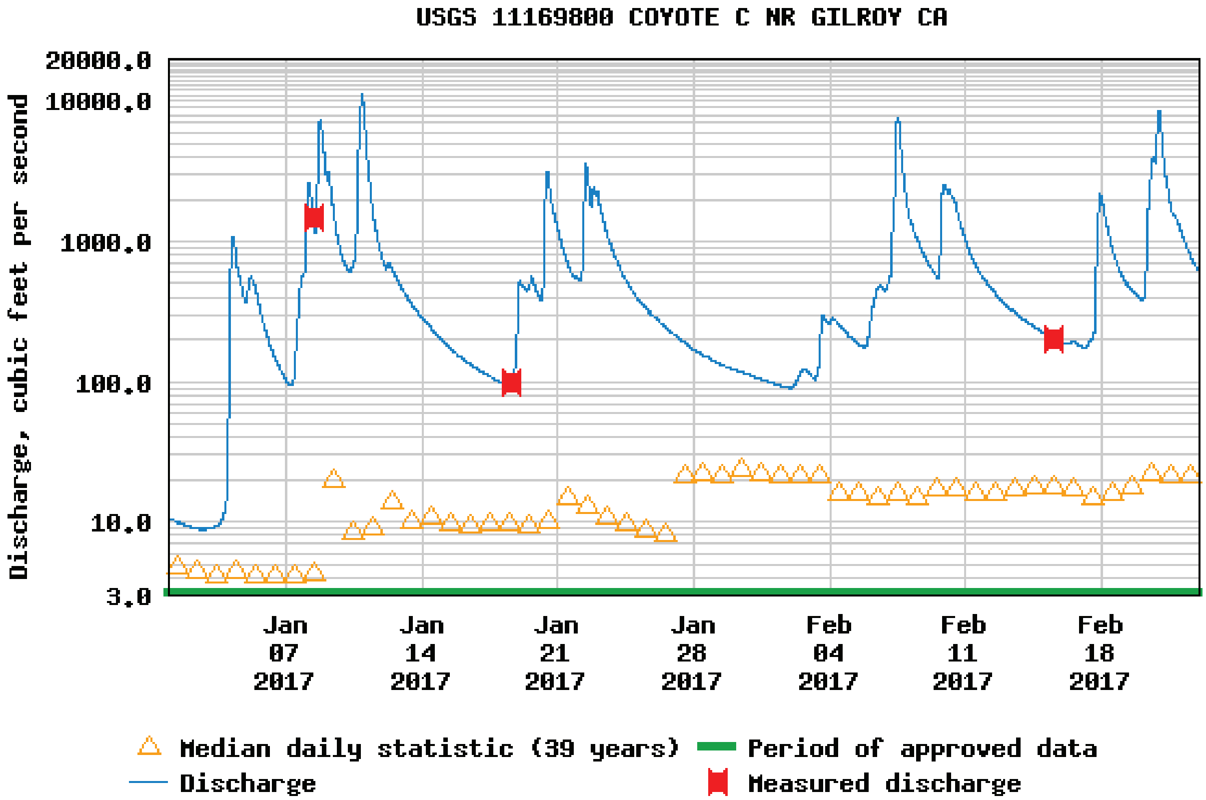


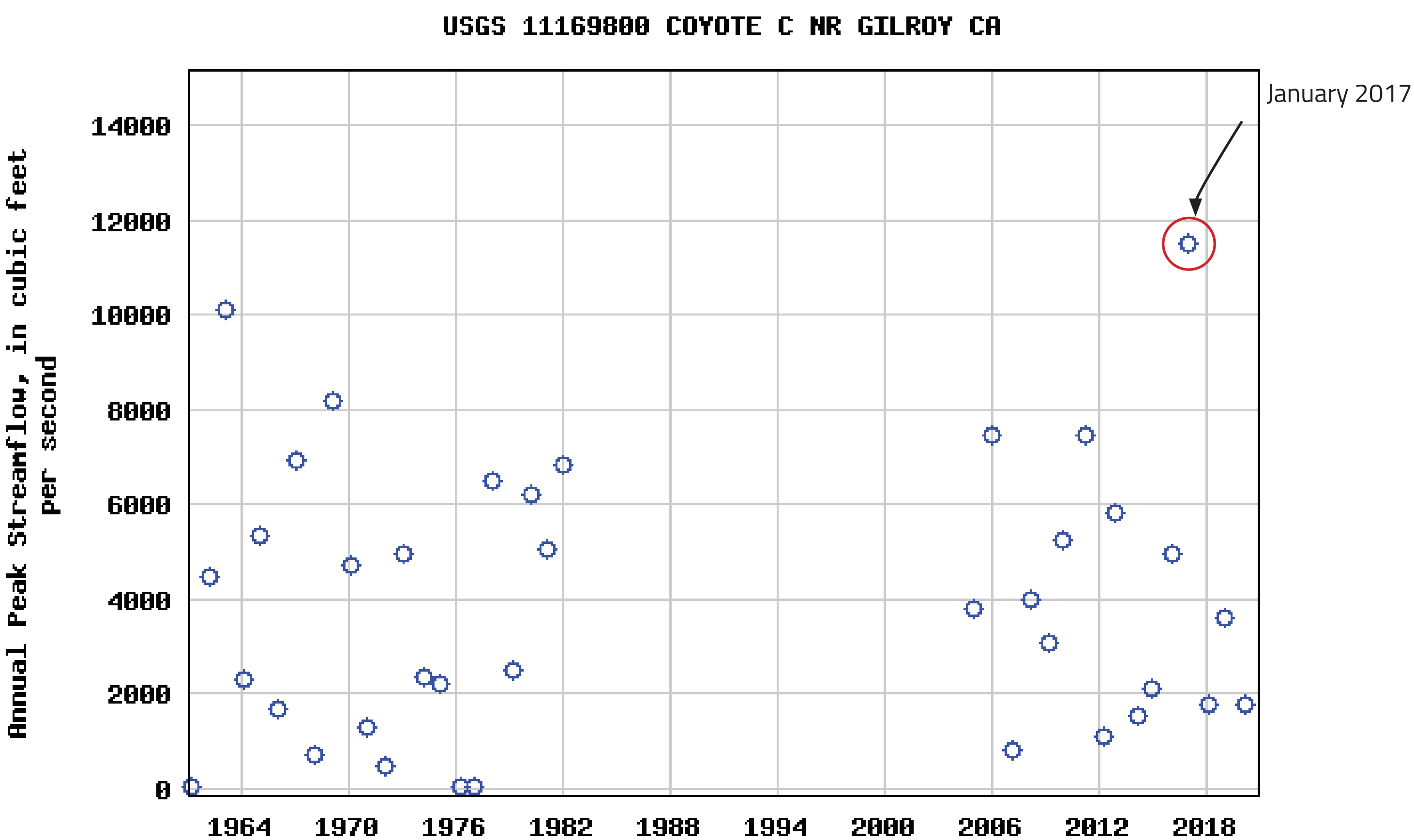
Winter 2017 Hydrograph at Pacheco Creek



Pacheco Creek Annual Peak Flows

FIGURE 4a. USGS Gage data for USGS 11153000 Pacheco C Nr Dunneville Ca

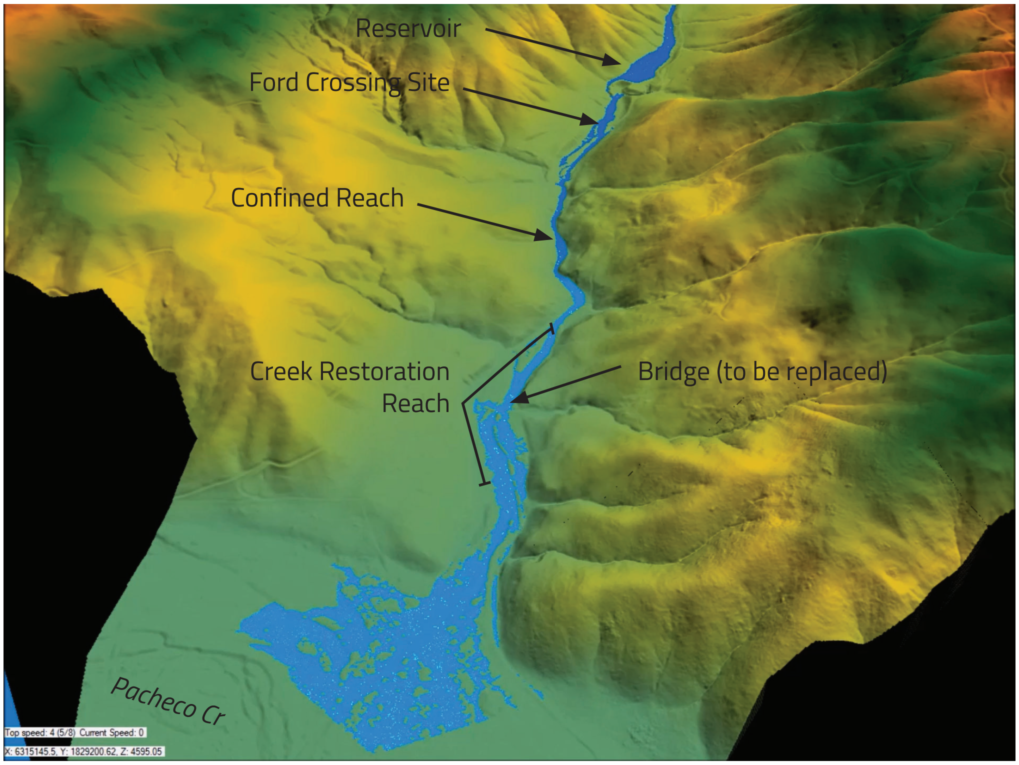




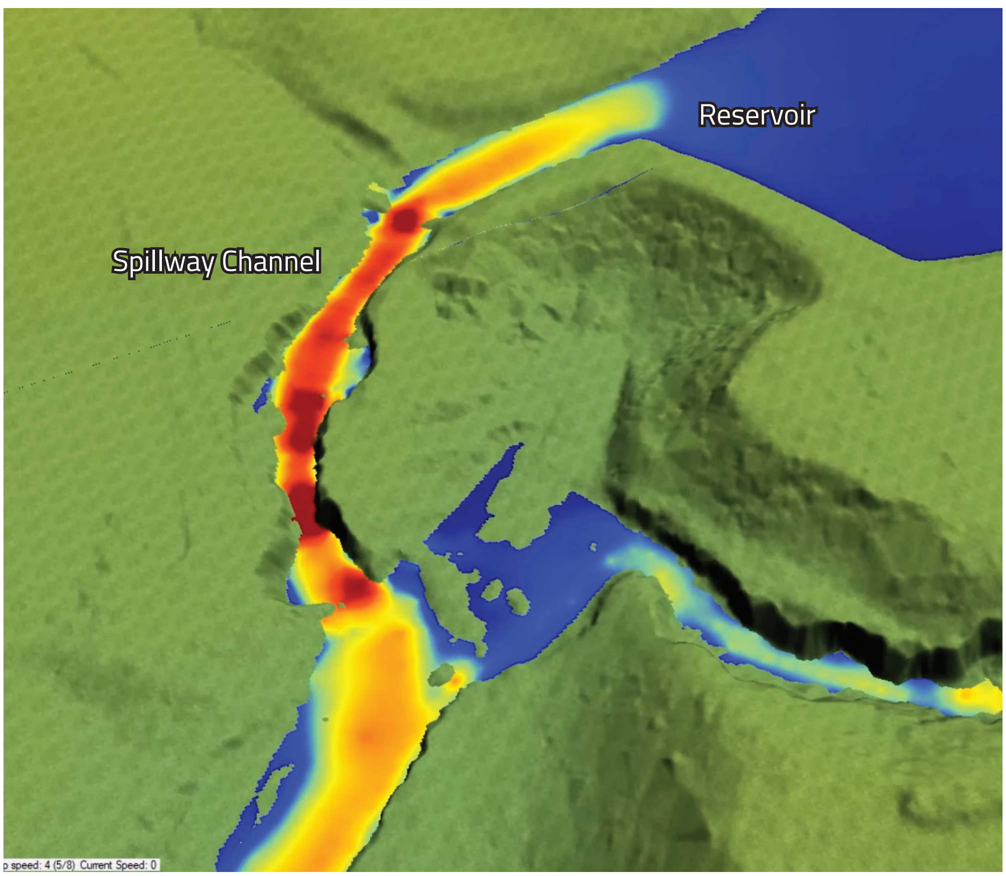
Upper Coyote Creek Annual Peak Flows

FIGURE 4b. USGS Gage data for USGS 11169800 Coyote C Nr Gilroy Ca

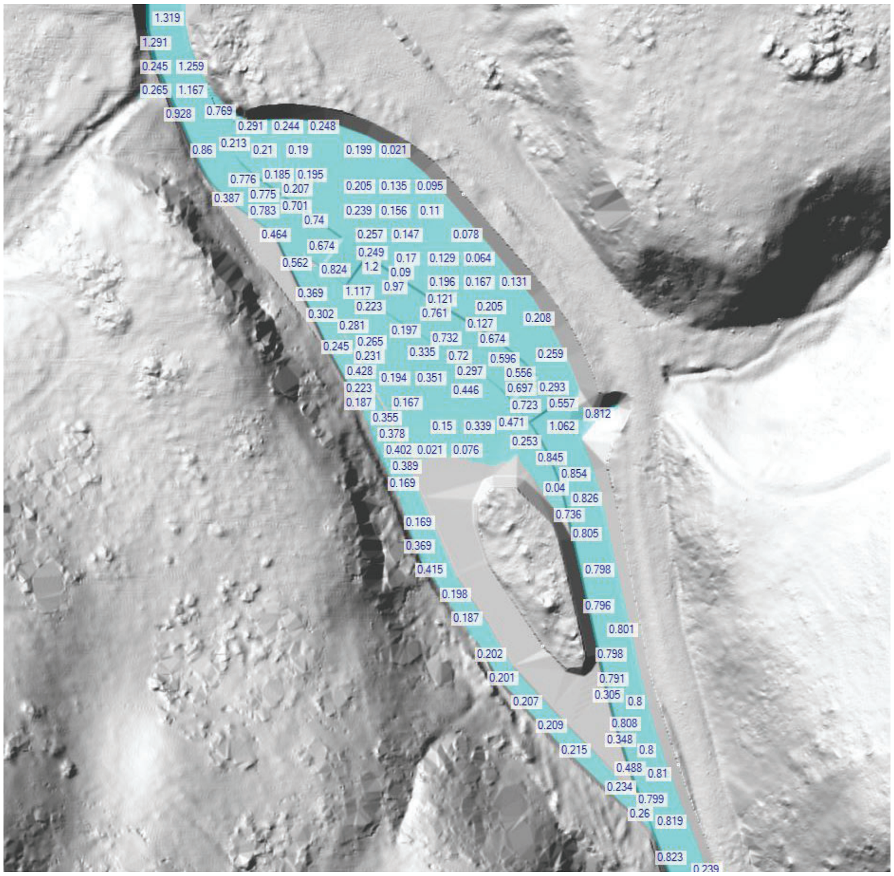
FIGURE 5. Comparison of LiDAR ground point cloud density between 2006 and 2018 datasets.



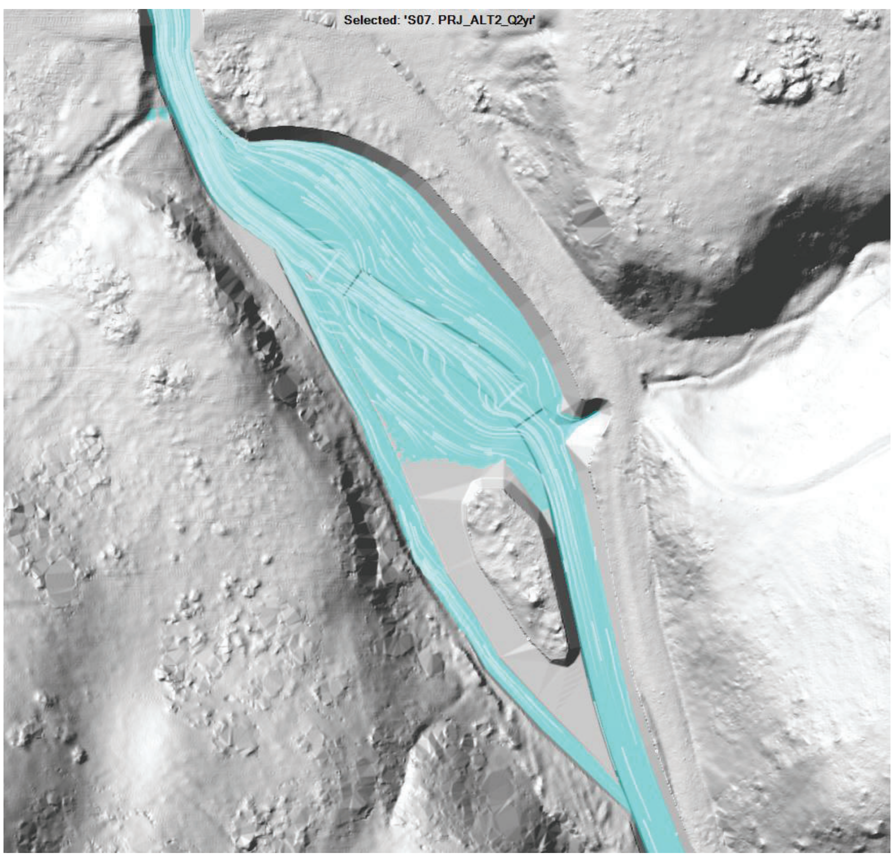
Screenshot of Existing Conditions 2-dimensional model of Harper Canyon Project Area



Screenshot of Existing Conditions 2-dimensional model of velocities in spillway channel



Screenshot of proposed 2-year flow water depths in the Creek Restoration site upstream of the bridge.



Screenshot of proposed 2-year flow vectors in the Creek Restoration site upstream of the bridge.

FIGURE 7. 2014-2018 Time Series of Spillway Channel and Ford Crossing Sites

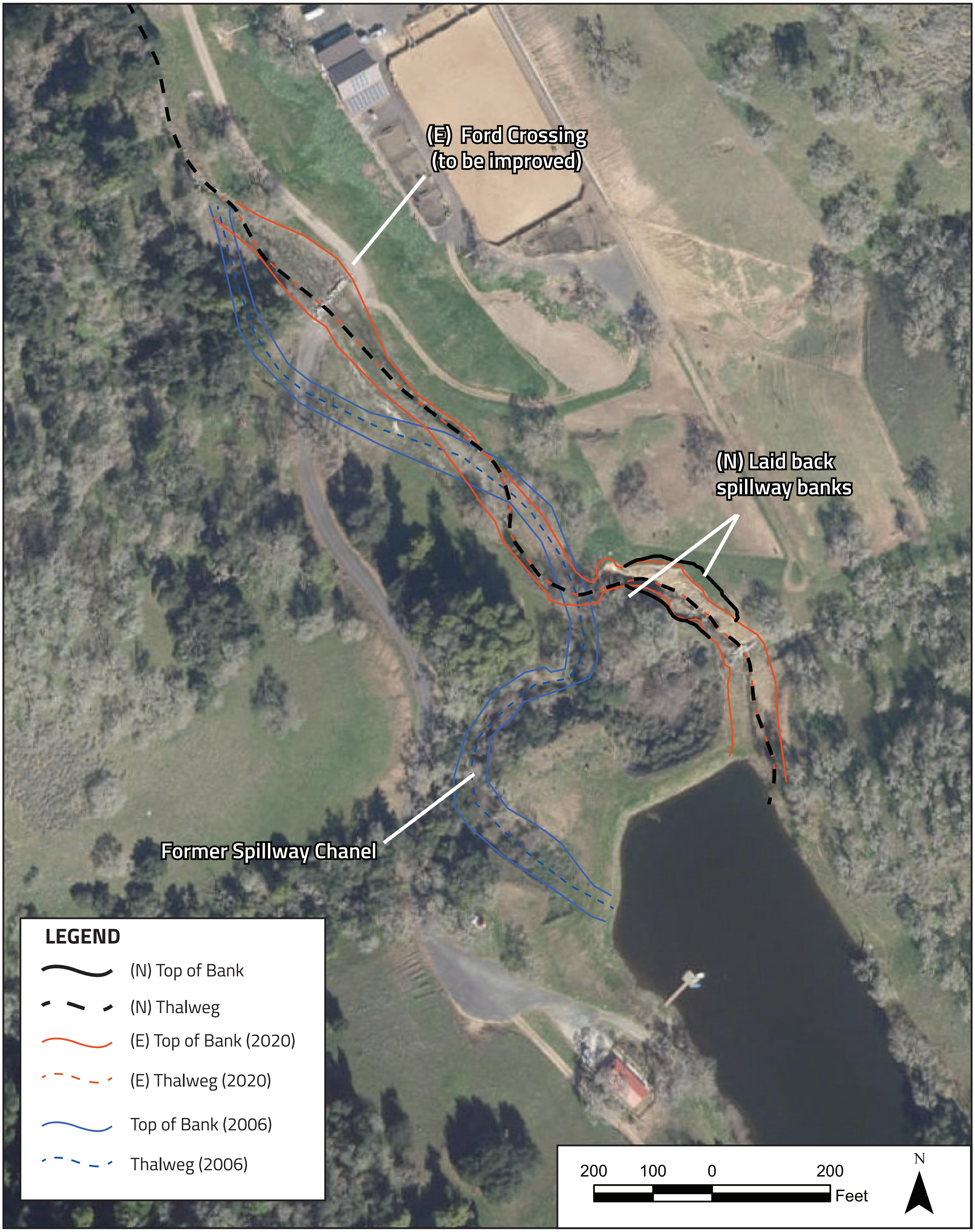


FIGURE 8. Pre-violation (2006), Existing Conditions (2020), and Proposed Top of Banks and Thalweg Alignments. See Sheets R8-R12 for details.

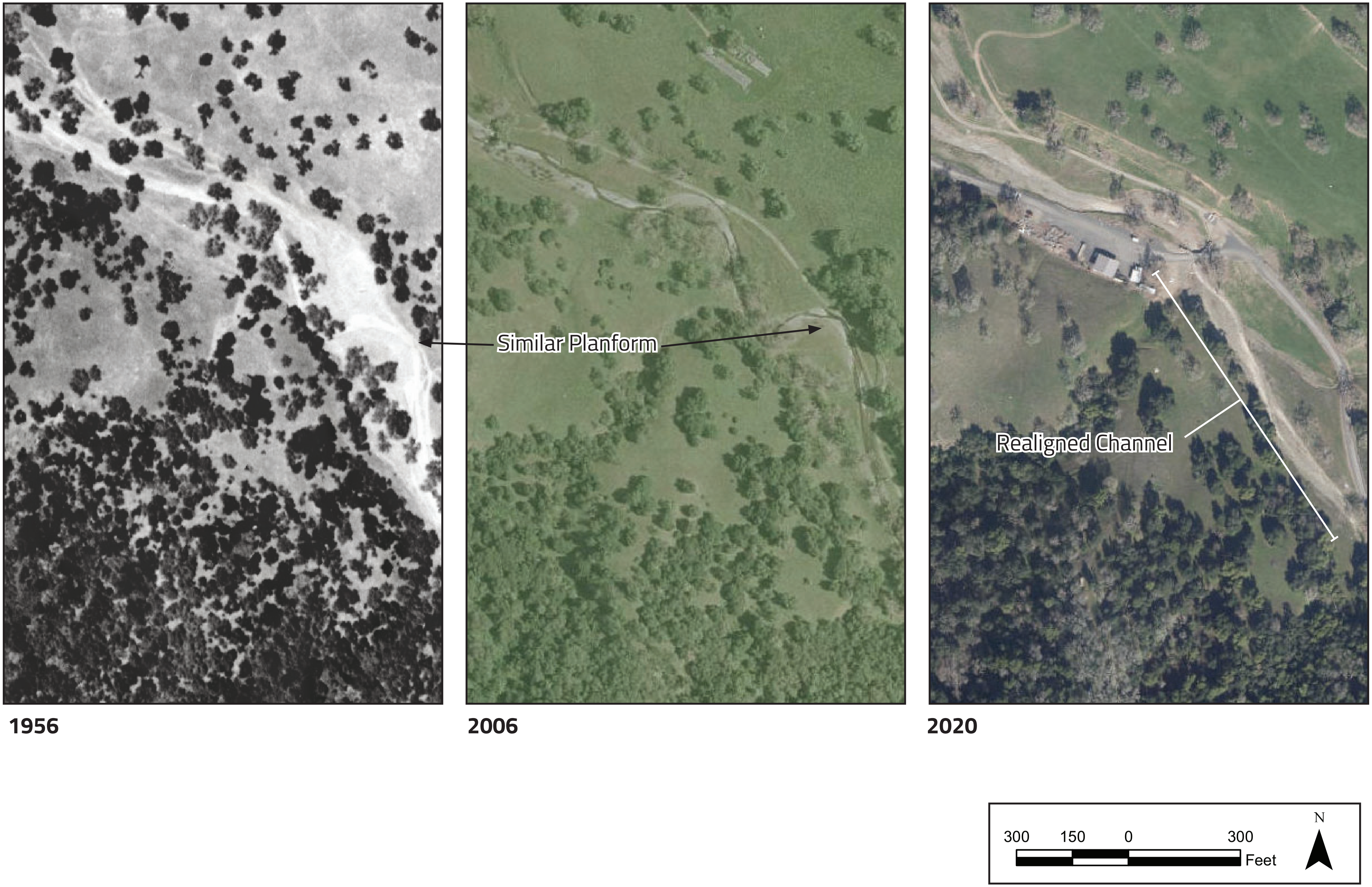


FIGURE 9. 1956-2020 Time Series of Creek Restoration and Bridge Replacement Reach

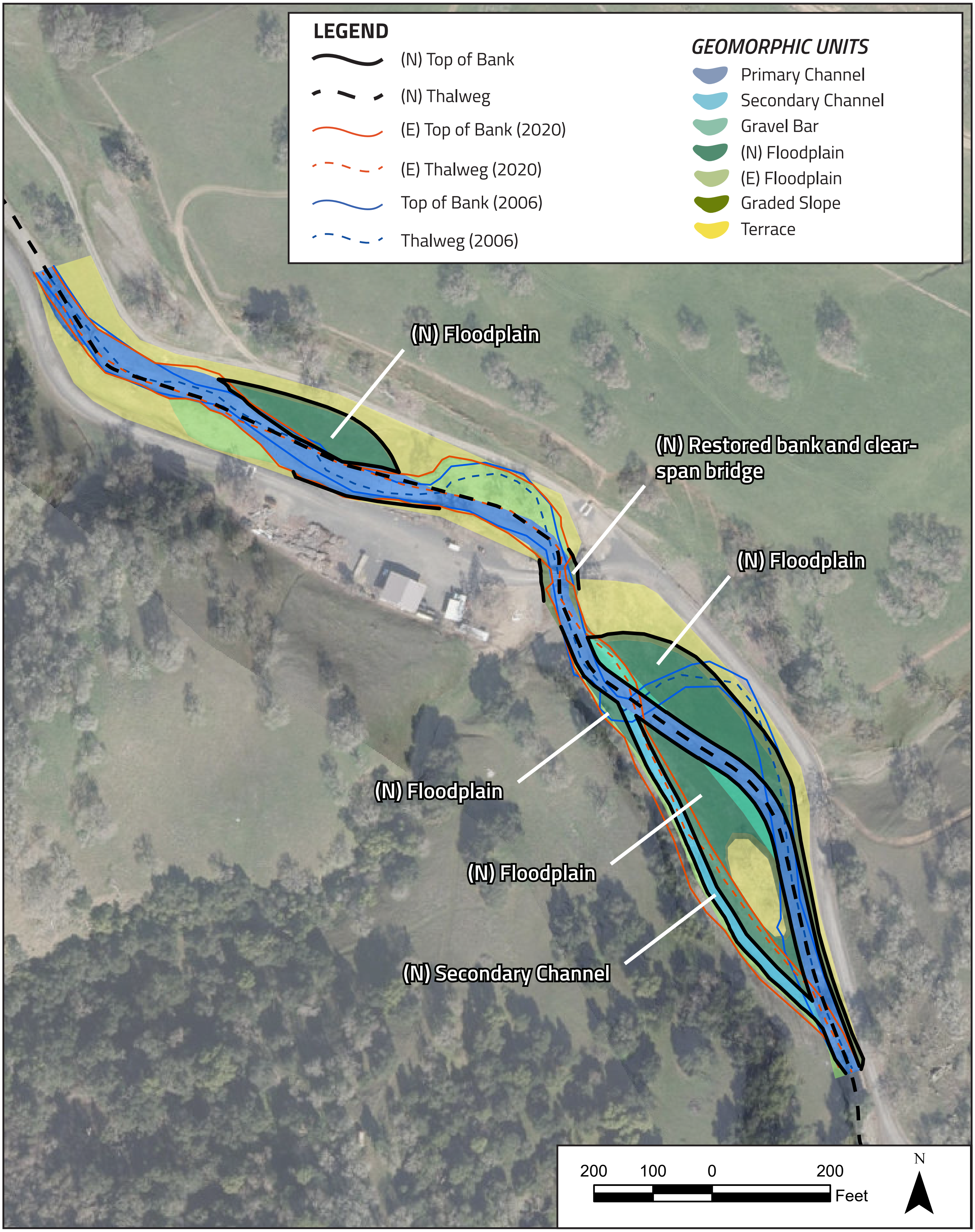


FIGURE 8. Pre-violation (2006), Existing Conditions (2020), and Proposed Top of Banks ,Thalweg Alignments, and Geomorphic Units. See Sheets R3-R7 for details.

FIGURE 11a. 2D Hydraulic Model Flow Depths in the Creek Restoration & Bridge Replacement Reach during 2-year flow

10-year flow (745 cfs)

200 100 0 200 Feet

N

FIGURE 11b. 2D Hydraulic Model Flow Depths in the Creek Restoration & Bridge Replacement Reach during 10-year flow

100-year flow (1,995 cfs)

FIGURE 11c. 2D Hydraulic Model Flow Depths in the Creek Restoration & Bridge Replacement Reach during 100-year flow

Attachments

Attachment A. Harper Canyon Creek Restoration and Reservoir Spillway Channel Erosion Protection Preliminary Design Plans. August 2021.

Appendices

Appendix A-1. Hydrologic Modeling Tech Memo #1

Appendix A-2. Roadway Culvert Capacity Evaluation Tech Memo #2

Appendix A-3. Hydraulic Modeling Tech Memo #3

Attachment A

Harper Canyon Creek Restoration and Reservoir Spillway Channel Erosion Protection Preliminary Design Plans. August 2021.

Appendix A-1

Hydrologic Modeling Tech Memo #1

geomorphDESIGN

fluvial geomorphology
landscape architecture
stream restoration

bioengineering
hydrology
design

TECH MEMO #1

REGISTERED PROFESSIONAL ENGINEER
MATT SMELTZER
C71671
Exp. 12/31/21
CIVIL
STATE OF CALIFORNIA

TO: Scott Walls, Walls Land + Water

FROM: Matt Smeltzer, P.E.
Guoyuan Li, Ph.D., P.E., P.H.

DATE: August 9, 2021

SUBJECT: TM#1 - HEC-HMS Modeled Harper Canyon Watershed Hydrology

---

1. Introduction

The Harper Canyon watershed is located in a remote area with little development (Figure 1). The total drainage area of the watershed is more than 4,000 acres. The land is mostly covered by forests and grasses, and soils are mostly permeable.

The Santa Clara County Drainage Manual (SCCDM) requires the unit hydrograph method for drainage areas larger than 200 acres.[1] The unit hydrograph method was required to use for this hydrology analysis.

The SCCDM manual lists several available hydrologic modeling software packages available that include the unit hydrograph method. The manual gives the highest recommendation to the HEC-HMS hydrology model developed by Army Corps of Engineers. HEC-HMS simulates the hydrologic process of rainfall, runoff, and stream flow routing. In flood control planning, HEC-HMS is often used to develop the streamflow hydrographs for the design flood events and, in particular, to estimate the magnitude of the peak stream flow discharges for the design events (e.g., 100-year peak flow).

HEC-HMS was selected for computing the peak stream flow discharges at key locations in the Harper Canyon watershed. The HEC-HMS model development, model parameterization, and modeling results are documented below.

[1] The Santa Clara County Drainage Manual also requires the unit hydrograph method for drainage areas between 50 acres and 200 acres, if there are large areas of impervious soils or substantial surface storage (lakes and reservoirs).

Northern California 2100 Fourth Street, No. 154, San Rafael, CA 94901
Interior Alaska 129 Kristin Drive, Fairbanks, AK 99712
mobile/office 510-219-1064
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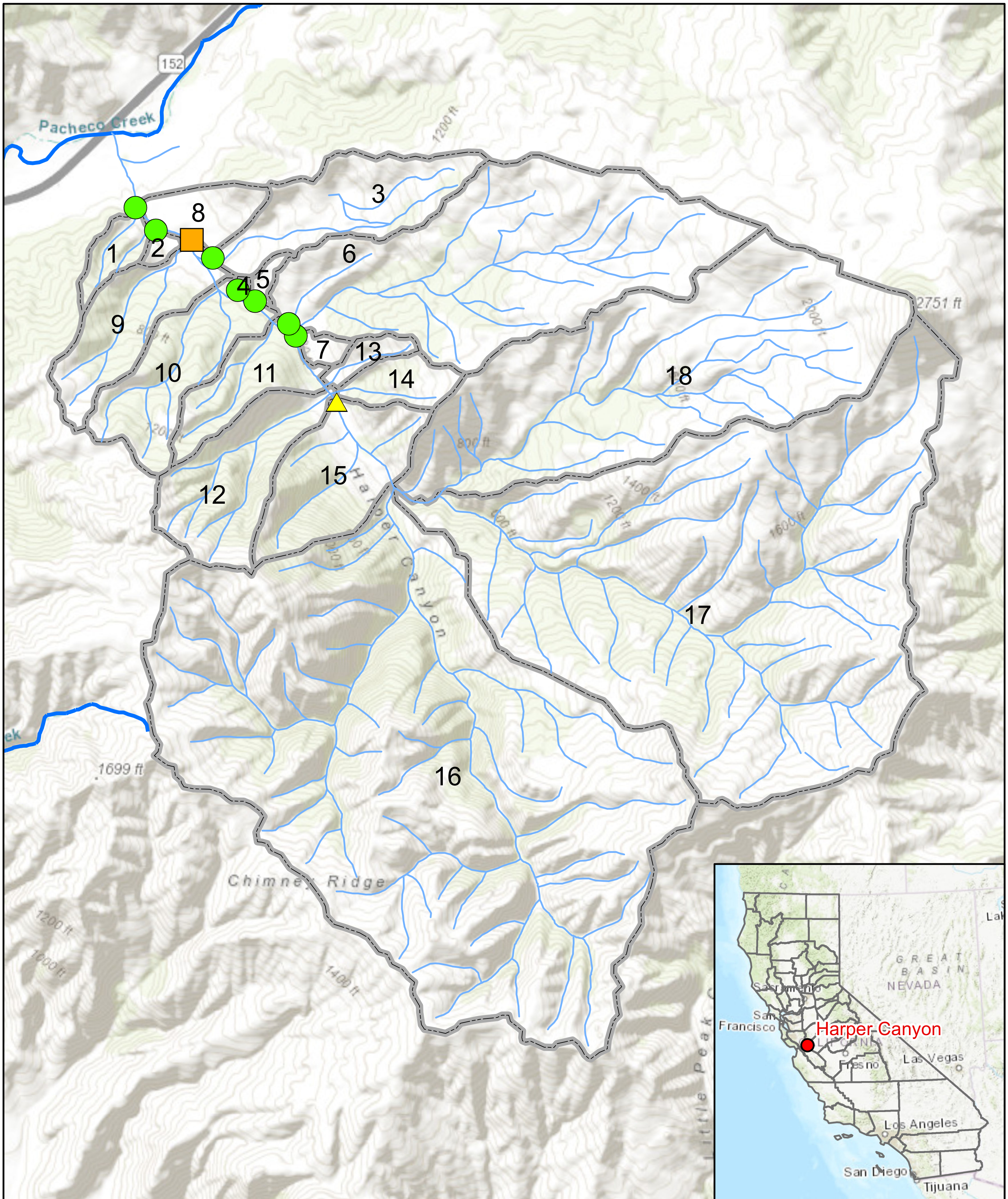
FIGURE 1



PROJECT LOCATION
AND SUB-BASIN DELINEATION

Legend

- Culvert
- Bridge
- Impoundment spillway
- Sub-basin & ID

2. HEC-HMS Model Development

The HEC-HMS model development involved the derivation of the following major parameters:

- Sub-basin characteristics
  - Sub-basin boundary
  - Longest flow path length
  - Average slope
  - Curve Number
  - Lag time
  - Baseflow
  - The reservoir elevation-storage-discharge curves
  - The reservoir spillway outflow rating curve

- Channel characteristics
  - Typical XS shapefile
  - Channel longitudinal slope
  - Channel roughness

- Design rainfall hyetographs
  - Hyetograph distribution pattern
  - Rainfall amounts for selected events

2.1 Sub-Basin Characteristics

The number of sub-basins is based on watershed characteristics and the key locations (i.e., the culvert and bridge locations and the spillway of the lake) where design flow data are required. The topography data in Digital Elevation Model (DEM) format derived from the 2020 LiDAR was used for the sub-basin delineation. A total of 15 sub-basins were delineated. The sub-basin boundaries and numeric IDs are shown in Figure 1.

The sub-basin longest flow path was determined based on the examination of the 2020 LiDAR DEM and the stream flow lines from the USGS National Hydrography Dataset (NHD).

The sub-basin average slope was calculated based on the 2020 LiDAR DEM. ArcMap was used for the slope calculations. In ArcMap, the DEM raster was first converted to a slope raster, and then a zonal statistics analysis was performed to derive the average slope within each of the sub-basins.

The runoff Curve Number (CN), developed by the USDA Natural Resources Conservation Service (NRCS), is an empirical parameter used in hydrology for predicting infiltration and direct runoff from rainfall excess. The CN is related to the hydrologic soil groups (HSGs) as defined by NRCS and the land cover as shown by the aerial photo.

NRCS has divided soils into four hydrologic soil groups (HSG), denoted as A, B, C, and D, where ranging from A to D the soil infiltration rate gets lower and the potential of surface runoff gets higher. The HSGs for the Harper Canyon watershed were obtained from USDA Web Soil Survey (WSS) [2] in the shapefile polygon format, as shown in Figure 2.

Within each HSG polygon, the land cover was examined in the aerial photo to determine it to be either mixed forest or grass land, and the hydrology condition was evaluated as good condition. The corresponding CNs from *Table E-1: Curve Numbers for AMC II* in the SCCDM was then assigned to each HSG polygon, and then the recommended modifications to the AMC II CNs from *Table E-2: Conversion of AMC II Curve Numbers to Other AMC Values* in the SCCDM were selected for the corresponding storm events. The HSG polygons were then intersected with the sub-basin boundaries polygon in ArcMap to derive the composite CNs for each of sub-basins.

The basin lag time is the time duration from the center of the excess rainfall to the peaking time of the hydrograph at the basin outlet. The SCS lag equation was used for the estimation of lag time. The equation is below[3]:

$$T_{lag} = \frac{L^{0.8}(S+1)^{0.7}}{1900(\%Slope)^{0.5}}$$

where :

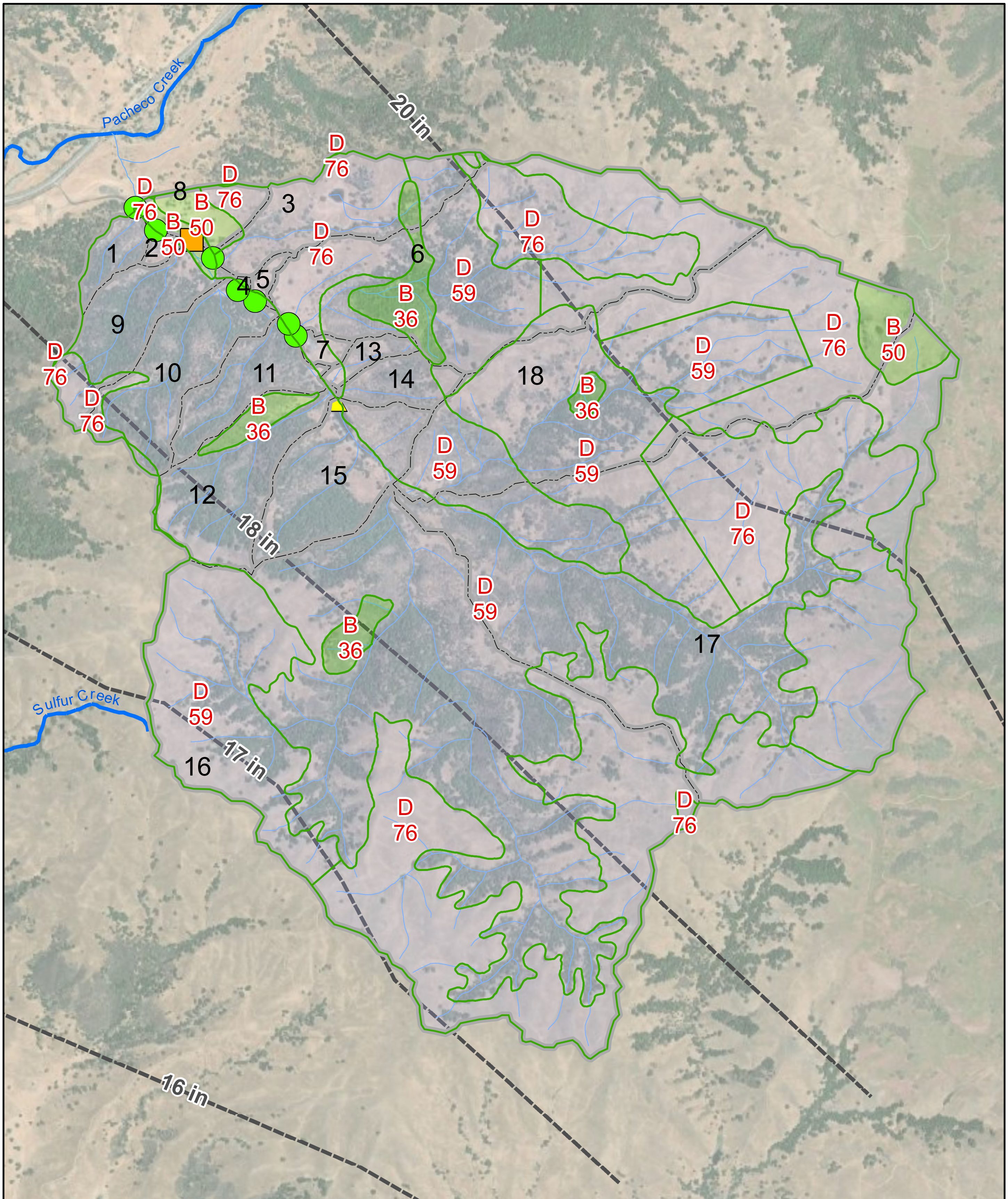
T_{lag} = Lag time in hours
L = Length of the longest drainage path in feet
S = (1000/CN) - 10, where: CN = Curve Number
%Slope = The average watershed slope in %

The baseflow is considered very minor for the study area. For the purpose of this study, the 1 cfs per square mile baseflow was assumed, which is about 5% of the simulated 2-year flow. The derived sub-basin parameters needed for HMS modeling are summarized in Table 1.

The existing reservoir on the mainstem stream has a small routing effect on the flow hydrographs (i.e., the reservoir reduces the mainstem peak flow downstream by a small percentage). To account for the routing effect, the Elevation-Storage (E-S) curve and spillway outflow rating curve were developed for reservoir routing calculations. We understand that

[2] https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm
[3] https://www.nohrsc.noaa.gov/technology/gis/uhg_manual.html

FIGURE 2



HYDROLOGIC SOIL GROUP
LAND COVER
AMC II CURVE NUMBER
AND ISOHYETAL CONTOUR

Legend

Culvert

Bridge

Sub-basins & ID

Impoundment spillway

HSG and AMC II CN

SCCDM Isohyetal

N

0 2,000 4,000 Feet

there is presently no available bathymetric survey for the reservoir. However, since the reservoir level is not managed, the reservoir may be assumed to be full to the spillway crest elevation at the beginning of storm flow simulation. Therefore, the reservoir provides storage above the spillway crest elevation - between the 432.6-foot (NAVD88) spillway crest elevation and the 441-foot (NAVD88) dam crest elevation. The elevation contours between the spillway and dam crests contained in the 2020 LiDAR DEM, combined with a simple HEC-RAS 2D model-derived spillway rating curve (Figure 3), were used to develop the available storage pond E-S curve (Figure 3).

Table 1
Hydrologic Parameters for HMS Model

#	NAME	Drainage Area		Longest Flow Path	Average Slope	Composite CN for Different Storm Events			Lag Time for Different Flow Events (min)		
		(acres)	(mi^2)	(ft)	(%)	2-yr	5-yr	10-yr +	2-yr	5-yr	10-yr +
1	Culvert1	24.8	0.039	1,460	52	63.5	65.8	68.0	5.66	5.33	5.03
2	Culvert2	7.2	0.011	700	31	62.3	64.6	66.9	4.21	3.97	3.74
3	Culvert3	121.5	0.190	4,730	34	74.6	76.6	78.4	13.29	12.57	11.90
4	Culvert4	1.7	0.003	220	33	72.0	74.1	76.0	1.25	1.18	1.11
5	Culvert5	7.7	0.012	910	28	76.6	78.5	80.3	3.69	3.48	3.30
6	Culvert6	427.1	0.667	8,020	34	71.4	73.4	75.3	22.25	21.04	19.92
7	Culvert7	10.3	0.016	1,010	31	74.2	76.1	78.0	4.11	3.89	3.68
8	D/S Bridge	41.8	0.065	2,040	23	62.9	65.2	67.4	11.41	10.76	10.15
9	U/S Bridge1	94.1	0.147	3,480	43	66.5	68.7	70.8	11.59	10.93	10.33
10	U/S Bridge2	108.1	0.169	3,540	47	65.2	67.4	69.6	11.58	10.92	10.31
11	U/S Bridge3	65.5	0.102	2,830	50	63.9	66.2	68.4	9.69	9.13	8.62
12	U/S Bridge4	119.2	0.186	3,890	58	60.5	62.8	65.0	12.72	11.99	11.32
13	U/S Bridge5	16.0	0.025	1,700	50	66.2	68.4	70.5	6.10	5.75	5.44
14	U/S Bridge6	35.1	0.055	1,950	52	63.8	66.0	68.2	7.11	6.70	6.32
15	U/S Pond1	119.7	0.187	2,400	51	63.5	65.8	68.0	8.48	7.99	7.54
16	U/S Pond2	1,278.4	1.997	11,360	45	68.4	70.6	72.6	27.68	26.13	24.71
17	U/S Pond3	1,018.8	1.592	14,130	37	71.7	73.7	75.6	33.35	31.51	29.80
18	U/S Pond4	518.4	0.810	9,770	36	68.0	70.1	72.2	27.63	26.08	24.65

2.2 Channel Characteristics

The HEC-HMS model routes the simulated flow hydrographs at the basin outlet to downstream based on user selected routing method. The Muskingum-Cunge routing method was selected, as it considers both the conservation of mass and the conservation of momentum. The required parameters include the channel geometry such as the reach length, slope, and typical XS profile, and the channel roughness or Manning's n. The channel lengths and XS profiles were derived from the 2020 LiDAR DEM, and the roughness was estimated to be 0.035 for main channel and 0.1 for floodplain.

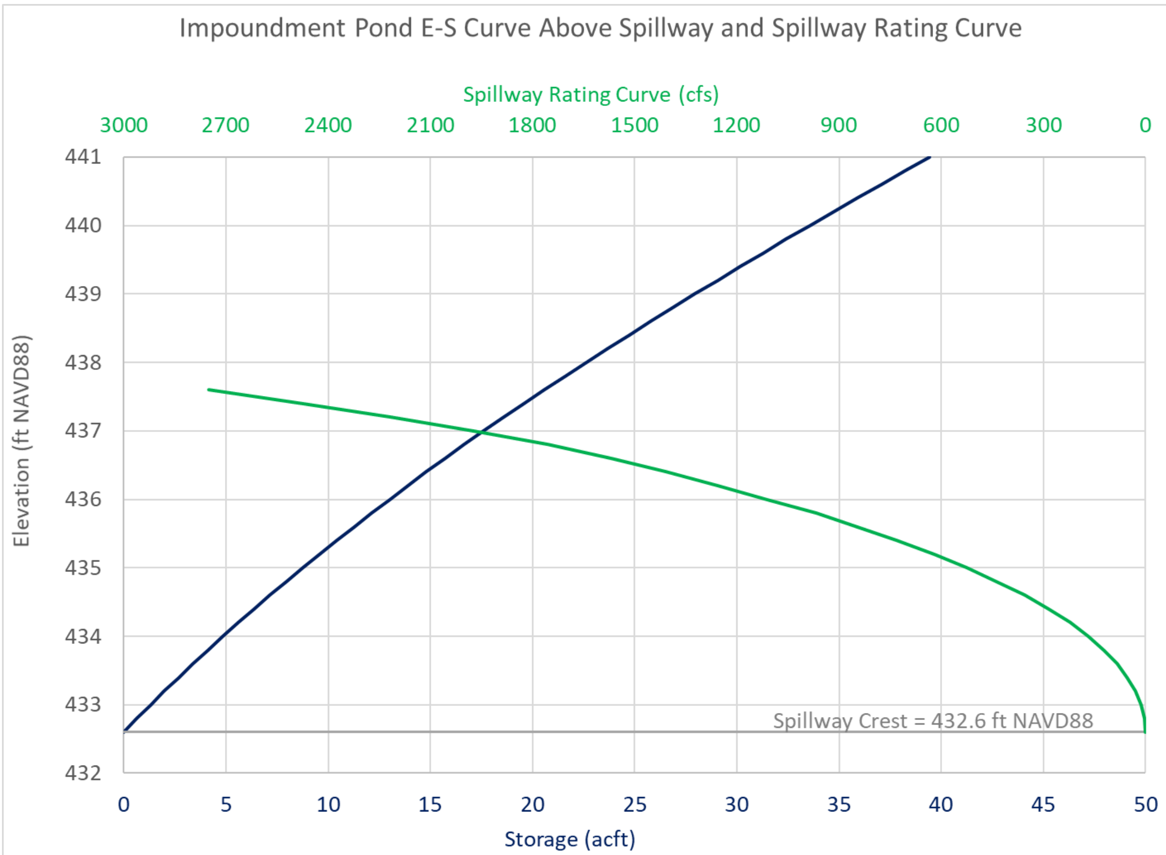


Figure 3 Reservoir Elevation-Storage Curve and Spillway Rating Curve

2.3 Design Rainfall Hyetographs for Rainfall Excess Calculation

The SCCDM adopted the normalized 24-hour 5-min interval rainfall distribution pattern based on the three-day December 1955 rainfall event. The adopted pattern has been adjusted to preserve the local rainfall statistics so that the 24-hour storm distribution may be used even where shorter duration storms are more critical. The SCCDM also provided the procedures and formulae for estimating the 24-hour total rainfall amount for different storm return intervals from mean annual precipitation map provided in the SCCDM. The normalized 24-hour rainfall pattern multiplied by the corresponding 24-hour rainfall amount derives the corresponding design 24-hour hyetographs at different return intervals.

Based on the SCCDM, the 24-hour total rainfall amount is a function of the mean annual precipitation. The mean annual precipitation isohyetal lines from the SCCDM were georeferenced and digitized as shown in Figure 2. For the purpose of HEC-HMS modeling, it was assumed the precipitation is uniform across the entire Harper Canyon watershed. Using the zonal statistics analysis, the average annual precipitation for the watershed from the SCCDM is 18.8 inches. The 24-hour rainfall amount for different return intervals were then estimated from the average annual precipitation using the following formula (excerpted from SCCDM):

$$x_{T,D} = A_{T,D} + \left(B_{T,D} MAP\right)$$

Where: $x_{T,D}$ = precipitation depth for a specific return period and storm duration (inches)

T = return period (years)

D = storm duration (hours)

$A_{T,D}$, $B_{T,D}$ = coefficients from Tables B-1 and -2 (dimensionless)

MAP = Mean Annual Precipitation (inches)

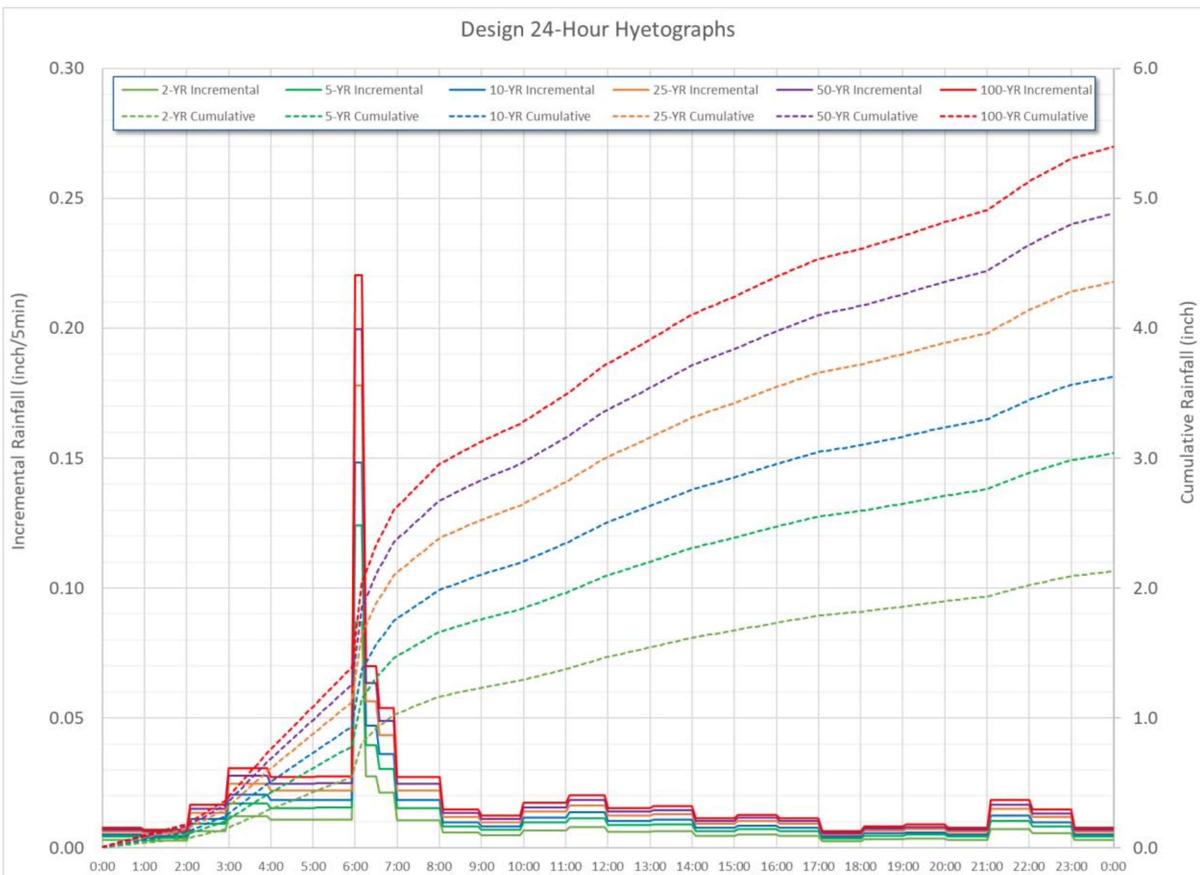


Figure 4 Design 24-hour hyetographs for the HEC-HMS model

The estimated 24-hr precipitation amounts for 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year events are shown in Table 2, and the derived corresponding hyetographs are shown in Figure 4.

Table 2
Estimated 24-hr Total Precipitation

Return Period	Duration	AT,D	BT,D	24-hr Total Rainfall (in)
2-YR RETURN PERIOD	24-hr	0.314185	0.096343	2.13
5-YR RETURN PERIOD	24-hr	0.474528	0.136056	3.03
10-YR RETURN PERIOD	24-hr	0.567017	0.16255	3.62
25-YR RETURN PERIOD	24-hr	0.675008	0.195496	4.35
50-YR RETURN PERIOD	24-hr	0.747121	0.219673	4.88
100-YR RETURN PERIOD	24-hr	0.814046	0.243391	5.39

With all the above parameters, the HEC-HMS model was developed with a configuration shown in Figure 5.

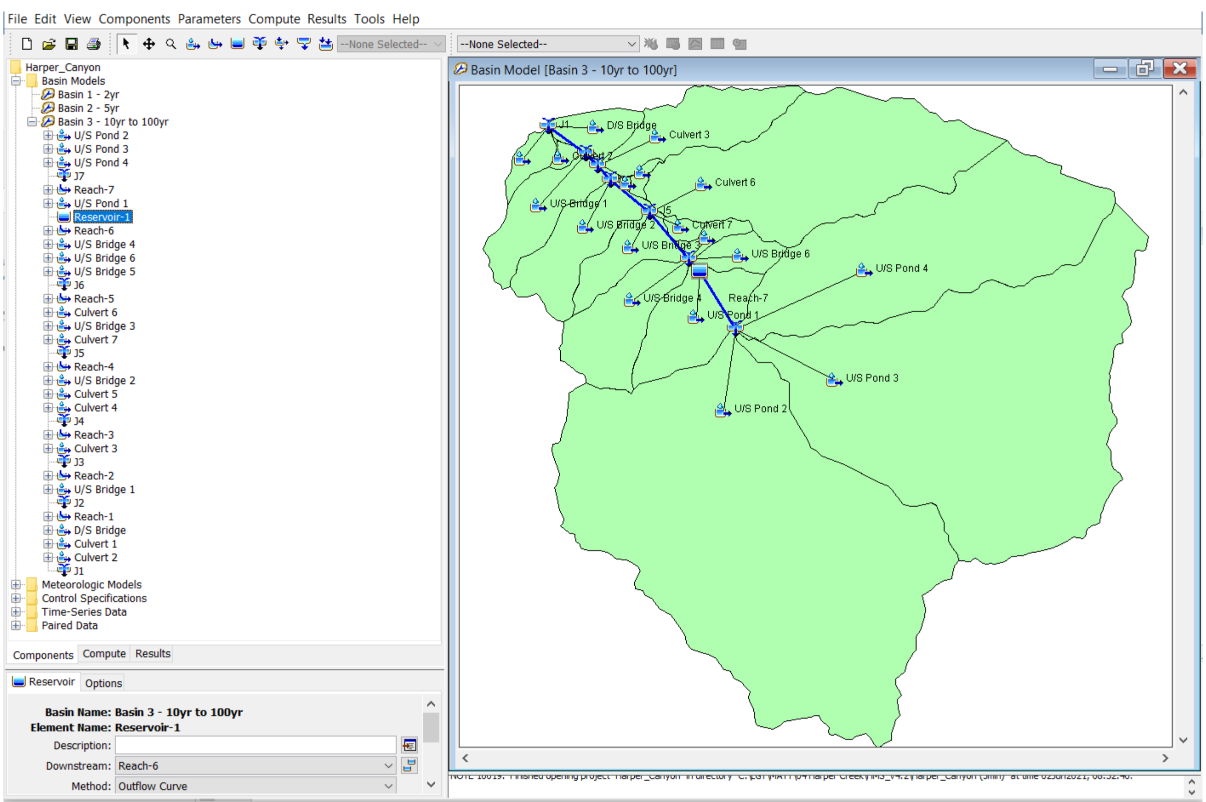


Figure 5 HEC-HMS model configuration

3. HEC-HMS Modeling Results

The HEC-HMS model was run with all the 6 design hyetographs described above. The model simulated peak discharges at key locations are shown in Table 3 and Figure 6. These values are the design peak discharges determined by methods required by the SCCDM for the evaluation and design of existing and new hydraulic structures within the Harper Canyon watershed.

Table 3
Harper Canyon Watershed
HEC-HMS Simulated Peak Flow at Selected Locations

#	Location	Drainage Area (mi^2)	HEC-HMS Simulated Peak Flow (cfs)					
			2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
1	Culvert 1	0.039	0.6	2.6	4.9	10.0	14.1	18.5
2	Culvert 2	0.011	0.1	0.6	1.3	2.7	3.9	5.2
3	Culvert 3	0.190	5.3	28.5	56	85	106	128
4	Culvert 4	0.003	0.1	0.5	1.0	1.5	1.9	2.3
5	Culvert 5	0.012	0.7	3.3	5.7	8.2	10.1	12.0
6	Culvert 6	0.667	15.5	52	118	187	242	298
7	Culvert 7	0.016	0.4	3.3	6.4	9.5	11.9	14.3
8	At Basin Outlet	6.274	116	318	737	1,205	1,585	1,976
9	At Bridge	6.159	117	320	745	1,217	1,600	1,995
10	Reservoir Inflow	4.586	92	257	592	965	1,264	1,573
11	Reservoir Outflow	4.586	88	249	568	938	1,235	1,541

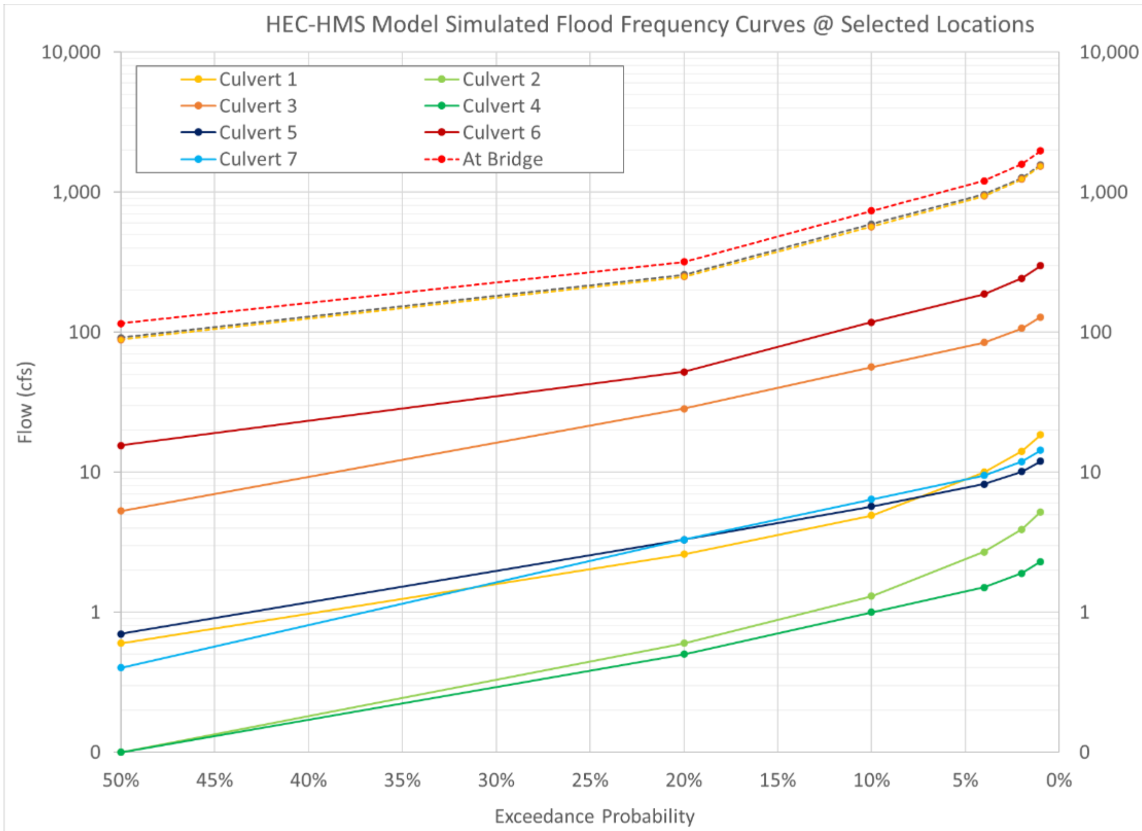


Figure 6 HEC-HMS model simulated flood frequency curves at selected locations

Appendix A-2

Roadway Culvert Capacity Evaluation Tech Memo #2

TECH MEMO #2

TO: Scott Walls, Walls Land + Water

FROM: Matt Smeltzer, P.E.
Guoyuan Li, Ph.D., P.E., P.H.

DATE: August 9, 2021

SUBJECT: TM#2 - Harper Canyon Roadway Culvert Capacity Evaluation

---

1. Introduction

TM#1 computed the peak stream flow discharges at key locations in the Harper Canyon watershed including the inlets to seven existing culverts running under the gravel roadway. This tech memo #2 (TM#2) compares the computed peak flow discharges at the culvert inlet locations to the estimated flow capacities of the existing culverts.

Santa Clara County requires the storm drainage system be adequately sized to convey the 10-year design storm within drainage facilities (underground pipes and open channels) and also to safely convey the 100-year design storm without contributing to upstream or downstream flooding conditions.

Flow through a culvert is either unlet inlet control or outlet control. Inlet control occurs when flow exiting the culvert flows freely (free outfall conditions) without affecting flow inside the culvert barrel. Under inlet control conditions (free outfall conditions), only the inlet area, the inlet configuration, and the inlet shape determine the amount of flow through the culvert for a given headwater elevation (i.e., depth of flow at the culvert inlet). Therefore, a modified form of the orifice equation may be used to calculate the capacity of a culvert under inlet control. The headwater elevation is calculated with respect to the inlet invert.

Outlet control conditions occur when flow existing the culvert flows into a reservoir, another stream, or another hydraulic structure that affects the hydraulics of flow inside the culvert barrel. Under outlet control conditions, the culvert flow capacity is determined by a combination of tailwater elevation (depth of flow at the culvert outlet), barrel conditions (pipe length, slope, and roughness), and inlet geometry. The headwater elevation is calculated with respect to the outlet invert. The energy equation and Manning's equation are used to calculate the capacity of a culvert flowing under outlet control.

The SCCDM suggests to use the nomographs developed by the Federal Highway Administration (FHWA) to determine the culvert headwater depths at given flows for both inlet control and outlet control culverts.

The FHWA nomograph method was used to determine the culvert capacities, assuming the culverts are under inlet control condition (free outfall condition specified by SCCDM). Per the SCCDM minimum design criteria cited above, the culvert flow capacities were estimated for two different headwater depth conditions:

> (1) When the headwater surface is at the top of culvert (H/D =1) to determine if the culvert can pass the Q10yr flood under that condition;
>
> (2) When the headwater is at the maximum headwater without overtopping (i.e., headwater surface at the roadway surface) to determine the maximum culvert flow capacity without overtopping of the road surface[1].

After the culvert capacities were determined for these two conditions, the corresponding peak flow return intervals were estimated based on the corresponding flood frequency curves developed using the HEC-HMS model (TM #1). which was then used to tell if the culverts meet the design capacity requirements.

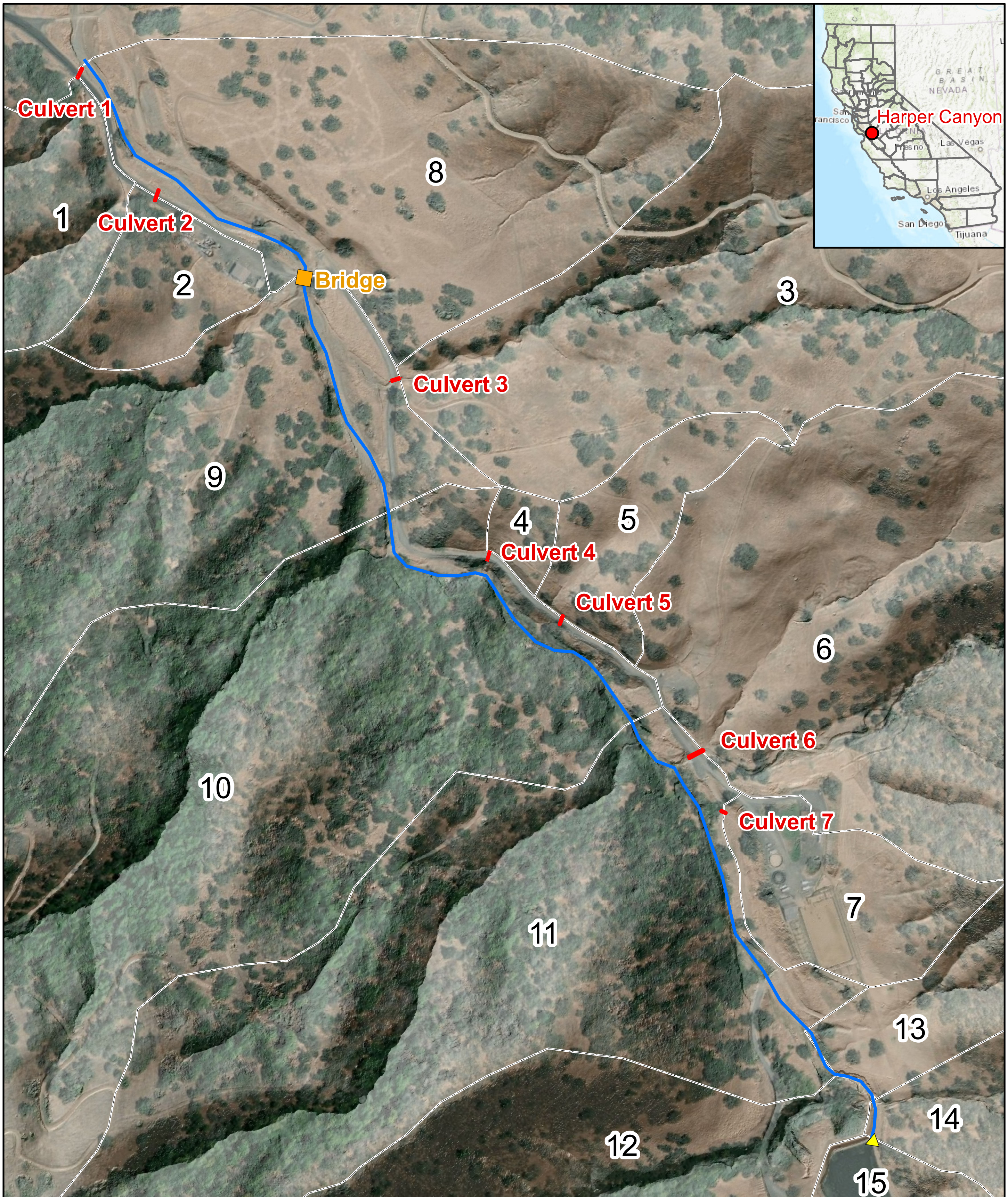
2. Culvert Inspection and Elevations Survey

In April 2021, inspections were made of the culvert inlet and outlet conditions and maximum headwater elevations and elevations were surveyed with a high accuracy RTK-GPS unit and/or Total Station theodolite in NAVD88 elevation datum.

The seven existing culverts were numbered from 1 to 7 from downstream to upstream (Figure 1). Culverts 1-6 were newer corrugated polyethylene pipes (CPP) with smooth interior. Culvert 7 is a possibly older cast iron pipe (CIP). Culvert 6 has two barrels, with identical size and material, but slightly different lengths. All of them have projecting inlet. Figures 2-3 show the typical culvert materials and culvert projecting inlet configurations.

[1] The evaluation of whether a culvert can safely pass the Q100yr flood will be checked after the pending mainstem stream channel hydraulic model runs are completed, in order to provide detailed tailwater conditions required to evaluate for potential outlet control conditions.

FIGURE 1



BOURDET-HARPER CANYON
CULVERT LOCATIONS

Legend

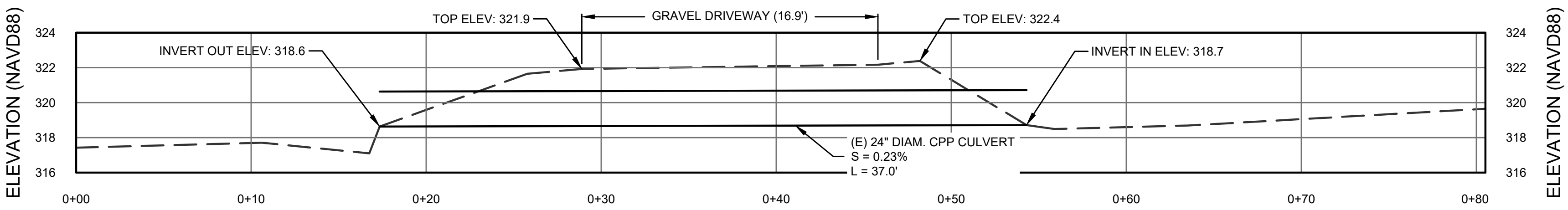
Culvert | Impoundment Spillway
Bridge | Sub-basin & ID

N

0 250 500 Feet

Figure 2 Upstream view of Culvert 6 (CPP, Projecting, Double-Barrel)

Figure 3 Upstream view of Culvert 7 (CIP, Projecting, Single-Barrel)



CULVERT 1 PROFILE
SCALE: 1" = 10'

CULVERT 1 EXISTING SITE PLAN
SCALE: 1" = 10'

LEGEND

400 401 EXISTING CONTOURS (TOWILL, JAN 2020)

EXISTING FENCE LINE

EXISTING EDGE OF TRAVELED WAY

0 5 10 20

SCALE: 1" = 10'

Harper Canyon
Pacheco Pass Highway
Santa Clara County, CA
A.P.N.s 898-19-005 & 043
Owner: Lacy Bourdet

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CULVERT 1 EXISTING PLAN AND PROFILE

HARPER CANYON

CREEK RESTORATION PLAN

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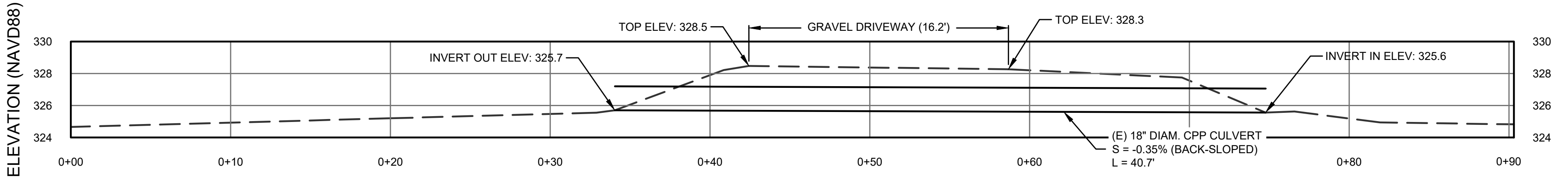
San Rafael, CA 94901
(510) 219-1064
www.geomorphdesign.com

geomorphDESIGN

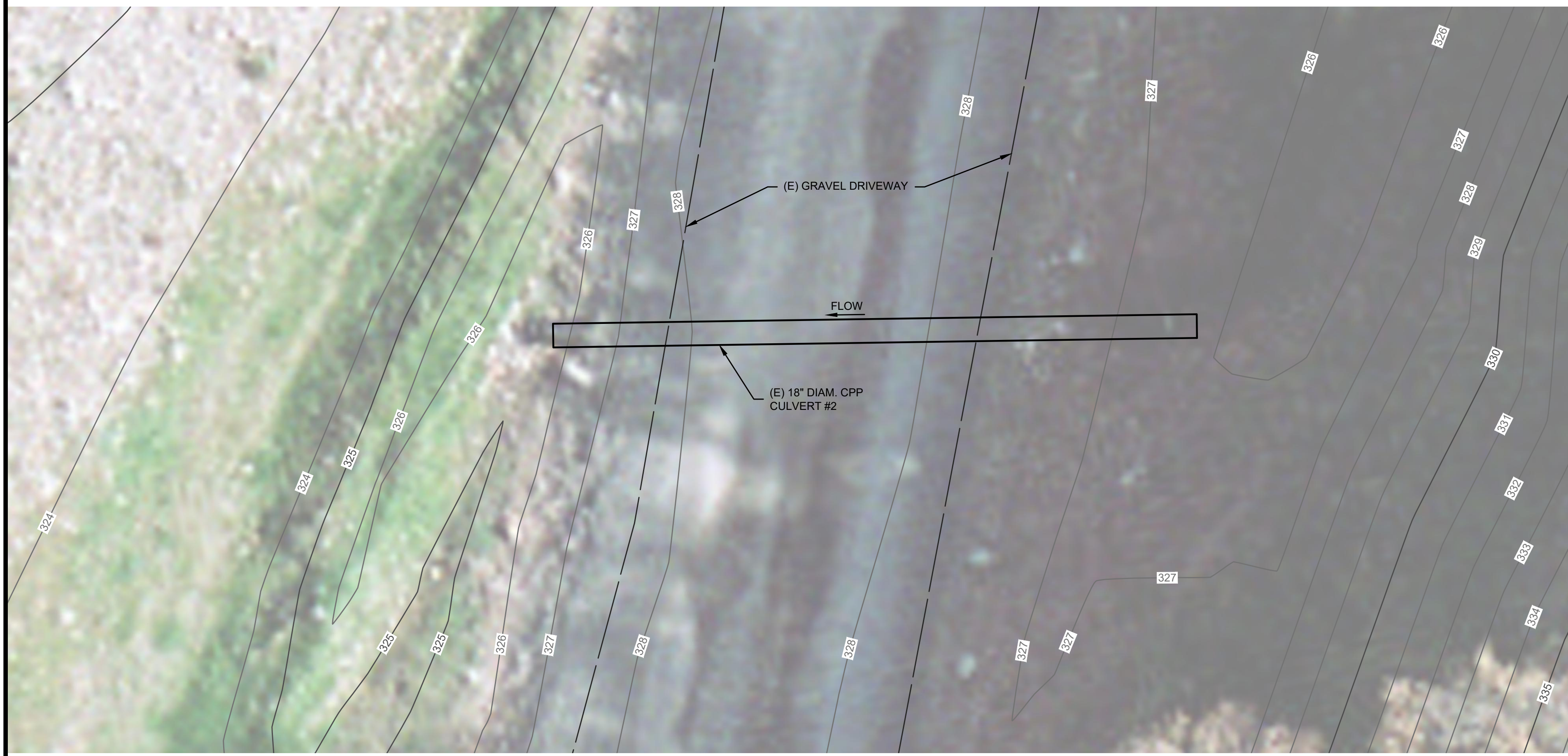
Date: 17 JUN 2021
Design by: GL
Drawn by: BRS
Checked by: GL
Scale: 1" = 10'

FIG.4



CULVERT 2 PROFILE

SCALE: 1" = 10'



CULVERT 2 EXISTING SITE PLAN

SCALE: 1" = 10'

LEGEND

400 / 401	EXISTING CONTOURS (TOWILL, JAN 2020)
	EXISTING FENCE LINE
	EXISTING EDGE OF TRAVELED WAY

0 5 10 20

SCALE: 1" = 10'

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CULVERT 2 EXISTING PLAN AND PROFILE

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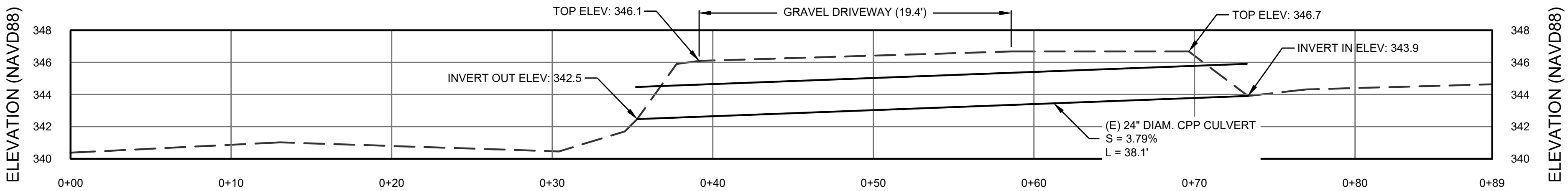
Scale: 1" = 10'

FIG.5



CULVERT 3 PROFILE
SCALE: 1" = 10'

CULVERT 3 EXISTING SITE PLAN
SCALE: 1" = 10'

LEGEND

400 401	EXISTING CONTOURS (TOWILL, JAN 2020)
	EXISTING FENCE LINE
	EXISTING EDGE OF TRAVELED WAY

0 5 10 20
SCALE: 1" = 10'

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CULVERT 3 EXISTING PLAN AND PROFILE

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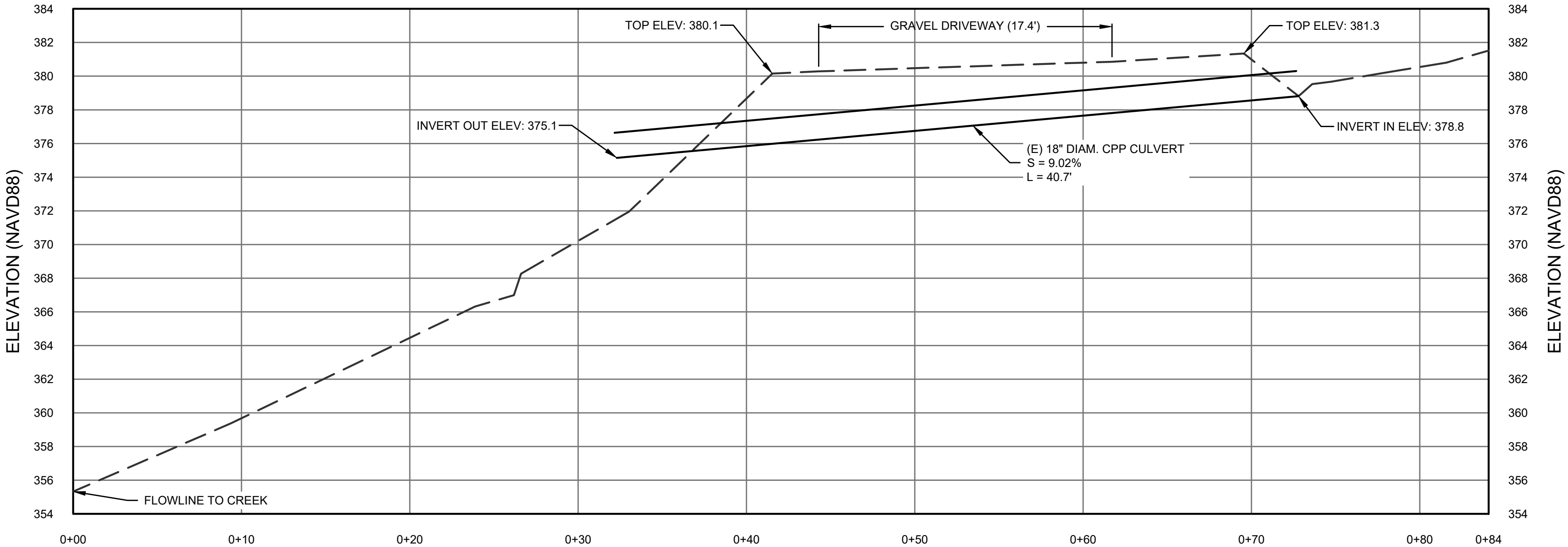
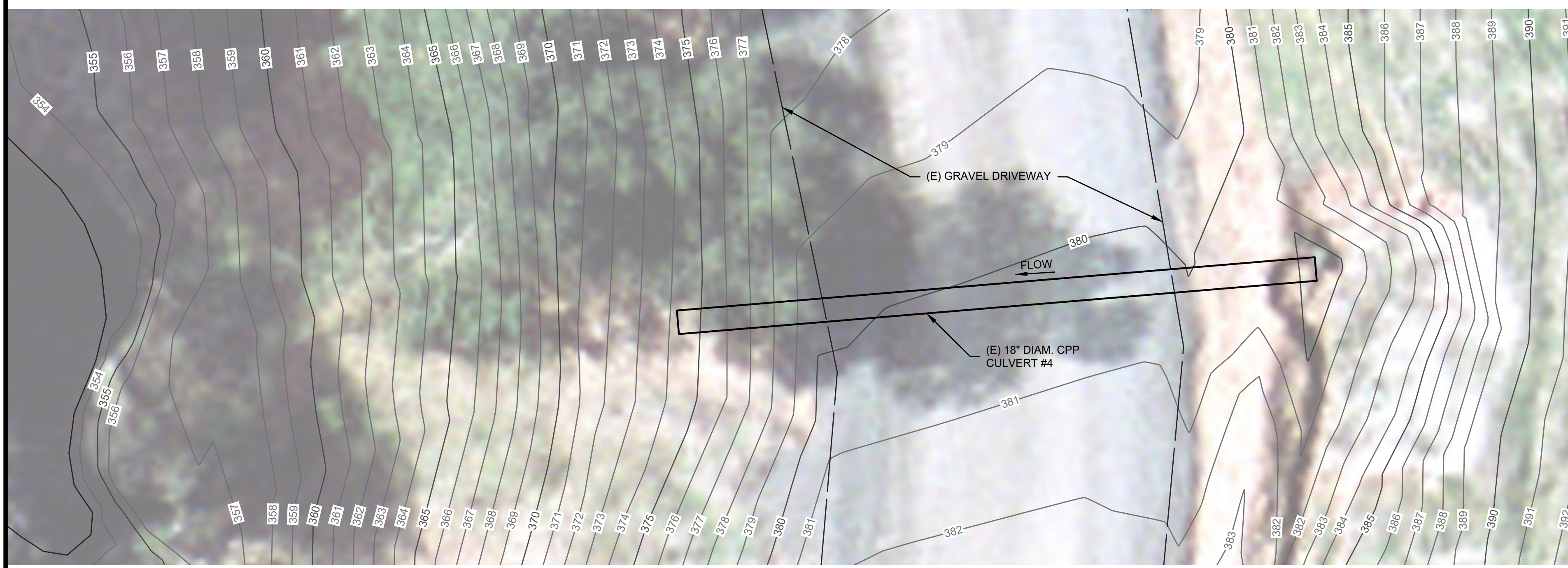
FIG.6



CULVERT 4 PROFILE

SCALE: 1" = 10'



CULVERT 4 EXISTING SITE PLAN

SCALE: 1" = 10'

LEGEND

400 — 401 EXISTING CONTOURS (TOWILL, JAN 2020)

EXISTING FENCE LINE

EXISTING EDGE OF TRAVELED WAY

0 5 10 20

SCALE: 1" = 10'

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CULVERT 4 EXISTING PLAN AND PROFILE

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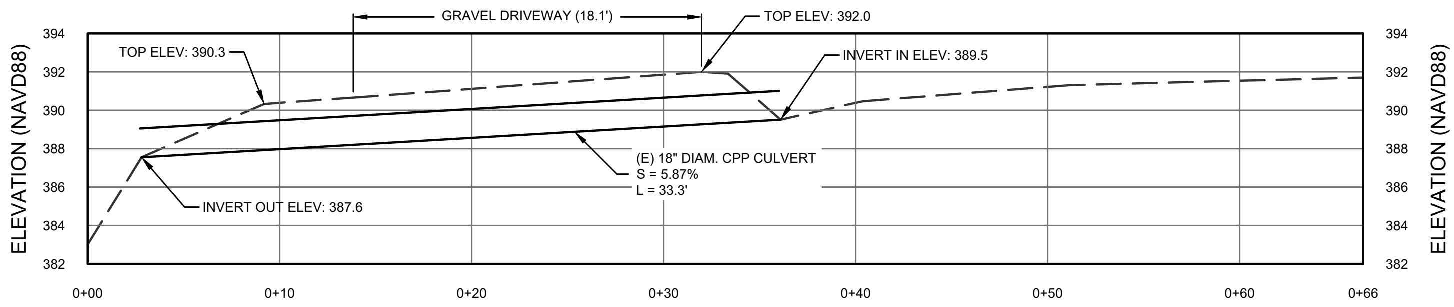
Scale: 1" = 10'

FIG.7



CULVERT 5 PROFILE
SCALE: 1" = 10'

CULVERT 5 EXISTING SITE PLAN
SCALE: 1" = 10'

LEGEND

400 / 401	EXISTING CONTOURS (TOWILL, JAN 2020)
	EXISTING FENCE LINE
	EXISTING EDGE OF TRAVELED WAY

0 5 10 20
SCALE: 1" = 10'

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CULVERT 5 EXISTING PLAN AND PROFILE

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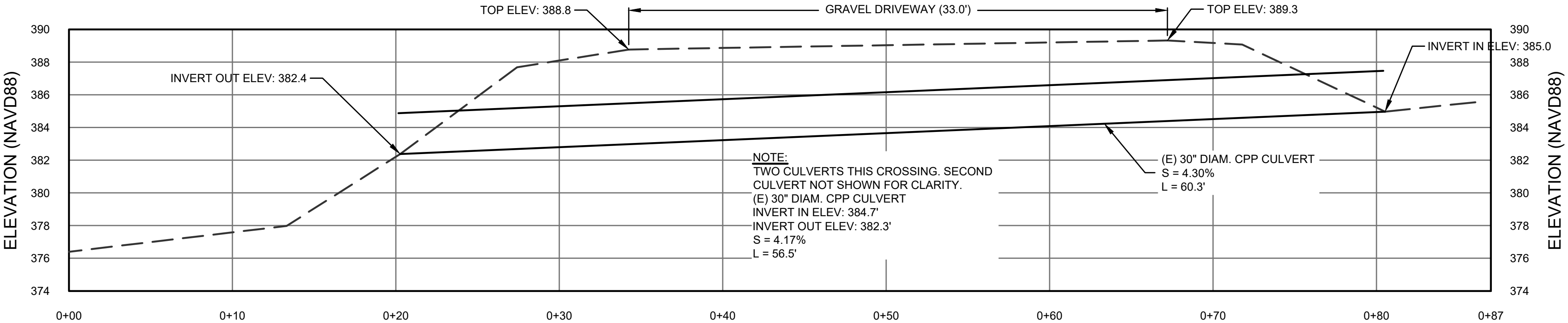
FIG.8



CULVERT 6 PROFILE

SCALE: 1" = 10'

CULVERT 6 EXISTING SITE PLAN

SCALE: 1" = 10'

LEGEND

EXISTING CONTOURS (TOWILL, JAN 2020)

EXISTING FENCE LINE

EXISTING EDGE OF TRAVELED WAY

0 5 10 20

SCALE: 1" = 10'

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CULVERT 6 EXISTING PLAN AND PROFILE

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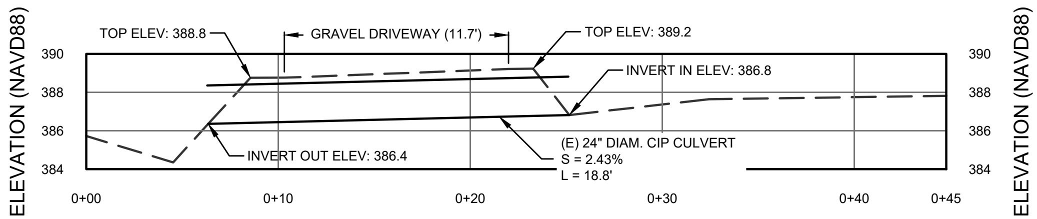
Scale: 1" = 10'

FIG.9



CULVERT 7 PROFILE

SCALE: 1" = 10'

CULVERT 7 EXISTING SITE PLAN

SCALE: 1" = 10'

LEGEND

400 401 EXISTING CONTOURS (TOWILL, JAN 2020)

EXISTING FENCE LINE

EXISTING EDGE OF TRAVELED WAY

0 5 10 20

SCALE: 1" = 10'

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Owner: Lacy Bourdet

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CULVERT 7 EXISTING PLAN AND PROFILE

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Scale: 1" = 10'

FIG.10

3. Existing Conditions Culvert Capacity Evaluation

The FHWA developed culvert flow nomographs for culverts under inlet control conditions based on the culvert material, shape, and the inlet configuration. Among these factors, the inlet configuration is a major factor in inlet control performance for a given culvert material, shape, and size. Typical inlet configurations are (1) headwall; (2) mitered to conforms to slope; and (3) projecting.

The materials of the culverts in Harper Canyon watershed are either CPP with a smooth interior or CIP. These culverts have an interior roughness around 0.012, similar to concrete pipes. For this reason, the FHWA nomograph Chart 1B was selected for the culvert capacity analysis. As all the culvert inlets in Harper Canyon watershed are configured as projecting, the scale (3) on Chart 1B was used.

Culvert flow capacities under inlet control for both headwater elevation conditions (headwater surface right at the top of culvert, and headwater surface at the maximum allowable headwater surface before overtopping the roadway) were determined using Chart 1B scale (3) as shown in Figures 11 and 12 respectively. The results are summarized in Table 1.

The individual culvert flow capacities were then compared to the flood frequency curves from TM#1 for the individual culvert inlet locations (Table 2) to determine the return interval of the capacity flows and compare them to SCCDM's 10-year, 25-year, and 100-year threshold design criteria (Table 3).

4. Results and Recommendations

1. Under existing conditions, culverts #3 and #6 do not pass the 10-year design flow under free outfall conditions when the headwater surface is at the top of the culvert inlet (H/W=1). Culverts #3, #5, and #6 do not pass the 25-year design flow under free outfall conditions when H/W=1. The number and/or size of the culverts should be increased at culvert site #3 and #6 for meeting both the 10-year and 100-year design criteria per Santa Clara County requirements.

2. The length of Culvert #3 should be extended to conform with the creek restoration grading plan in that vicinity.

3. Evaluate potential outlet flow conditions using pending results of the mainstem stream hydraulic model to ensure culverts 1 and 2 meet the 100-year design criteria under existing conditions. This evaluation will be discussed in TM#3 - Harper Canyon Watershed Hydraulic Modeling.

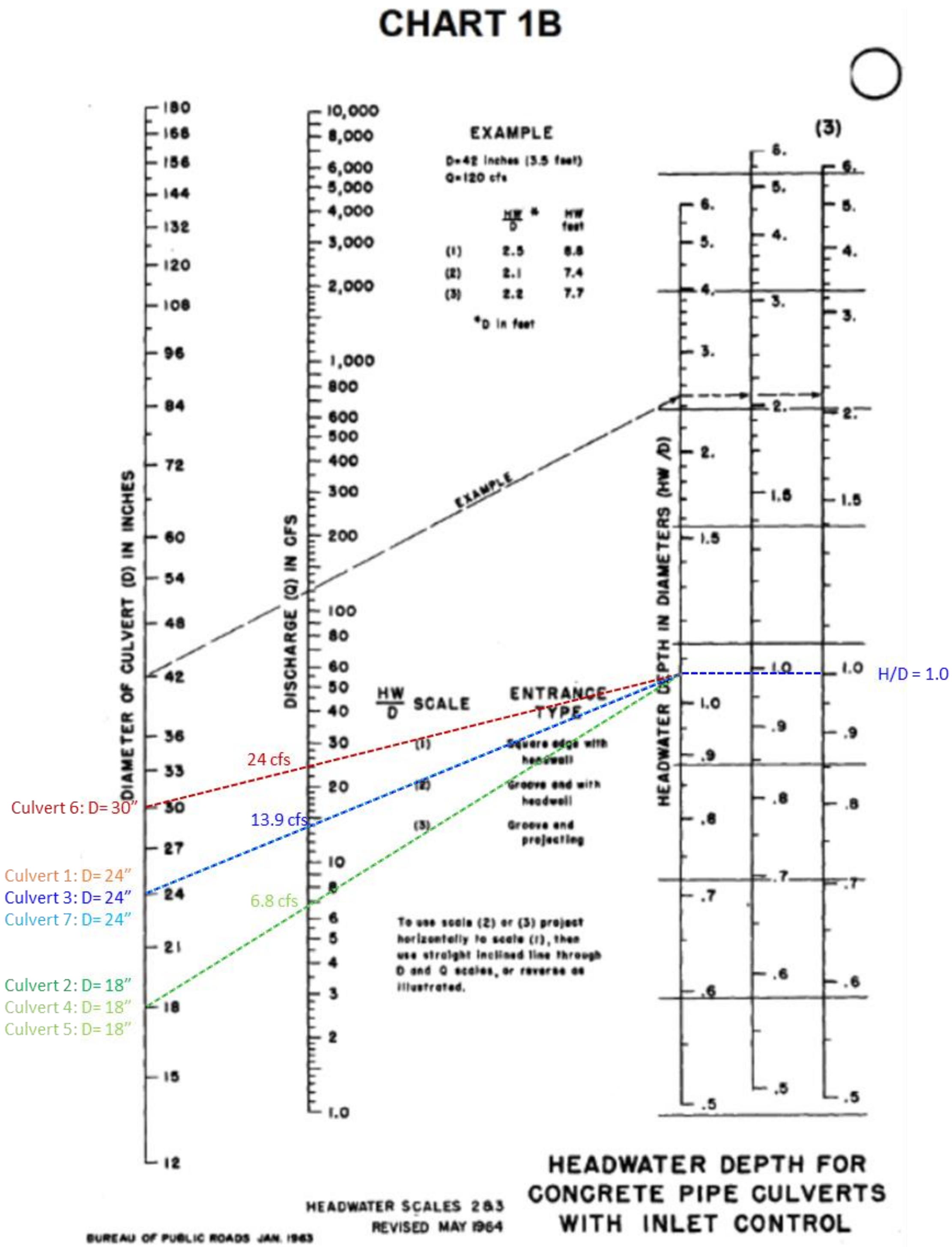

Figure 11 Culvert capacity determination with headwater at top of culvert (H/D = 1) using FHWA’s Chart 1B nomograph

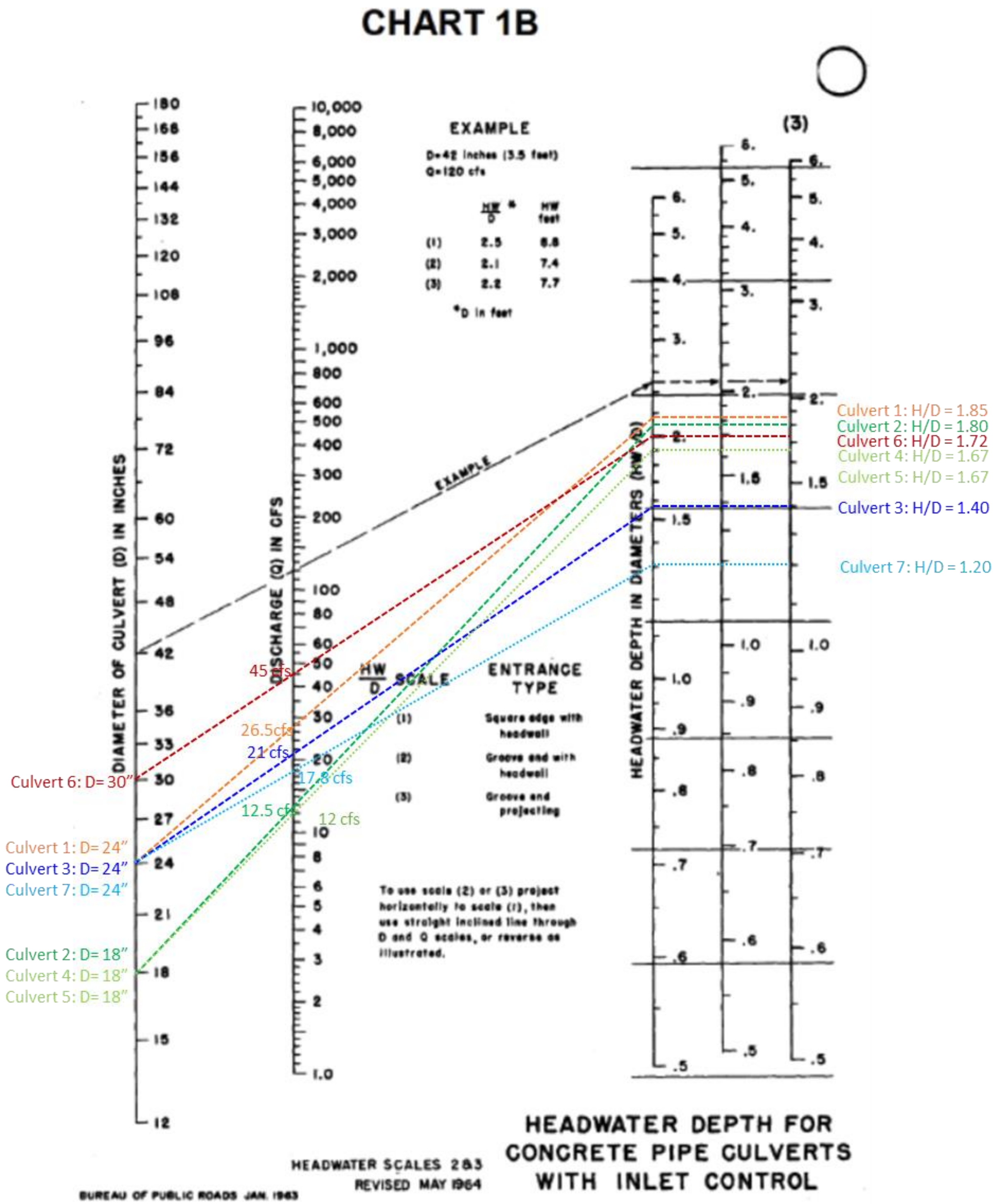

Figure 12 Culvert capacity determination at maximum headwater using FHWA's Chart 1B nomograph

Table 1
Harper Canyon Watershed
Existing Condition Culvert Flow Capacity and Exit Velocity Under Inlet Control

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]
Name	Ma-terial	Dia-meter	Length	U/S Invert Elevation (ft navd88)	D/S Invert Elevation (ft navd88)	Slope	Road Surface Elevation (ft navd88)	Max Allowable Headwater Depth (ft)	H/D	Flow Capacity with Headwater at Top of Culvert (cfs)	Exit Velocity with Headwater at Top of Culvert (ft/s)	Flow Capacity at Max Allowable Headwater (cfs)	Exit Velocity at Max Allowable Headwater (ft/s)
Culvert 1	CPP	24	37.0	318.7	318.6	0.27%	322.4	3.7	1.85	13.9	4.4	26.5	9.6
Culvert 2	CPP	18	40.7	325.6	325.7	-0.25%	328.3	2.7	1.80	6.8	5.4	12.5	7.5
Culvert 3	CPP	24	38.0	343.9	342.5	3.68%	346.7	2.8	1.40	13.9	13.0	21.0	14.5
Culvert 4	CPP	18	40.7	378.8	375.1	9.09%	381.3	2.5	1.67	6.8	15.1	12.0	17.7
Culvert 5	CPP	18	33.3	389.5	387.6	5.71%	392.0	2.5	1.67	6.8	12.8	12.0	14.9
Culvert 6	CPP	30	56.8	385.0	382.4	4.58%	389.3	4.3	1.72	24 x 2 = 48	16.1	45 x 2 = 90	19.1
Culvert 7	CIP	24	18.8	386.8	386.4	2.13%	389.2	2.4	1.20	13.9	10.7	17.8	11.4

Note:
- Culverts 1 and 3-7 have positive slope. The exit velocity was based on Manning's equation and solved using the tool @ URL: http://ponce.sdsu.edu/onlinechannel06.php
- Culvert 2 has negative slope. The exist velocity was assumed to be critical velocity and solved using the tool @ URL: http://ponce.sdsu.edu/onlinechannel07.php

Table 2
Harper Canyon Watershed
HEC-HMS Simulated Peak Flows at Culvert Inlet Locations

#	Location	Drainage Area (mi^2)	HEC-HMS Simulated Peak Flow (cfs)					
			2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
1	Culvert 1	0.039	0.6	2.6	4.9	10.0	14.1	18.5
2	Culvert 2	0.011	0.1	0.6	1.3	2.7	3.9	5.2
3	Culvert 3	0.190	5.3	28.5	56	85	106	128
4	Culvert 4	0.003	0.1	0.5	1.0	1.5	1.9	2.3
5	Culvert 5	0.012	0.7	3.3	5.7	8.2	10.1	12.0
6	Culvert 6	0.667	15.5	52	118	187	242	298
7	Culvert 7	0.016	0.4	3.3	6.4	9.5	11.9	14.3

Table 3
Harper Canyon Watershed
Existing Inlet Control Culvert Flow Capacities

Culvert	Flow Capacity with Headwater at Top of Culvert (H/W = 1) (cfs)	Return Interval with Headwater at Top of Culvert (H/W = 1) (years)	Able to Pass Q10yr Flow with Headwater at Top of Culvert (H/W=1)?	Able to Pass Q25yr Flow with Headwater at Top of Culvert (H/W=1)?	Flow Capacity at Max Allowable Headwater Elevation1 (cfs)	Return Period at Max Allowable Headwater Elevation (years)
Culvert 1	13.9	48	Yes	Yes	26.5	>100
Culvert 2	6.8	>100	Yes	Yes	12.5	>100
Culvert 3	13.9	2.6	No	No	21	3.4
Culvert 4	6.8	>100	Yes	Yes	12	>100
Culvert 5	6.8	13.6	Yes	No	12	>100
Culvert 6	48	4.3	No	No	90	7.0
Culvert 7	13.9	86	Yes	Yes	17.8	>100

Appendix A-3

Hydraulic Modeling Tech Memo #3

TECH MEMO #3

TO: Scott Walls, Walls Land + Water

FROM: Matt Smeltzer, P.E.
Guoyuan Li, Ph.D., P.E., P.H.

DATE: August 9, 2021

SUBJECT: TM#3 - Harper Canyon Watershed Hydraulic Modeling

---

1. Introduction

TM#1 computed the peak stream flow discharges at key locations in the Harper Canyon watershed including the inlets to seven existing culverts running under the gravel roadway. TM#2 compared the computed peak flow discharges at the gravel driveway culvert inlet locations to the estimated flow capacities of the existing culverts. This tech memo #3 (TM#3) summarizes HEC-RAS hydraulic modeling of the main channel from the reservoir to the watershed outlet to Pacheco Creek for both existing and proposed project conditions, and the evaluation of the existing culvert outlet flow conditions to ensure culverts meet the 100-year design criteria under existing conditions.

2. HEC-RAS Model Development

The Harper Canyon HEC-RAS model was developed as a 2D unsteady state model using the latest release HEC-RAS 6.0. HEC-RAS is distributed and maintained by the U.S. Army Corps of Engineers for open-channel hydraulic modeling. HEC-RAS 6.0 has the capability to simulate bridge hydraulics within 2D model; the 2D model for the Harper Canyon watershed can simulate hydraulics of the existing bridge and future proposed replacement bridge.

The HEC-RAS 2D model geometry requires a base terrain elevation surface, a 2D model grid configuration, and a spatially varying Manning's n roughness input data layer.

The terrain elevation surface was derived from the LiDAR data flown in 2020. Examinations of the LiDAR data suggest that the LiDAR data clearly captured the channel bottom elevations and channel bed elevation details that are critical for 2D modeling. The LiDAR data was processed into a 1 ft high resolution digital elevation model (DEM) to allow the HEC-RAS model to “see” the very detailed variations of the channel geometry.

The 2D model grid was developed to cover the entire model domain from the reservoir inlet to the watershed outlet to Pacheco Creek. The model grid was developed as 10-ft squares. The grid was refined with breaklines along high ground areas using finer resolution model grid as small as 2-ft. The fine resolution model grids capture detailed channel geometry to improve modeling accuracy.

The spatially varying Manning’s n roughness layer was determined based on land cover as suggested by the aerial photos and field pictures. The Manning’s n layer within the model domain consists of four categories: (a) channel, n = 0.035; (b) floodplain, n = 0.045; (c) road surface, n = 0.02; and (d) spillway bed rock, n = 0.05. These Manning’s n roughness values are based on literature recommended averages for the areas similar to the Harper Canyon watershed. Manning’s n may be refined in future model updates if calibration data become available.

The HEC-RAS model requires both boundary conditions and initial conditions to be able to run.

The boundary conditions were set as follows. At the downstream end, the normal depth was used as the downstream boundary condition. At the upstream end, a composite inflow hydrograph that combines all the sub-basin inflows from the upper watershed to the reservoir was used as the upstream boundary condition. In between the reservoir and the downstream end, the inflow hydrographs from the sub-basins on both sides of the main channel were used as the internal boundary conditions at corresponding locations.

The initial condition for the reservoir was set to be full, i.e., with a water surface elevation (WSE) at the spillway crest level of 432.6 ft NAVD88. The initial condition for the remaining portion of the model domain was assumed to be dry.

The unsteady state HEC-RAS model was set to run for a duration of 24 hours, which covers the entire flow hydrographs simulated by the HEC-HMS model. The computation time step was set to 0.5 second, as needed to achieve model stability and accuracy.

3. HEC-RAS Modeling Results

Six scenario runs were performed: combinations of three flow events (Q2yr, Q10yr, and Q100yr) and two geometry conditions (existing channel geometry and the proposed project channel geometry).

The modeling results at the key locations are highlighted as follows:

(a) In the spillway channel reach, the 10-year peak flow velocities vary from 7 ft/s to 22 ft/s for both existing and project conditions;

(b) At the Ford Crossing, the 100-year peak flow WSE is about 397.7 ft NAVD88 with a typical depth of 1.6 ft, and the velocity is about 15 ft/s for existing condition[1];

(c) On restored floodplain surfaces, the typical 2-year inundation depth is about 0.2 ft;

(d) At the bridge replacement crossing, the 100-year peak WSE is about 335.3 ft NAVD88 with a typical flow depth of 5.3 ft and velocity of 9 ft/s for restored, "pre-bridge channel with replacement clear span bridge" condition.

4. Culvert Tailwater Conditions Evaluation

The Harper Canyon HEC-RAS model simulated main channel WSE profiles can be used as the tailwater conditions to evaluate whether the driveway culverts #1 and #2 meet Santa Clara County's 100-year design criteria under existing conditions. Culverts #1 and #2 outfall directly into the main channel where high tailwater could make the inlet control assumption invalid under the 100-year flow condition.

Table 1 summarizes the HEC-RAS model simulated tailwater elevations for the seven driveway culverts, compared to the corresponding culvert outlet invert elevations. Both culverts #1 and #2 would have their outlet invert submerged by the 100-year tailwater elevations.

To double check whether the 100-year tailwater would lead to outlet control for culverts #1 and #2, a separate HEC-RAS model was developed for culverts 1 and 2 with the 100-year tailwater elevations as the downstream boundary conditions. The modeling results show that culvert #1 would still be under inlet control, with the flow capacity unchanged, while culvert #2 would be under outlet control, with the flow capacity reduced to 11.8 cfs. The reduced culvert #2 capacity is still much larger than the 100-year peak local inflow of 5.2 cfs, so Santa Clara County criteria are met.

Under the existing 100-year condition, overbank flows from the main channel upstream from culverts #1 and #2 flows overland to the inlets of culverts #1 and #2, ultimately flowing back to the main channel through those culverts or over the driveway near those culvert inlets. The escaped flows from the channel are much greater than the local inflows to the culverts under the 100-year condition. Although the culverts have enough capacity to pass all the local inflows,

---

[1] The model, as currently configured, represents current channel geometry conditions at the Ford Crossing Site Area reach, including the existing ford crossing and concrete barrier block grade control structure immediately downstream from the structure. Later model updates will evaluate potential crossing replacement or improvement measures to be determined.

the escaped flows exceed the culvert capacities. However, this problem is eliminated under the project condition. The main channel would be made large enough to contain the 100-year flow completely in the channel, eliminating the escaped flows to the inlets of culverts #1 and #2. Therefore, under the project condition, culverts #1 and #2 convey the 100-year peak flow to the main channel without overflowing the gravel driveway, meeting all Santa Clara County design criteria.

Similarly, although driveway culvert #7 can pass all the local 100-year inflows, the culvert #7 inlet would also receive overland flows escaping from the main channel in the Ford Crossing site area. The combined local and escaped flows exceed the capacity of culvert #7 under the existing 100-year condition. To prevent flow over the gravel driveway near culvert #7 inlet, either: (1) culvert #7 would have to be replaced with a number of parallel culverts, a single large arch culvert, or clear-span bridge deck; and/or (2) improvements would be needed near the Ford Crossing site to prevent floodplain overflows that flow to the culvert #7 inlet.

Table 1
Harper Canyon Watershed
Culvert Tailwater Conditions Evaluation

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]
Culvert	Diameter	Road Surface Elevation	Inlet Control Flow Capacity at Max Allowable Headwater	U/S Local Inflow (Q100yr)	U/S Escaped Channel Flow (Q100yr)	U/S Total Flow	D/S WSE (Q100yr)	D/S Invert	Tailwater Depth Above Invert	Evaluations
	(in)	(ft navd88)	(cfs)	(cfs)	(cfs)	(cfs)	(ft navd88)	(ft navd88)	(ft)	
Culvert 1	24	322.4	26.5	18.5	35	54	320.6	318.6	2.0	High Tailwater. But a simple culvert model using HEC-RAS suggests the culvert would be still inlet control.
Culvert 2	18	328.3	12.5	5.2	83	88	326.1	325.7	0.4	Outlet Control; Estimated new capacity is 11.8 cfs under Q100yr tailwater.
Culvert 3	24	346.7	21.0	128	0	128	341.5	342.5	-1.1	Free outflow; Inlet control
Culvert 4	18	381.3	12.0	2.3	0	2.3	357.5	375.1	-17.6	Free outflow; Inlet control
Culvert 5	18	392.0	12.0	12	0	12	365.7	387.6	-21.9	Free outflow; Inlet control
Culvert 6	30	389.3	45 x 2 = 90	298	0	298	379.2	382.4	-3.2	Free outflow; Inlet control
Culvert 7	24	389.2	17.8	14.3	52	66	384.6	386.4	-1.8	Free outflow; Inlet control