Placerville Drive at Hangtown Creek Bridge Replacement
Placerville, El Dorado County, California
Federal-Aid No. BRLS-5015(024)
Existing Bridge No. 25C0029; Proposed Bridge No. 25C0152

Bridge Design Hydraulic Study

Prepared for:

CITY OF PLACERVILLE
1854
OLD HANGTOWN

Prepared by:

HDR

October 2025

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Submitted to:
City of Placerville

This report has been prepared by or under the supervision of the following Registered Engineer. The Registered Civil Engineer attests to the technical information contained herein and has judged the qualifications of any technical specialists providing engineering data upon which recommendations, conclusions, and decisions are based.

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Registered Civil Engineer

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October 21, 2025
Date

October 2025

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Executive Summary

The City of Placerville (City) proposes to replace the existing Placerville Drive bridge crossing Hangtown Creek (Existing Bridge No. 25C0029) with a new bridge (Proposed Bridge No. 25C0152). The Placerville Drive at Hangtown Creek Bridge Replacement Project (Project) is located in Placerville within El Dorado County, approximately 0.7 miles (mi) west of the Placerville Drive undercrossing and 0.5 mi north of United States Highway 50. The existing bridge was previously identified as functionally obsolete by the California Department of Transportation (Caltrans) and is proposed to be replaced with a new concrete bridge designed to current City, American Association of State Highway and Transportation Officials, and Caltrans design criteria and standards.

The purpose of this *Bridge Design Hydraulic Study* is to present the design flow characteristics for Hangtown Creek, the hydraulic characteristics at the Project site, the calculated scour potential for the proposed bridge, and recommendations on the need for scour countermeasures for the proposed bridge.

The selected design discharges for the *Bridge Design Hydraulic Study* for the Project were obtained from a study by RBF Consulting. The RBF Consulting study for the *Hangtown Creek Comprehensive Watershed Plan* estimated a 100-year flow of 3,200 cubic feet (ft) per second (cfs) and a 50-year flow of 2,781 cfs for the Project.

A hydraulic model was developed using survey provided by R.E.Y. Engineers, Inc. to develop channel cross sections along Hangtown Creek. The proposed bridge was developed using information provided by Dewberry. The existing clear-span bridge has a minimum soffit elevation of 1,676.1 ft. The proposed three-span bridge will be wider and longer than the existing bridge. The minimum soffit elevation of the proposed bridge is 1,676.9 ft.

The results of the hydraulic model show reduced backwater effects just upstream of the bridge. There are localized increases in the WSE resulting from the proposed approach roadway improvements (with increases in WSE of 0.02 ft or less for the 100-year storm and 0.3 ft or less for the 50-year storm) and decreases in the WSEs immediately upstream of the bridge. The changes to the WSEs are attributed to the proposed approach roadway improvements and the removal of the existing bridge, which is currently a restriction to flow. The existing bridge does not provide freeboard during the 100-year and 50-year storm events. The proposed bridge would provide freeboard during the 100-year and 50-year storm events, and would meet Federal Highway Administration and Caltrans freeboard criteria.

Geotechnical investigations determined that there is bedrock at the Project site that is considered to be scour-resistant. The calculated total scour depths at the supports range from 6 to 9 ft. The presence of scour-resistant bedrock at the proposed Abutment 1 and Pier 2 supports would help to limit the progression of scour beyond the scour-resistant bedrock elevation. The bedrock elevation at Pier 3 and Abutment 4, is anticipated to be below the calculated scour elevation.

Per current Caltrans standards, abutment foundations should be designed such that they are stable with no lateral support, down to the calculated scour depth. However, rock slope protection (RSP) can still help to minimize bank erosion at the embankment slopes of the abutments. Based on calculations, rock slope protection Class VI, which has an approximate median weight of 3/8 ton, and a median diameter of 21 inches, is recommended at the embankment slopes. Additional RSP (or another properly designed erosion countermeasure) is also recommended to protect the area between the existing upstream culvert outfall and the new bridge.

Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ADT	average daily traffic
APN	accessor parcel number
BIR	Bridge Inspection Report
BMP	best management practice
Caltrans	California Department of Transportation
CDFW	California Department of Fish and Wildlife
City	City of Placerville
CSU	Colorado State University
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIS	Flood Insurance Study
ft	feet, foot
ft/s	feet per second
HBP	Highway Bridge Program
HDM	Highway Design Manual
HEC-18	Hydraulic Engineering Circular No. 18
HEC-23	Hydraulic Engineering Circular No. 23
HEC-RAS	Hydrologic Engineering Center's River Analysis System
HMS	hydrologic modeling system
LOTB	log of test boring
LRFD	Load and Resistance Factor Design
mi	mile
NBI	National Bridge Inventory
NCHRP	National Cooperative Highway Research Program
NGVD 29	National Geodetic Vertical Datum of 1929
Project	Placerville Drive at Hangtown Creek Bridge Replacement Project
RQD	rock quality designation
RS	river station
RSP	rock slope protection
RWQCB	Regional Water Quality Control Board
USFWS	U.S. Fish and Wildlife Service
USGS	United States Geological Survey
WSE	water surface elevation

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1 GENERAL DESCRIPTION

The City of Placerville (City) proposes to replace the existing Placerville Drive bridge crossing Hangtown Creek (Existing Bridge No. 25C0029) with a new bridge (Proposed Bridge No. 25C0152), which will accommodate the adjacent roadway facilities. The Placerville Drive at Hangtown Creek Bridge Replacement Project (Project) is located in Placerville within El Dorado County, approximately 0.7 miles (mi) west of the Placerville Drive undercrossing. See Figure 1 for the Regional Location Map and Figure 2 for the Project Location Map.

1.1 Introduction

The Placerville Drive at Hangtown Creek Bridge Replacement Project (proposed project), Federal Aid number BRLS-5015 (024), is located along Placerville Drive approximately 0.5 miles north of United States Highway 50, within the western portion of the City of Placerville. The general land use in the project vicinity consists of commercial and low-density residential uses. The existing roadway at the bridge is classified as a “Minor Arterial Road” and accommodates an Average Daily Traffic (ADT) of approximately 11,000 vehicle trips per day.

The proposed project is funded primarily by the federal-aid Highway Bridge Program (HBP) administered by the Federal Highway Administration (FHWA) through California Department of Transportation (Caltrans) Local Assistance. The replacement bridge would be designed to meet current applicable City, American Association of State Highway and Transportation Officials (AASHTO), and Caltrans design criteria and standards.

1.2 Project Purpose and Need

The bridge has an overall sufficiency rating of 60.4. The bridge has been previously identified as functionally obsolete due to substandard deck width and therefore, is eligible for rehabilitation or replacement under HBP guidelines.

The purpose of the proposed Project is to remove the existing functionally obsolete concrete bridge and replace it with a new concrete bridge designed to current structural and geometric standards that would provide adequate, reliable, and safe service for traffic. The new bridge would be designed to improve safety for vehicular, pedestrian, and bicycle traffic along Placerville Drive at the Project site.

1.3 Project Description

1.3.1 Existing Conditions

Constructed in 1931, the existing bridge is a single span reinforced concrete T-girder bridge on concrete abutments founded on spread footings. The bridge is approximately 45 feet (ft) long by 24 ft wide, and is within the City’s right-of-way. The bridge was previously determined to be functionally obsolete due to substandard deck geometry and

Figure 1. Regional Location Map

Source: Dewberry

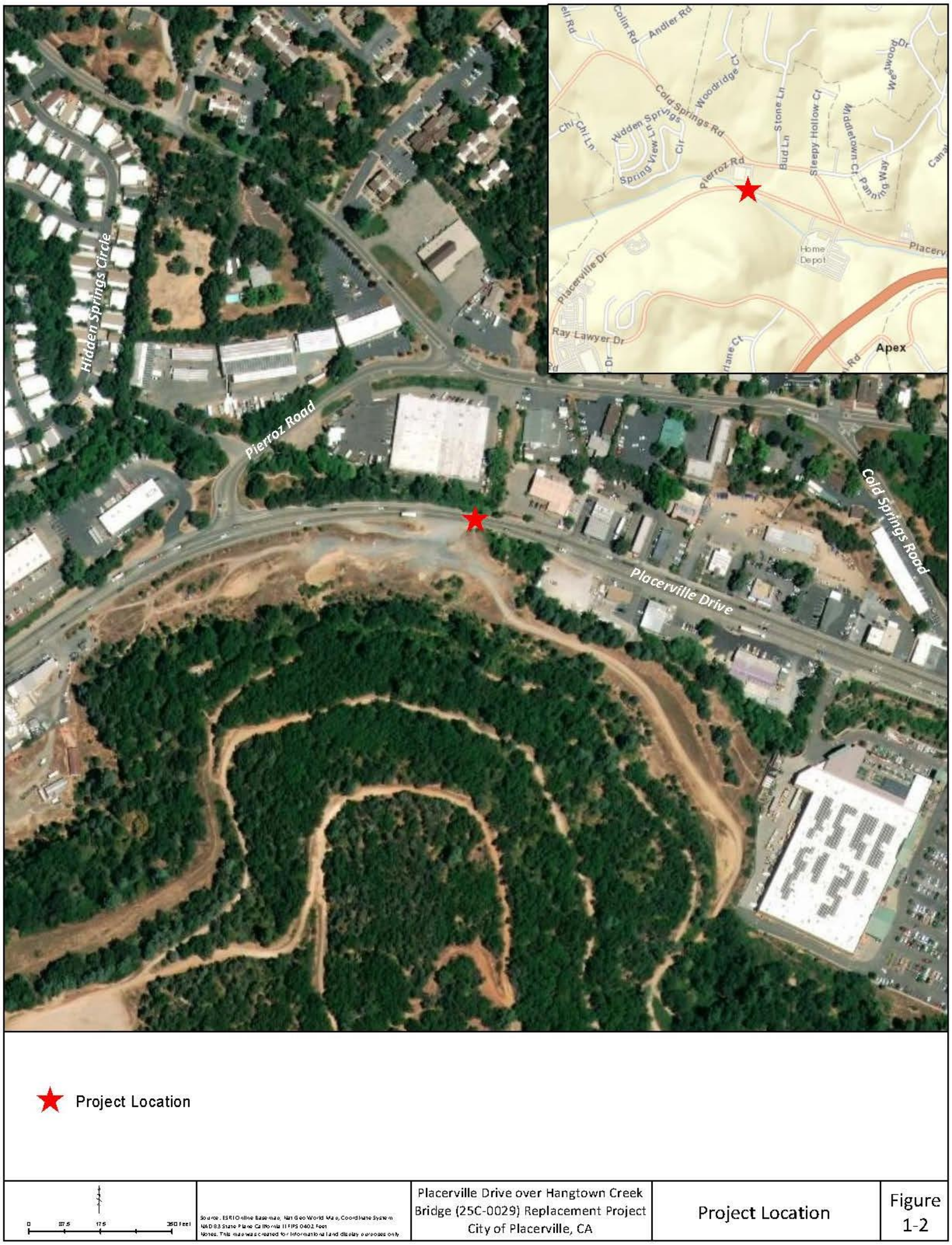

Figure 2. Project Location Map

Source: Dewberry

there are no accommodations for bikes or pedestrians across the bridge. The existing bridge is coded as a 5 “not eligible” by Caltrans for listing on the National Register of Historic Places. The City has determined the structure has no historical significance and therefore does not qualify for special historical considerations.

The most recent Caltrans Bridge Inspection Report noted that the existing bridge has vertical cracking in the concrete girders and both abutments. An 18-inch diameter spall exposing rebar was also observed along the northern railing at the eastern abutment.

1.3.2 Proposed Conditions

The proposed structure would be a three-span bridge, approximately 94 feet in length and approximately 64 feet in width, and would be raised 2 to 4 feet to accommodate Caltrans hydraulic standards for 50- and 100-year flood events. The proposed width would accommodate two 12-foot travel lanes, one 14-foot center turn lane as well as barriers, bicycle lanes, and pedestrian sidewalk facilities. The superstructure will be a cast-in-place concrete slab bridge supported by concrete abutments founded on spread footings. The piers will consist of concrete pile extensions socketed into rock. The new bridge would be lengthened on the western side to position the western abutment further away from the existing curve of Hangtown Creek at the bridge.

A creek diversion system would be used to divert flow through the construction zone and dewater the area around the bridge during construction. The creek diversion system would likely consist of placing coffer dams upstream and downstream of the construction site and conveying the water from Hangtown Creek through temporary culverts. Any temporary fill associated with the dewatering system would be removed at the end of construction, returning the creek to its original condition. The temporary cofferdams and culverts would be completely removed after the removal of the existing bridge and completion of the replacement bridge.

See Figure 3 for the proposed bridge replacement general plan.

1.3.3 Demolition

Demolition of the existing Placerville Drive Bridge (25C0029), existing retaining walls, asphalt, etc. would be performed in accordance with City standards, supplemented by Caltrans Specifications modified to meet environmental permit requirements. All concrete and other debris resulting from the demolition would be removed from the project site and properly disposed of by the contractor. The construction contractor would prepare a bridge demolition plan for the proposed project that would include the use of best management practices.

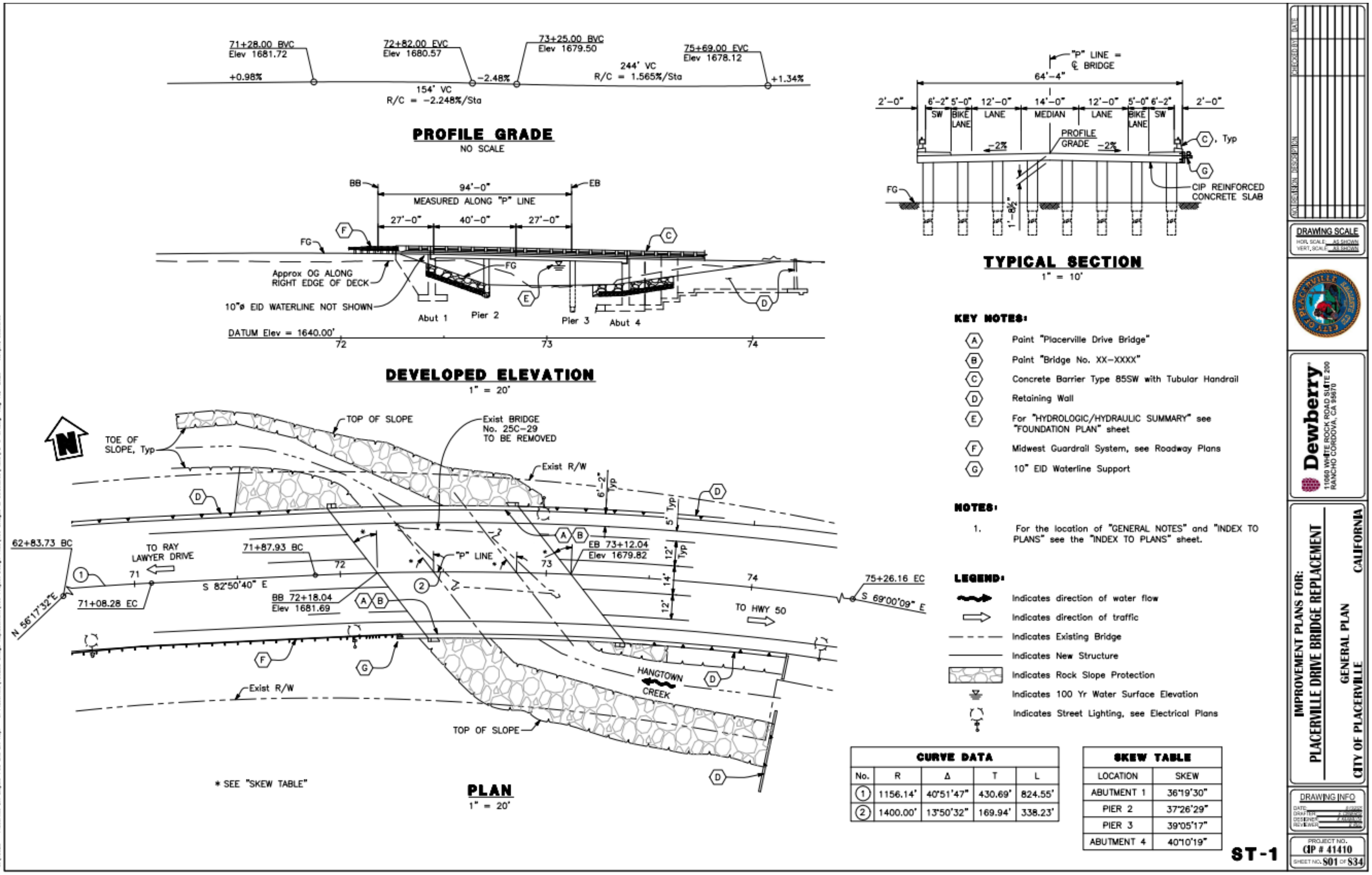

Figure 3. Proposed Bridge General Plan

Source: Dewberry

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1.3.4 Detour Route

Placerville Drive will be closed at the bridge during construction, and an approximate 0.5-mile temporary offsite detour utilizing Pierroz Road and Cold Springs Road will be used to maintain traffic (Figure 4). A detailed signage plans would be developed and approved by the City prior to the offsite detour implementation. Access to properties along Placerville Drive, between Cold Springs Road and Pierroz Road would be maintained throughout construction. Adjacent parcels would be informed of the project developments and of potential impacts to traffic operations prior to and during construction.

1.3.5 Right-of-Way

Permanent acquisition and temporary construction easements are needed to complete construction of the replacement bridge, approach roadway, and necessary driveway conforms.

1.4 Study Purpose

The purpose of this *Bridge Design Hydraulic Study* is to present the design flow characteristics for Hangtown Creek, the hydraulic characteristics at the Project site, the calculated scour potential for the proposed bridge, and recommendations on the need for scour countermeasures for the proposed bridge.

1.5 Key Tasks

Key tasks performed in this study included: 1) a review of available hydrologic data, 2) a hydraulic analysis to determine design water surface elevations (WSE) and flow velocities for the existing and proposed bridge conditions, 3) a scour analysis to estimate potential scour depths for the proposed bridge, and 4) scour countermeasure analyses and recommendations for the proposed bridge.

1.6 Design Criteria

1.6.1 Hydraulic Design Criteria

1.6.1.1 FHWA Standards

According to the *California Amendments to the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (2017 Eighth Edition)*, the FHWA mandated that LRFD be used on all new bridge design commencing on or after October 1, 2007 (Caltrans, 2019). The *California Amendments to the AASHTO LRFD BDS (2017 Eighth Edition)* Section 2.6.3 defers to state requirements for hydraulic studies.
From *Memo to Designers 16-1 Hydraulic Design for Structures over Waterways*, the proposed bridge soffit should provide adequate freeboard to pass anticipated drift for the 50-year design flood, or to pass the 100-year base flood without freeboard, whichever is greater (Caltrans, 2017).

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Figure 4. Detour Route

Source: Dewberr

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1.6.1.2 Caltrans Standards

From Chapter 820 of the Caltrans' *Highway Design Manual* (HDM), the criteria for the hydraulic design of bridges is that they be designed to pass the 2% probability of annual exceedance flow (50-year design discharge) with adequate freeboard to pass anticipated drift and debris (Caltrans, 2020a). Two (2) ft of freeboard is commonly used in bridge designs. Alternatively, the bridge can also be designed to pass the 1% probability of annual exceedance flow (100-year design discharge, or base flood). No freeboard is added to the base flood.

1.6.1.3 Project Freeboard Criteria

The hydraulic design criteria applicable to the Project site include the following:

- 50-year flow with freeboard (FHWA and Caltrans) with 2 ft of freeboard typically applied, and
- 100-year with no freeboard (Caltrans)

The Project has been designed to satisfy the above criteria from FHWA and Caltrans.

1.6.2 Foundation Criteria

Per the *California Amendments to the AASHTO LRFD BDS (2017 Eighth Edition)* (Caltrans, 2019), foundations should be designed to withstand the conditions of scour. Caltrans' *Memo to Designers 16-1* (2017) provides additional guidance on foundation placement:

> The top of a spread footing must be placed at or below the anticipated total scour (Degradation + Contraction + Local) elevation (*LRFD 2.6.4.4.2 and LRFD-BDS-CA Figure C2.6.4.4.2-1*) unless founded on competent, scour-resistant bedrock.
>
> The top of a pile cap footing must be placed at or below the estimated degradation plus contraction scour depth (*LRFD 2.6.4.4.2 and LRFD-BDS-CA Figure C2.6.4.4.2-2*). The bottom of a pile cap footing should be placed at or below the anticipated Total Scour elevation.

However, if the abutments are designed as piers, the foundations should be designed such that they are stable during the scour conditions, including loss of approach fill. This assumes an unsupported pile length above the total scour elevation.

1.6.3 Scour Design Criteria

The evaluation of potential scour at the proposed bridge followed the criteria described in the FHWA's *Hydraulic Engineering Circular No. 18* (HEC-18), "Evaluating Scour at Bridges" (FHWA, 2012). The minimum scour design flood frequency is based upon the hydraulic design flood frequency. For a 50-year hydraulic design frequency, the evaluation of potential scour is based upon the hydraulic characteristics of a 100-year design flood frequency. The total scour was estimated based upon the cumulative effects of the long-term bed elevation change, general (contraction) scour, and local scour. The life expectancy of the bridge is considered in determining the long-term bed elevation change of the waterway. A replacement bridge would be based on an assumed 75-year design life.

1.6.4 Rock Slope Protection Design Criteria

Two procedures for determining rock slope protection (RSP) design were considered for the proposed structure: the FHWA's *Hydraulic Engineering Circular No. 23* (HEC-23), "Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance" (FHWA, 2009), and Caltrans' HDM (2020a). The final selection considers both of these procedures and is based on engineering judgment.

1.7 Vertical Datum

The Project references the National Geodetic Vertical Datum of 1929 (NGVD 29). All elevations in this report reference NGVD 29 unless otherwise specified.

2 GEOGRAPHIC SETTING

2.1 Geographic Location

The City of Placerville is located approximately 40 mi east-northeast of the City of Sacramento. According to the Caltrans BIRs (2018), the existing Placerville Drive Bridge is located at 38°44’00.47” north latitude and 120°49’39.92” west longitude.

2.2 Watershed Description

Hangtown Creek originates in unincorporated El Dorado County near the community of Smithflat and the stream path passes through the City of Placerville (see Figure 5). The creek flows roughly from east to west. The drainage area of Hangtown Creek is approximately 7.6 square mi (United States Geological Survey [USGS], 2019), and includes the contributing area from Randolph Canyon, which joins with Hangtown Creek near the intersection of Mosquito Road and El Dorado Trail/Broadway. From its confluence with the Randolph Canyon unnamed stream, Hangtown Creek continues flowing west for approximately 2 mi before reaching the Project site at Placerville Drive.

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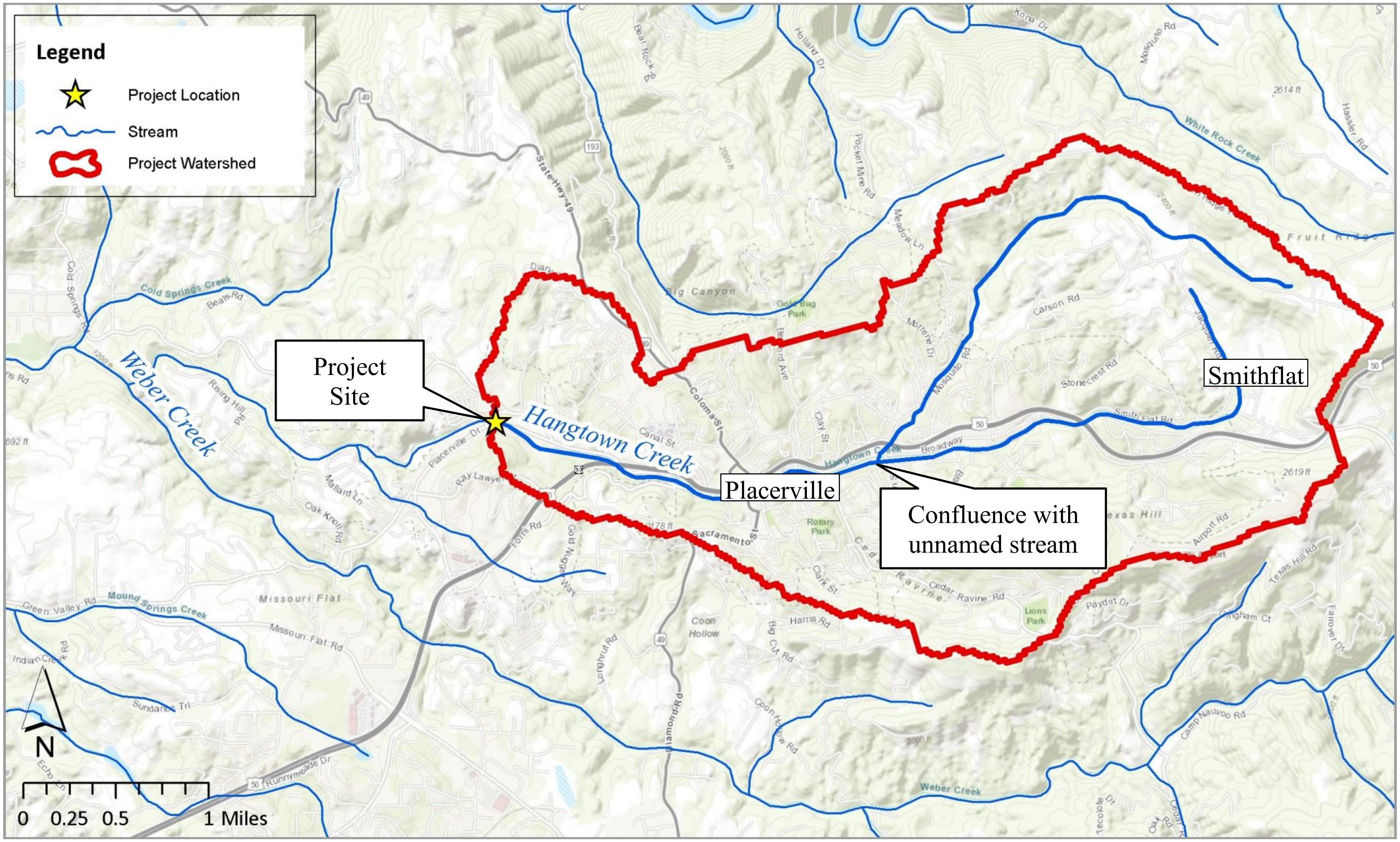

Figure 5. Project Watershed Map

Sources: USGS & Environmental Systems Research Institute

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3 HYDROLOGIC ANALYSIS

3.1 Hydrologic Design Methods

There are no known USGS peak streamflow gages along Hangtown Creek in the Project vicinity. The design discharges for the Project were obtained from the Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) and a Hangtown Creek hydrologic study performed by RBF Consulting in 2012.

3.2 Design Discharge Summary

3.2.1 Federal Emergency Management Agency Flood Insurance Study

The FEMA FIS describes a detailed study of Hangtown Creek, and the FIS includes four flow change locations in the summary of discharges table. The peak design flows for Hangtown Creek are shown below in Table 1. The hydrologic analysis of Hangtown Creek was completed in 1980.

Table 1. Hangtown Creek Peak Discharges from FIS

Location	Drainage Area (square mi)	Peak Discharge (cfs)	
		100-year	50-year
At western corporate limits	8.0	2,920	2,410
At confluence with Cedar Ravine	5.6	1,870	1,540
At confluence with Randolph Canyon	4.7	1,380	1,140
At confluence with Hangtown Creek tributary	1.6	490	410

Source: FEMA

Hangtown Creek drains an area of approximately 7.6 square mi at the Project site, which is nearly identical to the drainage area listed for the location described as “at western corporate limits” in the FIS. Therefore, the discharge values at the western corporate limits were used for the Project because of its proximity to the Project site.

3.2.2 Hangtown Creek Comprehensive Watershed Plan Study

Flows for the Project site were also obtained from the *Hangtown Creek Comprehensive Watershed Plan* (RBF Consulting, 2012); see Table 2. The hydrologic analysis performed by RBF Consulting was based on the methodologies from the *El Dorado County Drainage Manual*, and it incorporates development that has occurred within the Hangtown Creek watershed since FEMA’s study was completed in 1983. Differences in the flows presented in the FIS and calculated by RBF Consulting were attributed to development within the watershed as well as differences in the modeling methodologies.

Table 2. Hangtown Creek Peak Discharges from the Hangtown Creek Comprehensive Watershed Plan

Location	Drainage Area (square mi)	Peak Discharge (cfs)	
		100-year	50-year
HMS Node J50	7.05	3,200	2,781
HMS Node J53	7.51	3,370	2,929

Source: RBF Consulting

Note: HMS = hydrologic modeling system

The drainage areas associated with these two nodes are reported to be less than the drainage area at the Project site. However, RBF Consulting developed a Hydrologic Engineering Center's River Analysis System (HEC-RAS) model of Hangtown Creek, and these are the flows associated within the Project reach. The flow at hydrologic modeling system (HMS) Node J50 is associated with the area upstream and through Placerville Drive, and the flow at HMS Node J53 is associated with the area just upstream of and through Pierroz Road.

3.3 Selected Design Discharges

The RBF Consulting flows from the *Hangtown Creek Comprehensive Watershed Plan* were selected as the design discharges used to assess the hydraulic conditions at the Project site and for the design of the bridge. The RBF Consulting flows are more updated and conservative than those presented in the FEMA FIS.

3.4 Hydrologic Stability

Due to the nature of the work proposed by the Project, the Project would not change the overall land use within the watershed. The design discharges selected for the Project were estimated in the RBF Consulting study of the Hangtown Creek watershed, which incorporated development that has occurred within the watershed from 1983 to 2011. The design discharges for the Project would not be anticipated to significantly change during the lifetime of the bridge at the Project location; however, additional future developments within the watershed would have the potential to impact the hydrologic conditions of the watershed and at the Project site.

4 HYDRAULIC ANALYSIS

The following sections discuss the development of the hydraulic models and summarize the results for the existing and proposed conditions. The water surface profile plots, hydraulic summary tables, and channel cross sections are included in Appendix A for the existing bridge and Appendix B for the proposed bridge.

4.1 Design Tools

A one-dimensional steady-state hydraulic model was developed to evaluate and assess whether the Project improvements would impact the 100-year or 50-year WSEs of Hangtown Creek. The hydraulic analyses were performed for the existing and proposed conditions using the HEC-RAS modeling software, Version 5.0.7.

4.2 Hydraulic Model Development

4.2.1 Cross Section Data

The hydraulic model was developed to encompass the area in the vicinity of Placerville Drive where the bridge is proposed to be replaced. Cross sections of Hangtown Creek were developed using elevation data provided by R.E.Y. Engineers, Inc. in 2019 and 2021 (R.E.Y. Engineers, Inc., 2019 and 2021a). The survey references NGVD 29. The surveyed cross sections extend approximately 580 ft upstream and 740 ft downstream of the existing bridge and encompass a river reach of approximately 1,320 ft in length along Hangtown Creek. The cross section naming convention is by river station (RS) with the cross section number increasing in the upstream direction. The most downstream cross section in the Project hydraulic model was based on data from the HEC-2 effective hydraulic model. Cross sections were updated in the proposed condition model using grading and surface data provided by Dewberry (2021).

4.2.2 Modeled Hydraulic Structures

The existing bridge was modeled at RS 736.2. The existing clear-span bridge has a structure length of 44.9 ft and a width of 27.6 ft. The minimum soffit elevation of the existing bridge is 1,676.1 ft.

The proposed bridge was modeled at RS 750.9. The proposed three-span bridge will have a structure length of 94 ft and a width of 64 ft, 4 inches, which will be wider and longer than the existing bridge. The minimum soffit elevation of the proposed bridge is 1,676.9 ft. The two piers were modeled with a width of 2.5 ft and triple the width of the pier along their full height to account for debris. They are modeled to be aligned with the flow direction.

Two other structures are located in the vicinity of Placerville Drive: a box culvert upstream of Placerville Drive and an arch culvert downstream of Placerville Drive. Both structures are included in the existing and proposed condition models. Blocked obstructions were also utilized in the hydraulic model to represent buildings that are located adjacent to the creek.

4.2.3 Model Boundary Condition

A WSE of 1,653.2 ft was used as the boundary condition for the FEMA FIS flow, and it was based on the WSE from the effective hydraulic model. A normal depth slope of 0.02 ft/ft was used as the boundary condition for the RBF flow, and it was based on the survey of Hangtown Creek within the vicinity of the Project.

4.2.4 Manning's Roughness Coefficients

Manning's roughness coefficients were used in the hydraulic model to estimate energy losses in the flow due to friction. A roughness coefficient of 0.06 was used to describe the main channel. The roughness coefficients used to describe the overbank areas varied from 0.045 in grassy areas to 0.1 in areas with denser vegetation. A roughness coefficient of 0.016 was used to represent the paved surfaces such as roadways and parking lots. These values were selected based on visual observations of the Project vicinity. Photo 1 shows the channel at the upstream side of the bridge.

Photo 1. South Side of Existing Bridge (Looking East)

Source: Dewberry

4.2.5 Expansion and Contraction Coefficients

Expansion and contraction coefficients were used in the hydraulic model to represent energy losses in the channel. An expansion coefficient of 0.3 and a contraction coefficient of 0.1 were used to represent the channel in the vicinity of the Project. These values represent a channel with a gradual transition between cross sections. The expansion and contraction coefficients used in the vicinity of the bridges were 0.5 and 0.3, respectively. These values represent the flow interference caused by the bridges.

4.3 Hydraulic Model Results

4.3.1 Water Surface Elevations

The cross sections at the upstream faces of the existing and proposed bridges are shown in Figure 6 and Figure 7, respectively. The cross sections are shown facing the downstream (or northwesterly) direction.

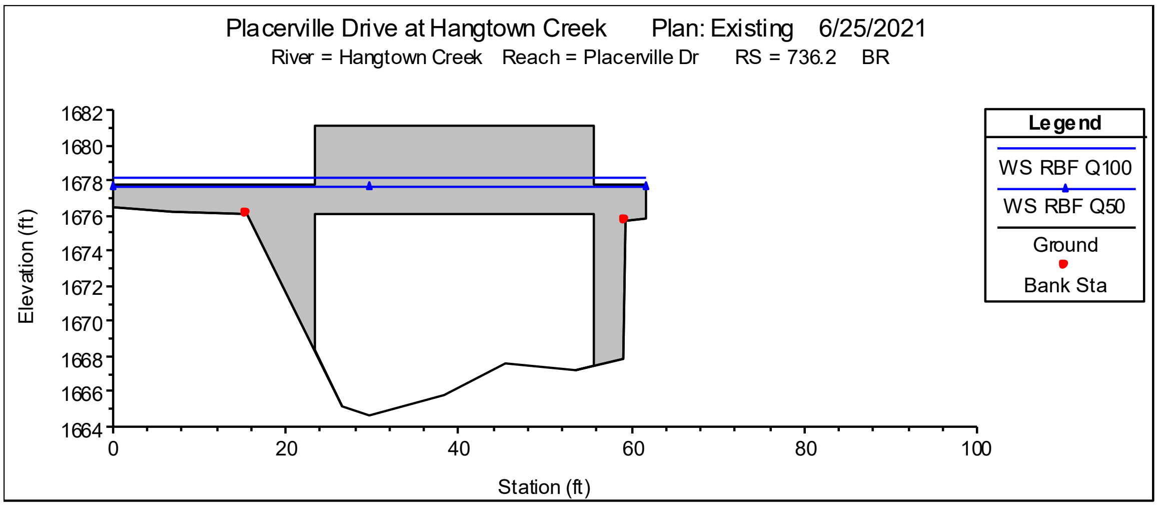

Figure 6. Existing Bridge Cross Section (Facing Downstream)

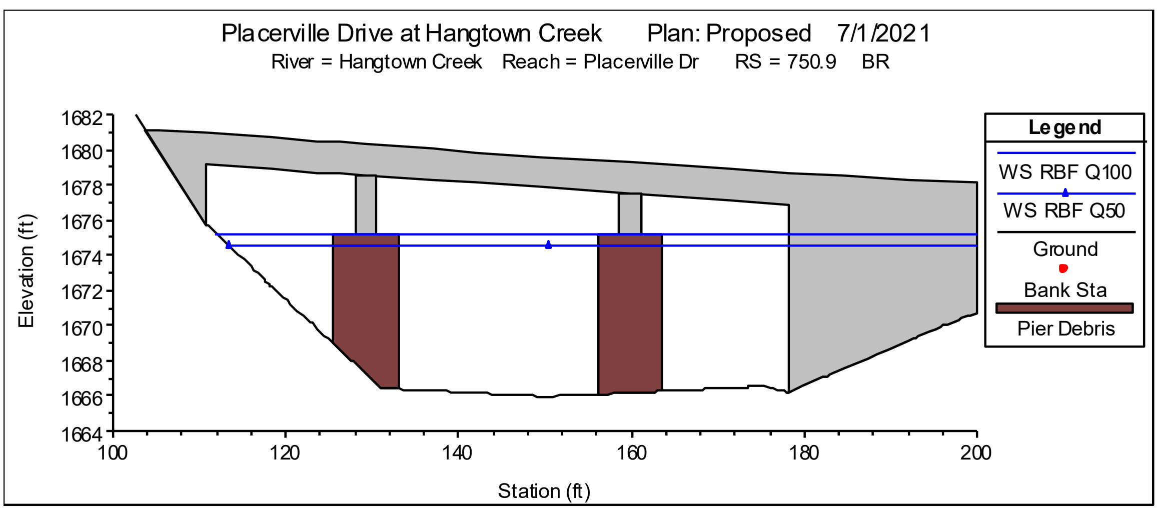

Figure 7. Proposed Bridge Cross Section (Facing Downstream)

The cross sections show the existing approach roadways are overtopped during the 100-year and 50-year design storm events, and the proposed bridge would clear both design storm events.

The water surface profiles along Hangtown Creek in the Project vicinity are shown in Figure 8 and Figure 9 for the evaluated flows. The downstream direction is on the left side of the figures. Both the existing and proposed bridges at Placerville Drive are shown for comparison, but because the existing bridge will be removed as part of the Project, only the existing bridge is modeled in the existing condition, and only the proposed bridge is modeled in the proposed condition. The results of the hydraulic model show reduced backwater effects just upstream of the bridge.

There are localized increases in WSE resulting from the proposed approach roadway improvements and decreases in the WSEs immediately upstream of the bridge. The maximum increases in WSE along the study reach are 0.02 ft or less for the 100-year storm and 0.3 ft or less for the 50-year storm. The magnitude of the differences in WSEs are summarized in Table 4 and Table 5.

The changes to the WSEs are attributed to the proposed approach roadway improvements and the removal of the existing bridge, which is currently a restriction to flow. Additional hydraulic modeling result details are included in Appendix A for the existing condition and Appendix B for the proposed condition.

According to Caltrans' *Memo to Designers 16-1* (2017), a hydrologic summary table shall be placed on the Project Foundation Plan. The hydrologic summary is shown in for the proposed bridge (25C0152).

Table 3. Hydrologic Summary Table

Hydrologic Summary for Proposed Bridge No. 25C0152			
Drainage Area: 7.6 mi^2			
Frequency	**Design Flood**	**Base Flood**	**Flood of Record**
	50-year	100-year	N/A
Discharge	2,781	3,200	N/A
Water Surface Elevation at Bridge	1,674.9	1,675.7	N/A

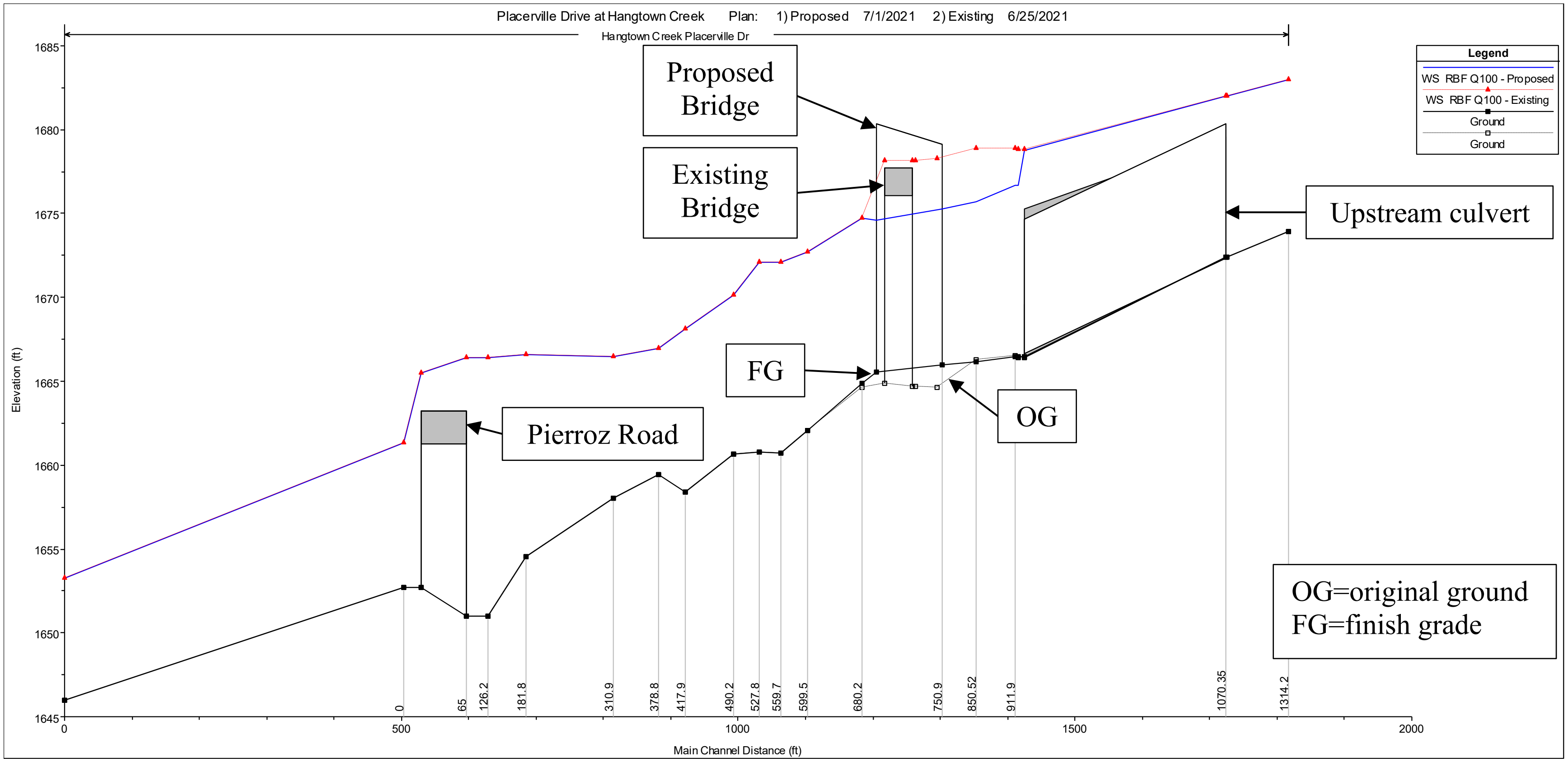

Figure 8. 100-year Flow Water Surface Profile Comparison

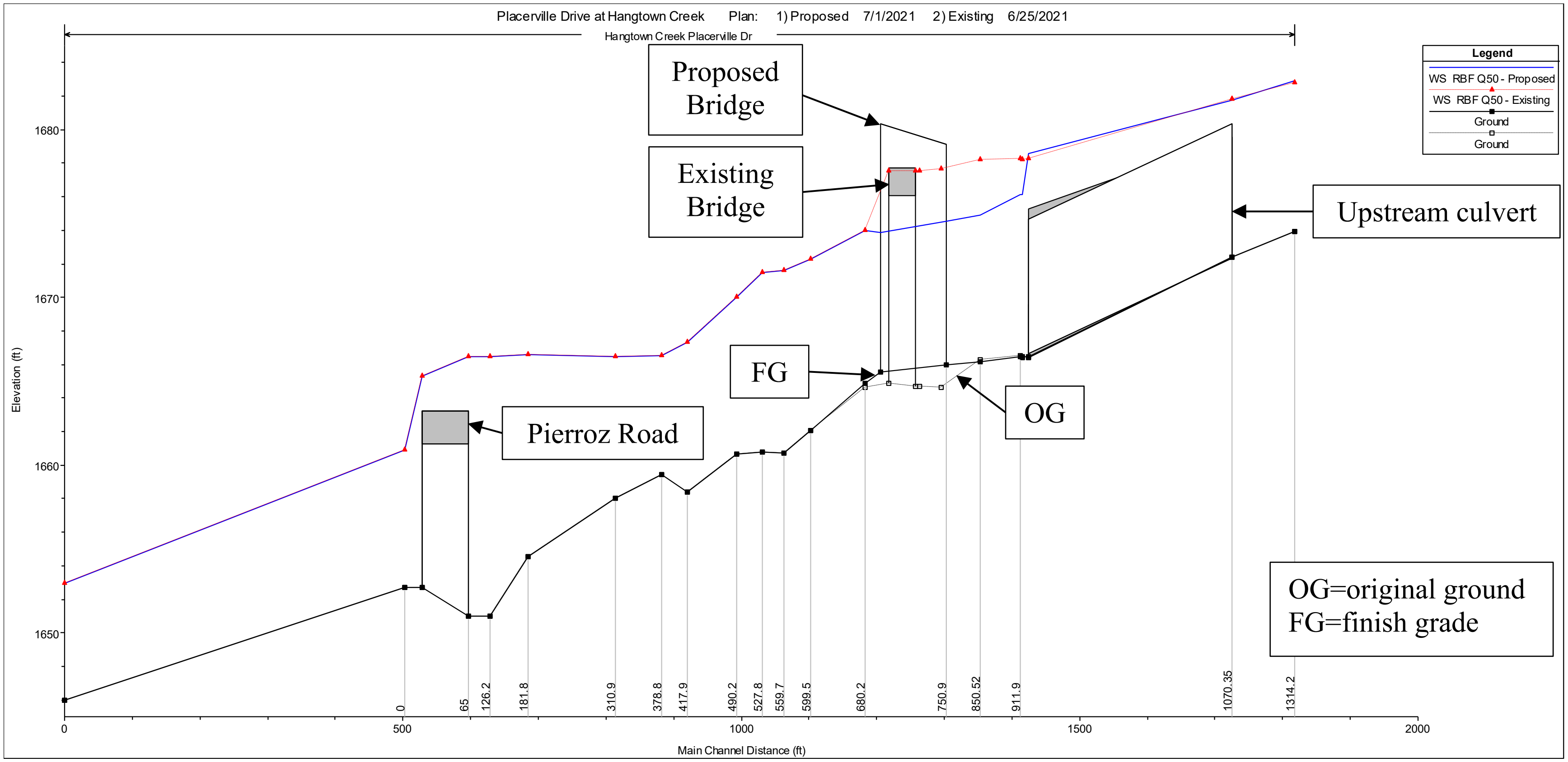

Figure 9. 50-year Flow Water Surface Profile Comparison

Table 4. 100-year Flow Water Surface Elevations

River Station	Description/Distance from Existing Bridge Centerline (ft)	WSE (ft)		Difference in WSE (ft)
		Existing	Proposed	
		[1]	[2]	[2]-[1]
1314.2	580 ft upstream	1,683.01	1,683.01	0.00
1222	487 ft upstream	1,682.01	1,682.01	0.00
1070.35 US Culv	Upstream face of culvert	1,682.01	1,682.01	0.00
1070.35 DS Culv	Downstream face of culvert	1,678.83	1,678.77	-0.1
915.1	180 ft upstream	1,678.83	1,676.67	-2.2
911.9	177 ft upstream	1,678.91	1,676.67	-2.2
850.5	116 ft upstream	1,678.91	1,675.69	-3.2
750.9 BR U	Upstream face of proposed bridge	--	1,675.25	--
791.8	57 ft upstream	1,678.26	--	--
760.4	26 ft upstream	1,678.16	--	--
736.2 BR U	Upstream face of existing bridge	1,678.16	--	--
736.2 BR D	Downstream face of existing bridge	1,678.15	--	--
750.9 BR D	Downstream face of proposed bridge	--	1,674.58	--
680.2	54 ft downstream	1,674.72	1,674.74	0.02
599.5	135 ft downstream	1,672.72	1,672.72	0.00
559.7	175 ft downstream	1,672.12	1,672.12	0.00
527.8	207 ft downstream	1,672.08	1,672.08	0.00
490.2	244 ft downstream	1,670.12	1,670.12	0.00

Notes:
BR U = upstream face of bridge
BR D = downstream face of bridge

Table 5. 50-year Flow Water Surface Elevations

River Station	Description/Distance from Existing Bridge Centerline (ft)	WSE (ft)		Difference in WSE (ft)
		Existing	Proposed	
		[1]	[2]	[2]-[1]
1314.2	580 ft upstream	1,682.8	1,682.9	0.1
1222	487 ft upstream	1,681.8	1,681.7	-0.1
1070.35 US Culv	Upstream face of culvert	1,681.8	1,681.7	-0.1
1070.35 DS Culv	Downstream face of culvert	1,678.3	1,678.6	0.3
915.1	180 ft upstream	1,678.2	1,676.1	-2.1
911.9	177 ft upstream	1,678.3	1,676.1	-2.2
850.5	116 ft upstream	1,678.2	1,674.9	-3.3
750.9 BR U	Upstream face of proposed bridge	--	1,674.6	--
791.8	57 ft upstream	1,677.7	--	--
760.4	26 ft upstream	1,677.6	--	--
736.2 BR U	Upstream face of existing bridge	1,677.6	--	--
736.2 BR D	Downstream face of existing bridge	1,677.6	--	--
750.9 BR D	Downstream face of proposed bridge	--	1,673.9	--
680.2	54 ft downstream	1,674.0	1,674.0	0.0
599.5	135 ft downstream	1,672.3	1,672.3	0.0
559.7	175 ft downstream	1,671.6	1,671.6	0.0
527.8	207 ft downstream	1,671.5	1,671.5	0.0
490.2	244 ft downstream	1,670.0	1,670.0	0.0

Notes:
BR U = upstream face of bridge
BR D = downstream face of bridge

4.3.2 Freeboard

Based on the results of the analysis, the existing bridge does not provide freeboard (or clearance from the soffit) during the 100-year and 50-year storm events. The proposed replacement bridge would provide freeboard during the 100-year and 50-year storm events. Table 6 presents the applicable WSEs and freeboard during the respective 100-year and 50-year storm events at the upstream side of the bridges.

Table 6. WSEs and Freeboard Comparison for Existing and Proposed Bridges

Alternative	Soffit Elevation (ft)	Return Period	WSE (ft)	Freeboard (ft)
Existing	1,676.1	100-year	1,678.2	no freeboard
Proposed	1,676.9		1,675.7	1.2
Existing	1,676.1	50-year	1,677.6	no freeboard
Proposed	1,676.9		1,674.9	2.0

The proposed bridge would meet the freeboard criteria of the FHWA and Caltrans by passing the 50-year storm event with at least 2 ft of freeboard and the 100-year storm event without freeboard.

4.3.3 Flow Velocities

The 100-year average channel flow velocities were estimated for the existing and proposed conditions from the developed hydraulic models, and are summarized in Table 7 for the locations in the vicinity of the bridges. Based on the hydraulic analysis, the proposed bridge would result in localized increases in average channel velocities upstream of the proposed bridge due to the decreased backwater. The magnitude of the changes in velocities are presented in Table 7. RSP will be incorporated as part of the bridge design in the vicinity of the bridge, and downstream of the bend (see Section 6) to protect the bridge abutments and channel banks. The hydraulic model also shows increases in velocities in the area downstream of the existing culvert and upstream of the replacement bridge. Additional bank protection measures should be considered as part of the design to address the increases in velocities in the area downstream of the existing culvert and upstream of the replacement bridge. The disturbed channel should be designed to maintain a roughness value comparable to the existing conditions to avoid floodplain impacts. Measures to address velocities could include RSP, articulated concrete blocks with planting, or another form of erosion control.

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Table 7. 100-year Average Channel Flow Velocities

River Station	Description/Distance from Existing Bridge Centerline (ft)	Velocity (ft per second [ft/s])		Difference in Velocity (ft/s)
		Existing	Proposed	
		[1]	[2]	[2]-[1]
915.1	180 ft upstream	4.6	7.9	3.3
911.9	177 ft upstream	3.7	7.8	4.2
850.5	116 ft upstream	4.2	9.4	5.2
750.9 BR U	Upstream face of proposed bridge	--	8.3	--
791.8	57 ft upstream	6.9	--	--
760.4	26 ft upstream	6.6	--	--
736.2 BR U	Upstream face of existing bridge	9.8	--	--
736.2 BR D	Downstream face of existing bridge	9.6	--	--
750.9 BR D	Downstream face of proposed bridge	--	8.0	--
680.2	54 ft downstream	5.3	5.5	0.2
599.5	135 ft downstream	8.8	8.8	0
559.7	175 ft downstream	9.5	9.5	0

Notes:
BR U = upstream face of bridge
BR D = downstream face of bridge

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5 SCOUR ANALYSIS

HDR evaluated bridge scour per the criteria described in "Evaluating Scour at Bridges" (FHWA, 2012). The minimum design criterion for bridge scour is the 100-year design storm. The scour calculations assume the channel bed material is erodible down to the elevations of scour-resistant rock. The following sub-sections summarize the results of the analysis.

5.1 Caltrans Bridge Inspection Reports

Available Caltrans BIRs for the existing bridge were reviewed for relevant scour information. The 2024 BIR is the most recently available routine inspection at the time of this study. At the time of the inspection, it was reported that clear water was flowing in the creek against Abutment 1 and that the lower portion of the abutment was submerged but could be viewed through the clear water while Abutment 2 was dry. The National Bridge Inventory (NBI) Item 113 (scour critical bridges) rating from this BIR assigned the existing bridge a code 5 (Caltrans, 2020b). A rating of 5 was assigned to bridges with foundations that have been determined to be stable for assessed or calculated scour conditions; scour is determined to be within the limits of footings or piles by assessment, calculations, or the installation of properly designed countermeasures (FHWA, 1995). A summary of BIRs is presented in Table 8.

Table 8. BIR Channel and Waterway Summary

Date	Summary
07/13/2024	Channel Description: High flow with a rocky channel bottom. Scope and Access: Up to 10 inches of clear water was flowing in the creek near Abutment 1 during this inspection. The lower portion of Abutment 1 was submerged but could be viewed through the clear water and probe stick was utilized. Abutment 2 was dry. The substructure and superstructure were inspected by walking under the bridge. The bridge deck was inspected by walking the deck. All visible elements received a complete routine inspection. The bridge NBI Item Code 113 is rated 5. This BIR measured the channel cross-section along the top of rail at the time of the investigation and noted no significant change when compared to the previous cross section dated 07/26/2016.
07/26/2022	Channel Description: High flow with a rocky channel bottom. Scope and Access: Six to 10 inches of clear water was flowing in the creek against Abutment 1 during this inspection. The lower portion of Abutment 1 was submerged but could be viewed through the clear water. Abutment 2 was dry. The substructure and superstructure were inspected by walking under the bridge with a probe stick. The bridge deck was inspected by

Date	Summary
	walking the deck. All visible elements received a complete routine inspection. The bridge NBI Item Code 113 is rated 5.
07/15/2020	Channel Description: High flow with a rocky channel bottom. Scope and Access: Six to 10 inches of clear water was flowing in the creek against Abutment 1 during this inspection. The lower portion of Abutment 1 was submerged but could be viewed through the clear water. Abutment 2 was dry. The substructure and superstructure were inspected by walking under the bridge with waders and a probe stick. The bridge deck was inspected by walking the deck. All visible elements received a complete inspection. The bridge NBI Item Code 113 is rated 5. It notes that the lower 6 inches of the right half of Abutment 1 is abraded with no loose aggregate.
07/19/2018	Channel Description: High flow with a rocky channel bottom. Scope and Access: There was 10 inches of water in the creek flowing against Abutment 1. The lower portion of Abutment 1 was submerged but still accessible with waders and a probe stick. The dry portions under the structure and the deck was inspected by walking on and under the structure. All elements received a complete inspection. The bridge NBI Item Code 113 is rated 5.
07/26/2016	Channel Description: Rocky swift flow. Scope and Access: There was 8 inches of water under the structure. A complete inspection was performed. The inspection frequency was changed from 48 months to 24 months. Waterway: A 10 year channel cross section was measured during this inspection. The data was compared to the data taken on 10/16/2007. This comparison yielded minor degradation (~12-18 inches) at the center of the channel. No corrective action is required. The bridge NBI Item Code 113 is rated 5. At the time of the investigation, a channel cross-section was measured along the top of rail.

Date	Summary
07/23/2012	Scope and Access: A complete inspection was done. No defects were found in the substructure. Slope protection is in place and functioning well. The bridge NBI Item Code 113 is rated 5.
07/24/2008	Inspection Access: A complete inspection was done. No defects were found in the substructure. Slope protection is in place and functioning well. The bridge NBI Item Code 113 is rated 5.
10/16/2007	History: This is a single span bridge built in 1930. A review of all existing BIRs reveals there is a history of Abutment 1 footing exposure but no undermining. Miscellaneous: The NBI Item 113 code had been previously revised from 6 to 8 in 2000. However, no documentation was provided at that time. The purpose of this report is to assess the bridge for scour given the information available within Caltrans archives and to provide documentation regarding the 113 code. Revisions: The NBI Item 113 Code has been revised from 8 to 5. Scour: This report addresses hydraulic issues only. The structure's scour potential has been assessed in accordance with the FHWA Technical Advisory T5140.23, "Evaluating Scour at Bridges". The NBI Item 113 Code, "Vulnerability to Scour", has been changed to 5: "Bridge Foundations determined to be stable for assessed or calculated scour conditions. Scour is determined to be within the limits of footings or piles (Example B) by assessment {i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge}, by calculations or by installation of properly designed countermeasures." This report is generated based on a hydraulic field investigation (10/16/2007). At the time of the investigation, a channel cross-section was taken. No comparison of the cross-section could be made because this is the only cross-section on record for this bridge. The channel had slow moving water at the time of the investigation. The water depth was about 1.0 ft. The channel bed material consisted mainly of sand, cobbles, and some large rocks with heavy vegetation growing throughout the embankments. The channel does not have a good

Date	Summary
	alignment to the bridge opening. The thalweg appears to flow towards Abutment 1. The upstream top of Abutment 1 footing is exposed about 2.0-2.5 ft in length and about 0.58 ft (0.18 m) vertically. No undermining was observed. This portion of the footing of Abutment 1 is protected by large RSP and broken concrete that appears to be in good condition. Given the amount of foundation exposure relative to the entire length of the foundation, this scour appears minor at this time. No other scour was noted at Abutment 1 and the channel appears to become well aligned after entering the upstream bridge opening. No foundation exposure around Abutment 2. There appears to be plenty of ground cover. No scour issues were noted during the field investigation. After a hydraulic field review, and a review of historical bridge inspection reports, a revision of the item 113 scour code is warranted at this time. However, scour currently does not pose a major threat to the integrity of the structure and this bridge is not scour critical at this time.

5.2 Existing Channel Bed

The soils along the channel are gravel with sand per the 2019 boring presented in the *Draft Foundation Report* for the Project (HDR, 2025). Based on this boring, the channel bed material was considered cohesionless for the purposes of analyzing scour. Per the *Draft Foundation Report* (HDR, 2025), the exposed, weathered rock in the channel is deemed resistant to scour. The report also states that the dense angular gravel and cobbles, which is considered scourable, is approximately at an elevation of 1,666.1 ft at Abutment 1 and Pier 2 and 1,659.5 ft at Pier 3 and Abutment 4. Scour resistant bedrock starts approximately at an elevation of 1,661.7 ft at Abutment 1 and Pier 2 and 1,655.4 ft at Pier 3 and Abutment 4.

5.3 Long-Term Bed Elevation Change

Long-term bed elevation changes can be due to either aggradation or degradation. Aggradation at the bridge site is a result of the deposition of material eroded from the channel. Degradation at the bridge site is a result of scouring of the channel due to sediment deficit. Only degradation is accounted for in scour calculations. The long-term bed elevation changes are typically based on historical data at the bridge site. The purpose of determining the long-term degradation of a channel is to inform the depth of pier and abutment foundations required for the life span of the bridge. Since degradation is a natural process and not due to the Project, per HEC-23 guidance it is not addressed by countermeasures such as channel hardening.

Three stream measurements were included in the BIRs from the October 2007,July 2016 and July 2024 bridge inspections. The elevation data from the 2019 Project survey and the 1981 FEMA effective study were used along with the stream measurements from the BIRs to assess the long-term bed elevation change at the bridge location. The stream measurements from the BIRs were measured along the top of rail. The channel elevation data from the survey and FEMA effective study referenced NGVD 29, and were adjusted to show the relative distances from the top of rail using an elevation of 1,681.1 ft as the top of rail. Figure 10 shows a comparison of the upstream cross sections at the existing bridge.

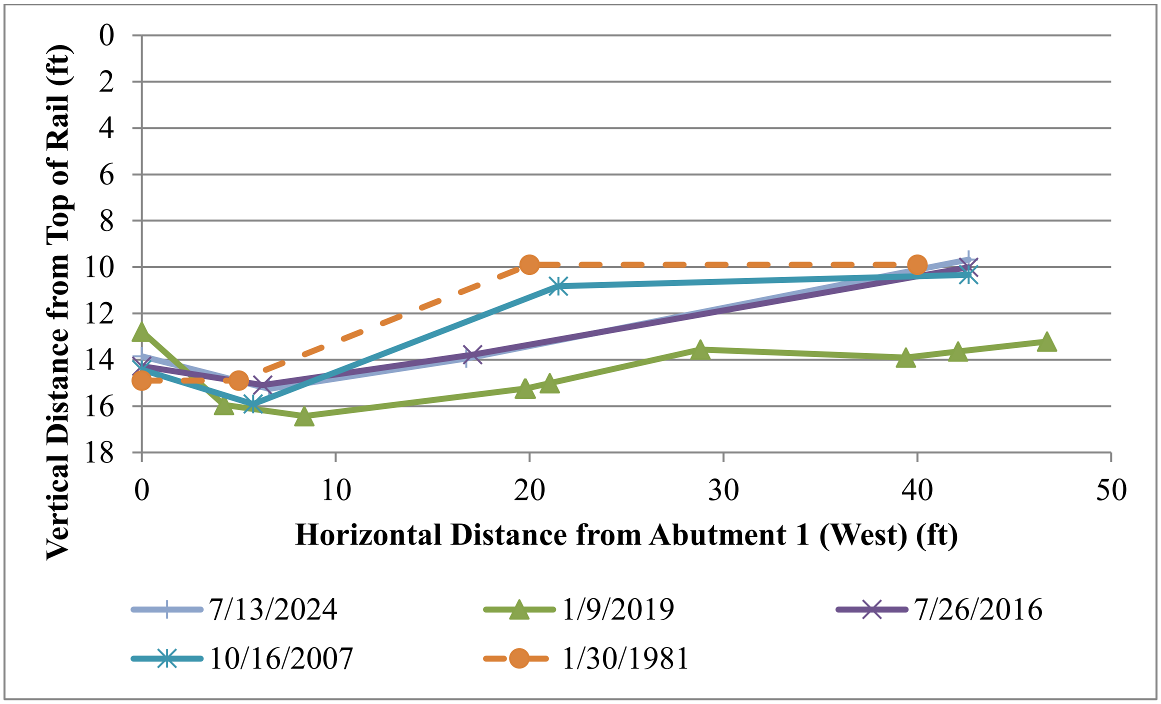

Figure 10. Historical Stream Measurements at Upstream Face of Existing Bridge
Sources: Caltrans (2007, 2016 and 2024), FEMA, 1981, and R.E.Y. Engineers, Inc., 2019

The cross section measurements show a trend of degradation of the channel thalweg. Based on the available historical stream measurements, the channel bed degraded approximately 0.020 ft per year. The long-term bed elevation change in the scour analysis was estimated by taking the average degradation rate and projecting it to a 75-year design life for the proposed replacement bridge. The long-term bed elevation change was estimated to be 1.5 ft.

5.4 Contraction Scour

Contraction scour occurs when the flow area of a stream is reduced by: 1) the natural contraction of the stream channel; 2) a bridge structure; or 3) the overbank flow forced back to the channel. From the continuity equation, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction section, and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases. Thus, the velocity and shear stress decrease until relative equilibrium is reached (i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach, or the bed shear stress is decreased to a value such that no sediment is transported out of the reach). Contraction scour, in a natural channel or at a bridge crossing, involves removal of material from the bed across all or most of the channel width (FHWA, 2012).

Based on the LOTBs for the Hangtown Creek Bridge widening (see Section 5.2), the channel bed material was considered cohesionless for the purposes of calculating scour. For cohesionless soils, contraction scour is classified to be clear-water or live-bed, depending on the stream flow. Live-bed contraction scour occurs when the bed material upstream of the contraction is in motion. Clear-water contraction scour occurs when the bed material is not in motion. To determine whether clear-water or live-bed contraction scour is occurring, the calculated average approach velocity is compared to the threshold velocity at which incipient motion of the bed material is expected (or the critical velocity).

If the critical velocity is greater than the mean channel velocity, then clear-water contraction scour will exist. If the critical velocity is less than the mean channel velocity, then live-bed contraction scour will exist.

The mean approach velocity (7.8 ft/s) exceeded the critical velocity (1.4 ft/s). Therefore, live-bed contraction scour was evaluated at the bridge.

The contraction scour was calculated to be negative. A scour calculation that results in a negative value does not indicate aggradation; it is an indication that there will not be scour. Therefore, the contraction scour is considered to be 0 ft. The calculation of the cohesionless contraction scour for the proposed bridge is presented in Appendix C.

5.5 Pier Scour

Pier scour is caused by vortices (known as a horseshoe vortex) forming at the base of the pier. The horseshoe vortex results from the pileup of water on the upstream surface of the pier and subsequent acceleration of the flow around the base of the pier.

Local scour at piers is a function of bed material characteristics, bed configuration, flow characteristics, fluid properties, and the geometry of the pier and footing. The bed material characteristics can be granular or non-granular, cohesive or cohesionless, erodible or non-erodible rock (FHWA, 2012).

For piers in cohesionless materials, the HEC-18 manual recommends the Colorado State University (CSU) equation to determine pier scour. The pier scour calculation assumes the footings will not be exposed to flow (i.e., that the footings will be below the estimated total scour depth).

The local pier scour was calculated to be 6.9 ft at Pier 2 and 7.0 ft at Pier 3. The calculated local pier scour depths for the proposed bridge are presented in Table 9. The scour calculations are also included in Appendix C.

5.6 Abutment Scour

Abutment scour occurs when the bridge abutments or embankments block approaching flow. According to HEC-18 (FHWA 2012), local scour at bridge abutments is commonly evaluated using either the Froehlich or HIRE live-bed scour equations. The National Cooperative Highway Research Program (NCHRP) developed alternative abutment scour equations based on a range of abutment types, locations, flow conditions, and sediment transport conditions.

While the Froehlich, HIRE, and NCHRP equations are acceptable methods that can be used to estimate scour at abutments, the NCHRP equations have several advantages. Unlike the Froehlich and HIRE equations, the NCHRP equations do not use an effective embankment length, which can be difficult to determine. The NCHRP equations are also considered to be more representative of the physical scouring processes at the abutments. Because of these reasons, the NCHRP method was used to estimate scour at the abutments.

The NCHRP method calculates a scour depth that includes both local and contraction scour components at the abutment. The NCHRP scour was calculated using the equations presented in Section 8.6.3 of HEC-18 (FHWA 2012). The contraction scour depth is subtracted from the calculated scour depth to determine the local abutment scour. The NCHRP method includes three methods for determining a flow depth that includes contraction scour, which is then used to calculate abutment scour. The flow depth was calculated using the live-bed equation. Using the NCHRP method, the local abutment scour was calculated to be 4.1 ft at both abutments. The abutment scour depths for the proposed bridge are presented in Table 9. The scour calculations are also included in Appendix C.

5.7 Total Scour

Total scour is the sum of the long-term bed elevation change (degradation scour), contraction scour, and local scour (see Table 9 for the scour components and the total scour depths).

Table 9. Scour Summary Table

Long Term & Short Term Scour Depths Placerville Drive Bridge over Hangtown Creek Proposed Br. No. 25C0152				
Support No.	**Degradation Scour Depth (ft)**	**Contraction Scour Depth (ft)**	**Short Term (Local) Scour Depth (ft)**	**Total Scour Depth (ft)**
Abut 1	1.5	0	4.1	5.6
Pier 2	1.5	0	6.9	8.4
Pier 3	1.5	0	7.0	8.5
Abut 4	1.5	0	4.1	5.6

According to *Memo to Designers 16-1* (Caltrans, 2017), a scour data table that presents the long-term scour elevations and short-term scour depths, must be placed on the Foundation Plan (see Table 10).

Table 10. Scour Data Table

Support No.	Long Term (Degradation and Contraction) Scour Elevation (ft)	Short Term (Local) Scour Depth (ft)
Abut 1	1,664.0	4.1[1]
Pier 2	1,664.0	6.9[1]
Pier 3	1,664.0	7.0
Abut 4	1,664.0	4.1

Note:
[1]The actual scour will be limited by the presence of bedrock at the support location.

The long-term scour elevation was based upon the long-term bed degradation and contraction scour depths, and the short-term scour depth was based upon the local scour depth. The scour elevations at the piers and abutments were based upon the thalweg elevation of the channel, which is approximately 1,665.6 ft at the proposed bridge.

The bedrock elevations as depicted in the *Draft Foundation Report* are presented in Table 11 (HDR, 2025). The presence of the scour-resistant rock would help limit the potential for scour to occur at Abutment 1 and Pier 2. Based on the report, the bedrock elevation at Pier 3 and Abutment 4 is anticipated to be below the calculated scour elevation.

Table 11. Calculated Scour and Bedrock Elevations

Support No.	Calculated Total Scour Elevation[1] (ft)	Bedrock Elevation (ft)
Abut 1	1,660.0	1,661.7[2]
Pier 2	1,657.1	1,661.7[2]
Pier 3	1,657.0	1,655.4
Abut 4	1,660.0	1,655.4

Notes:
[1]The calculated total scour elevations are based on the channel reference elevation of 1,665.6 ft at the supports.
[2]The actual scour elevation will be limited by the presence of bedrock at the support location.

6 SCOUR AND EROSION COUNTERMEASURES

As stated in Section 5.2, the *Draft Foundation Report* indicated the presence of scour-resistant bedrock at the Project site (HDR, 2025). Per Caltrans standards, abutment foundations should be designed such that they are stable with no lateral support, down to the calculated scour depth (2019). However, RSP is still useful in minimizing bank erosion and protecting the bridge structures from impact and abrasion. RSP is a typical scour and erosion countermeasure installed at bridges and culverts, and it generally consists of rocks on channel and structure boundaries to limit the effect of erosion. It is the most common type of erosion countermeasure due to its general availability, ease of installation, and relatively low cost.

The equations included in Caltrans' HDM (2020a) were used to estimate the weight of the RSP required to protect the proposed bridge abutments.

The RSP diameter was calculated upstream, at the upstream face, at the downstream face, downstream of the proposed bridge, and in the area between the existing upstream culvert and the proposed bridge. As stated in Section 4.3.3, there would be increases in velocities in the area between the existing upstream culvert and the proposed bridge, and RSP could be placed in these areas to address the increases in velocities.

The methodologies from FHWA HEC-23 were also used to calculate the RSP size for the Project. Based on the calculations, RSP Class VI would be recommended at the abutments to protect the embankments from erosion. Class VI RSP has an approximate median diameter of 21 inches and an approximate median weight of 3/8 ton. RSP Class IV would be recommended in area between the upstream culvert and the bridge to address the increases in average channel velocities. Class IV RSP has an approximate median diameter of 15 inches and an approximate median weight of 300 pounds. The detailed calculations are included in Appendix D.

6.1 RSP at Embankment Slopes of Bridge

6.1.1 RSP Sizing

According to the HDM, the minimum layer thickness for Class VI RSP is 3.5 ft (Caltrans, 2020a). A layer of Class III RSP is recommended to be placed beneath the layer of Class VI RSP to help maintain permeability of water and reduce migration of fine particles. Class III RSP has an approximate median diameter of 12 inches and an approximate median weight of 150 pounds. The minimum layer thickness for Class III RSP is 2 ft. Method A or B placement can be used for Class VI RSP, and Method B placement can be used for Class III RSP. Method A places rocks individually by equipment to ensure three-point bearing on adjacent rocks. Method B involves dumping the rock near its planned location and working the rock to its final position with machinery.

6.1.2 RSP Placement

The footprint of application of RSP is based on guidance from HEC-23 and HEC-15. The RSP should extend from the face of the abutment to the toe of the slope and wrap around the bridge abutments to an elevation that is 2 ft above the 100-year design WSE. For this Project, the 100-year WSE is 1,675.7 ft upstream of the bridge and 1,674.7 ft downstream of the bridge. At locations where the elevation that corresponds to 2 ft above the WSE would be above the top of bank, the vertical limit of the RSP can end at the top of bank.

In general, the RSP should extend 25 ft upstream and downstream of the bridge, but because of the impinging flow with the sharp channel bends at the Project, the RSP would need to extend further at two corners of the bridge. At the upstream side of Abutment 1 (the southwest abutment), the RSP should extend to the tangent point of the channel curve. At the downstream side of Abutment 4 (the northeast abutment), the RSP should extend 124 ft beyond the tangent point. At the other two corners of the bridge, the RSP should extend 25 ft from the bridge. A schematic of the RSP extents is shown in Figure 11.

The HEC-23 guideline restricts the application of RSP to bank slopes 1.5:1 (horizontal to vertical) or flatter. The side slopes for this Project will be 2:1 (horizontal to vertical), which will comply with HEC-23. The RSP should be keyed-in a depth of 5 ft.

6.2 RSP at Area Between Upstream Culvert and Bridge

6.2.1 RSP Sizing

According to the HDM, the minimum layer thickness for Class IV RSP is 2.5 ft (Caltrans, 2020a). Method B placement can be used for Class IV RSP.

6.2.2 RSP Placement

The RSP should extend up the slope to an elevation that is 2 ft above the 100-year design WSE. At locations where the elevation that corresponds to 2 ft above the WSE would be above the top of bank, the vertical limit of the RSP can end at the top of bank. The RSP should conform to the limit of the larger rock used for the embankment slopes of the abutments (as described in Section 6.1.2), and extend upstream to the culvert outfall along the south bank. Along the north bank, the footings for Abutment 4 and the retaining wall between Abutment 4 and the outlet of the upstream culvert will embedded into the scour resistant rock. A schematic of the RSP extents is shown in Figure 11.

The HEC-23 guideline restricts the application of RSP to bank slopes 1.5:1 (horizontal to vertical) or flatter. The side slopes for this Project will be 2:1 (horizontal to vertical), which will comply with HEC-23. The RSP should be keyed-in a depth of 5 ft.

6.3 RSP Filter

Class 8 RSP geotextile filter fabric is typically placed on the bank as a separator material between the RSP layers and the channel bank. A gravel (granular) filter can be used as an alternative to geotextile filter fabric. The design of the gravel filter would depend on the underlying base soil parameters as well as the overlaying RSP. The gravel filter is typically between 6 and 15 inches thick.

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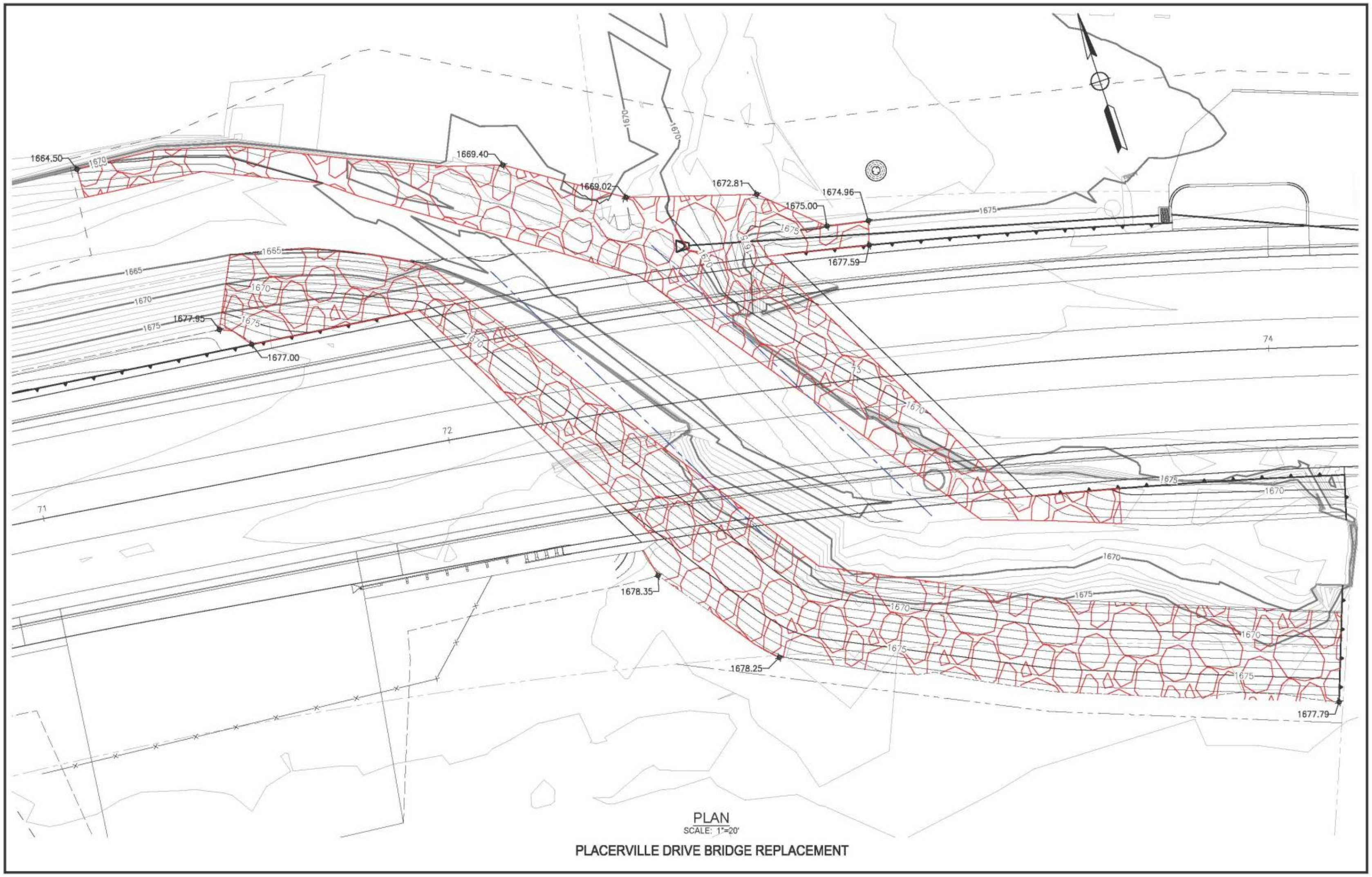

Figure 11. RSP Schematic

Sources: Dewberry and HDR

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Appendix A Existing Condition HEC-RAS Results

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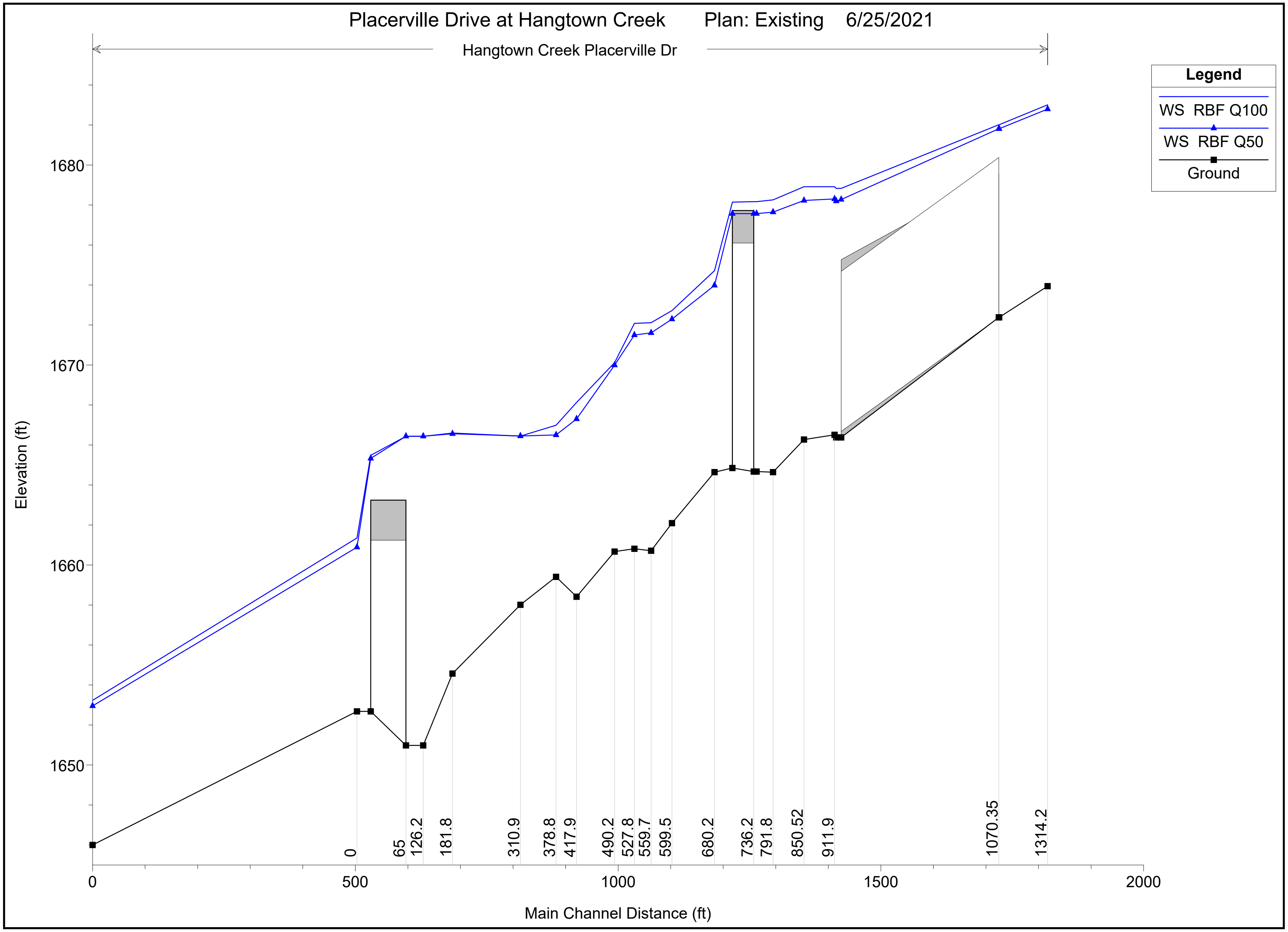
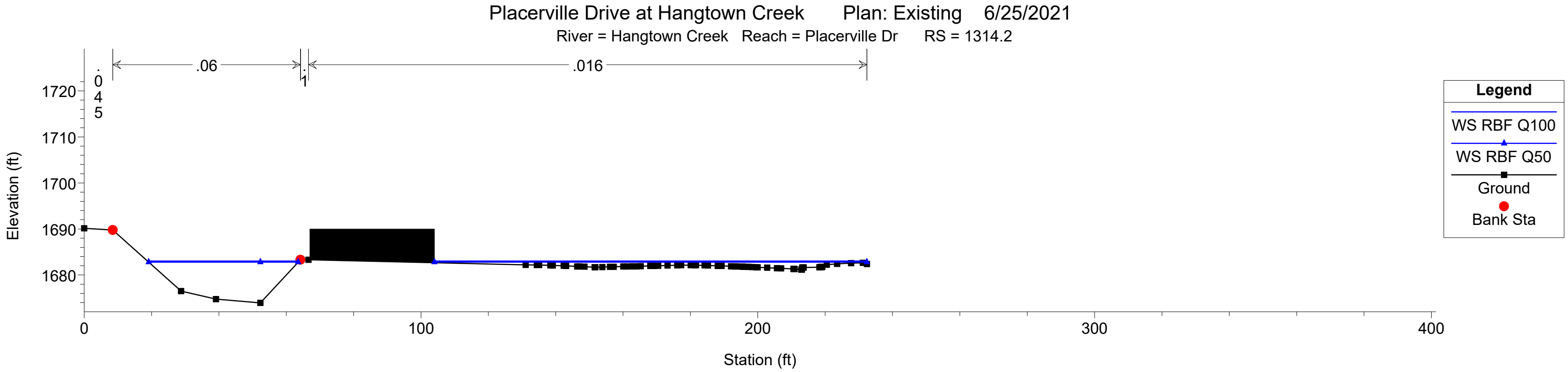

Placerville Drive at Hangtown Creek Plan: Existing 6/25/2021
Hangtown Creek Placerville Dr
Legend
WS RBF Q100
WS RBF Q50
Ground
Elevation (ft)
1650
1660
1670
1680
Main Channel Distance (ft)
0
500
1000
1500
2000
0
65
126.2
181.8
310.9
378.8
417.9
490.2
527.8
559.7
599.5
680.2
736.2
791.8
850.52
911.9
1070.35
1314.2

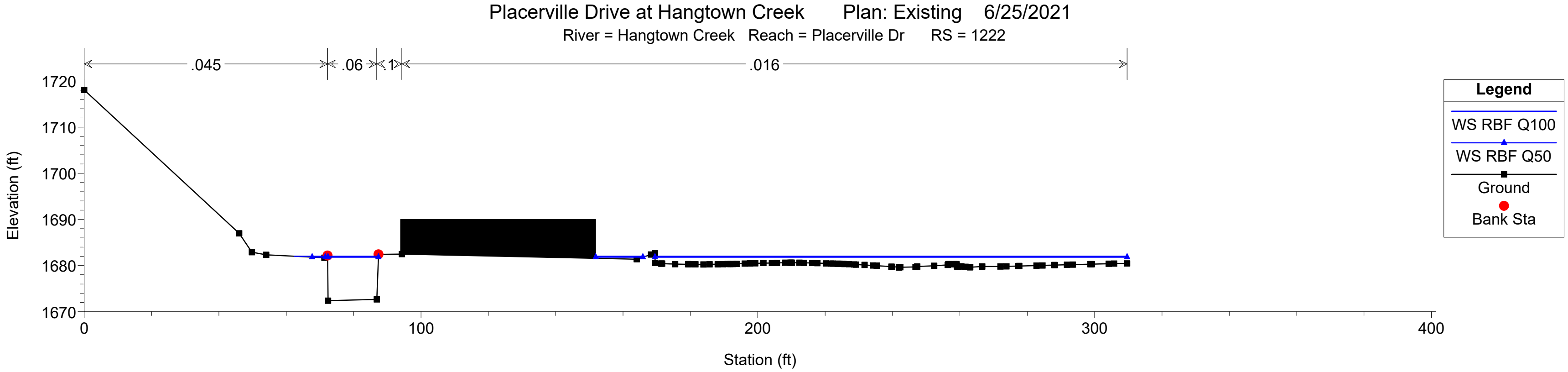
HEC-RAS Plan: Existing River: Hangtown Creek Reach: Placerville Dr

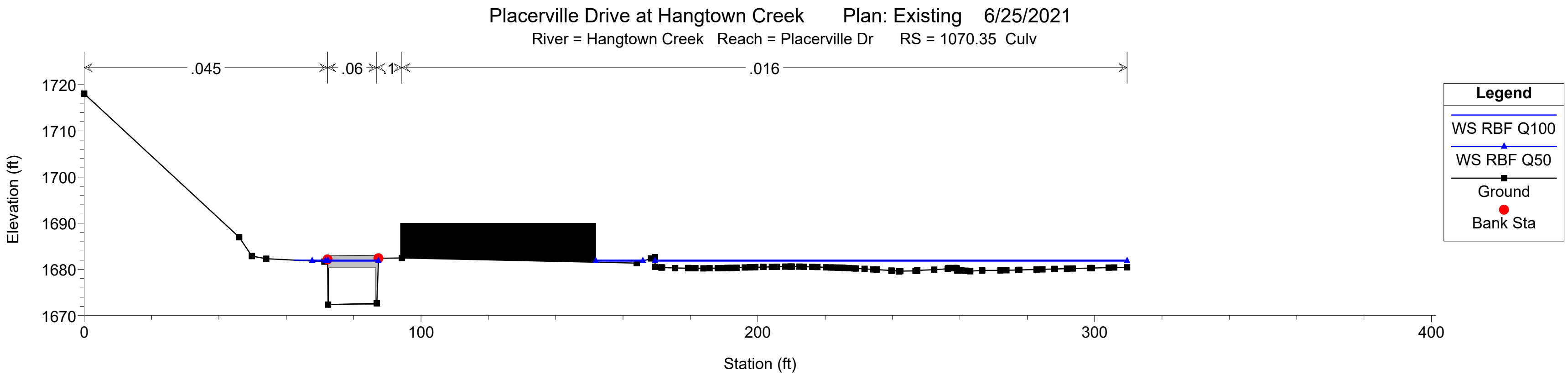
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Placerville Dr	1314.2	RBF Q100	3200.00	1673.94	1683.01	1682.85	1684.02	0.009768	7.60	399.38	173.37	0.54
Placerville Dr	1314.2	RBF Q50	2781.00	1673.94	1682.79	1682.69	1683.71	0.010169	7.65	362.49	172.78	0.55
Placerville Dr	1222	RBF Q100	3200.00	1672.38	1682.01	1682.01	1683.33	0.005470	3.95	406.98	179.69	0.23
Placerville Dr	1222	RBF Q50	2781.00	1672.38	1681.81	1681.81	1682.99	0.005748	4.03	371.56	173.17	0.24
Placerville Dr	1070.35		Culvert									
Placerville Dr	915.1	RBF Q100	3200.00	1666.38	1678.83		1679.43	0.001863	4.58	583.82	181.87	0.29
Placerville Dr	915.1	RBF Q50	2781.00	1666.38	1678.20		1678.81	0.002640	5.17	474.53	163.49	0.33
Placerville Dr	911.9	RBF Q100	3200.00	1666.51	1678.91		1679.34	0.001331	3.65	717.63	188.05	0.21
Placerville Dr	911.9	RBF Q50	2781.00	1666.51	1678.30		1678.70	0.001727	3.98	606.98	172.59	0.23
Placerville Dr	850.52	RBF Q100	3200.00	1666.27	1678.91		1679.20	0.002161	4.21	779.93	230.05	0.26
Placerville Dr	850.52	RBF Q50	2781.00	1666.27	1678.23		1678.54	0.003002	4.69	632.30	201.81	0.30
Placerville Dr	791.8	RBF Q100	3200.00	1664.64	1678.26		1678.97	0.004740	6.87	517.92	166.71	0.40
Placerville Dr	791.8	RBF Q50	2781.00	1664.64	1677.65		1678.30	0.004526	6.48	450.81	81.30	0.38
Placerville Dr	760.4	RBF Q100	3200.00	1664.67	1678.16	1672.74	1678.82	0.004175	6.55	502.05	61.75	0.35
Placerville Dr	760.4	RBF Q50	2781.00	1664.67	1677.57	1672.19	1678.14	0.003959	6.14	465.16	61.75	0.34
Placerville Dr	736.2		Bridge									
Placerville Dr	680.2	RBF Q100	3200.00	1664.64	1674.72		1675.15	0.003685	5.29	640.78	126.00	0.35
Placerville Dr	680.2	RBF Q50	2781.00	1664.64	1673.99		1674.40	0.004015	5.17	554.06	110.46	0.36
Placerville Dr	599.5	RBF Q100	3200.00	1662.09	1672.72	1671.42	1674.29	0.013480	8.80	341.75	61.46	0.61
Placerville Dr	599.5	RBF Q50	2781.00	1662.09	1672.30	1671.10	1673.60	0.013089	8.45	315.76	60.57	0.60
Placerville Dr	559.7	RBF Q100	3200.00	1660.71	1672.12	1671.19	1673.74	0.013984	9.54	321.09	60.03	0.61
Placerville Dr	559.7	RBF Q50	2781.00	1660.71	1671.61	1669.73	1673.04	0.014433	9.44	290.94	58.53	0.62
Placerville Dr	527.8	RBF Q100	3200.00	1660.81	1672.08	1669.78	1673.25	0.009058	8.29	375.20	63.70	0.54
Placerville Dr	527.8	RBF Q50	2781.00	1660.81	1671.50	1669.23	1672.55	0.009532	8.15	338.67	62.85	0.55
Placerville Dr	490.2	RBF Q100	3370.00	1660.67	1670.12	1669.91	1672.54	0.031082	12.48	270.06	51.75	0.96
Placerville Dr	490.2	RBF Q50	2929.00	1660.67	1670.00	1669.30	1671.91	0.024564	11.11	263.73	50.39	0.86

HEC-RAS Plan: Existing River: Hangtown Creek Reach: Placerville Dr (Continued)

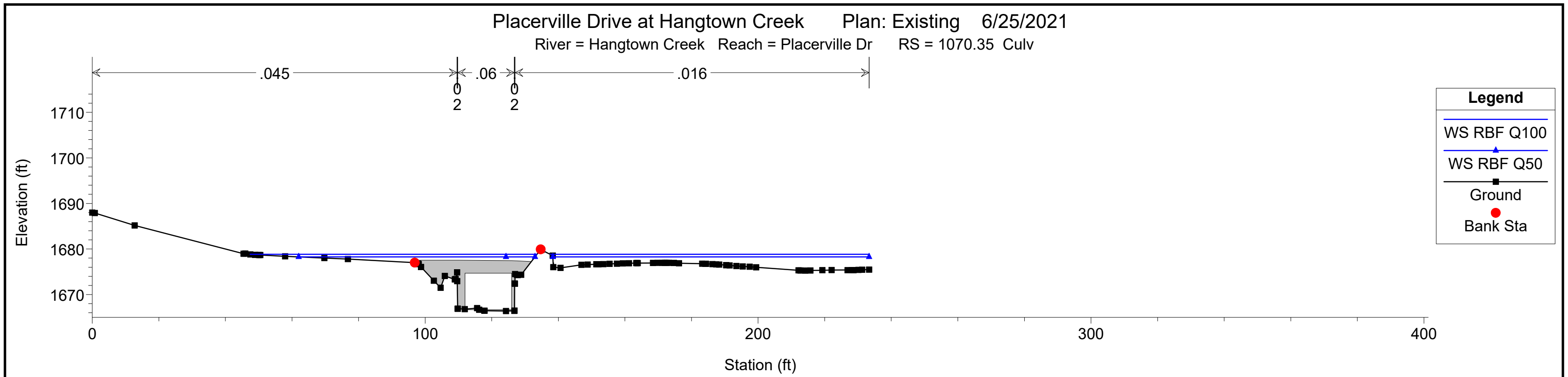
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Placerville Dr	417.9	RBF Q100	3370.00	1658.41	1668.13	1668.13	1670.52	0.025183	12.54	283.52	60.63	0.88
Placerville Dr	417.9	RBF Q50	2929.00	1658.41	1667.30	1667.30	1669.85	0.031377	12.86	235.91	50.57	0.97
Placerville Dr	378.8	RBF Q100	3370.00	1659.41	1666.99	1666.99	1668.74	0.024253	10.71	324.32	95.03	0.87
Placerville Dr	378.8	RBF Q50	2929.00	1659.41	1666.50	1666.50	1668.26	0.027748	10.66	280.93	84.75	0.91
Placerville Dr	310.9	RBF Q100	3370.00	1658.01	1666.44	1664.48	1667.65	0.003668	4.19	534.40	132.46	0.34
Placerville Dr	310.9	RBF Q50	2929.00	1658.01	1666.45	1664.31	1667.36	0.002756	3.64	535.18	132.48	0.29
Placerville Dr	181.8	RBF Q100	3370.00	1654.57	1666.60		1667.09	0.002012	3.97	765.04	127.21	0.26
Placerville Dr	181.8	RBF Q50	2929.00	1654.57	1666.56		1666.93	0.001553	3.47	760.20	127.15	0.23
Placerville Dr	126.2	RBF Q100	3370.00	1650.98	1666.43	1661.38	1666.94	0.003252	5.92	599.94	84.28	0.32
Placerville Dr	126.2	RBF Q50	2929.00	1650.98	1666.44	1660.76	1666.82	0.002451	5.14	600.40	84.29	0.28
Placerville Dr	65		Bridge									
Placerville Dr	0	RBF Q100	3370.00	1652.68	1661.35	1659.78	1662.63	0.014195	9.08	371.18	65.77	0.67
Placerville Dr	0	RBF Q50	2929.00	1652.68	1660.89		1662.03	0.013516	8.58	341.50	63.61	0.65
Placerville Dr	-503	RBF Q100	3370.00	1646.00	1653.23	1652.54	1653.89	0.020039	6.51	517.55	200.55	0.71
Placerville Dr	-503	RBF Q50	2929.00	1646.00	1652.95	1652.27	1653.57	0.020016	6.31	463.89	187.99	0.71

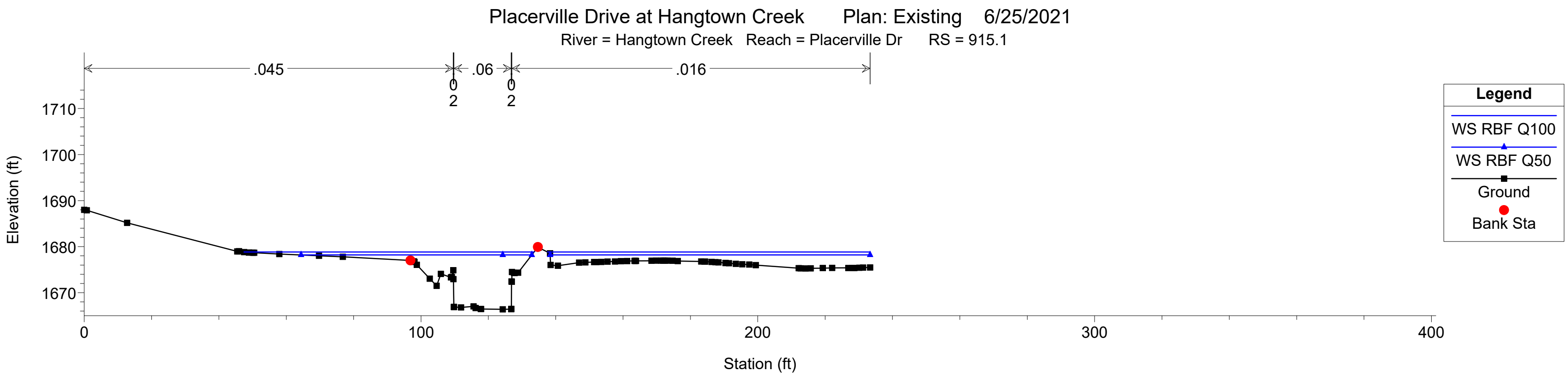


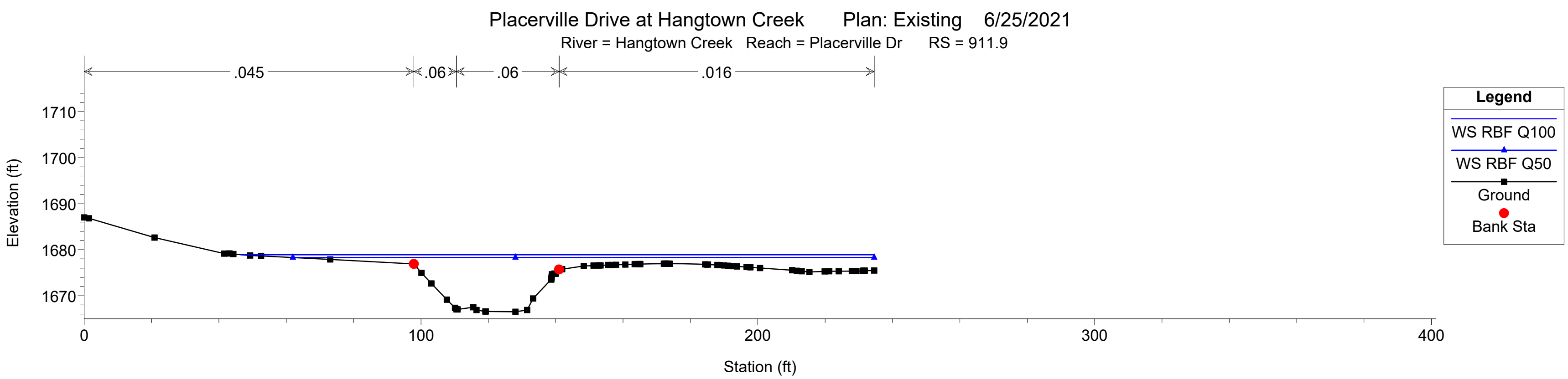




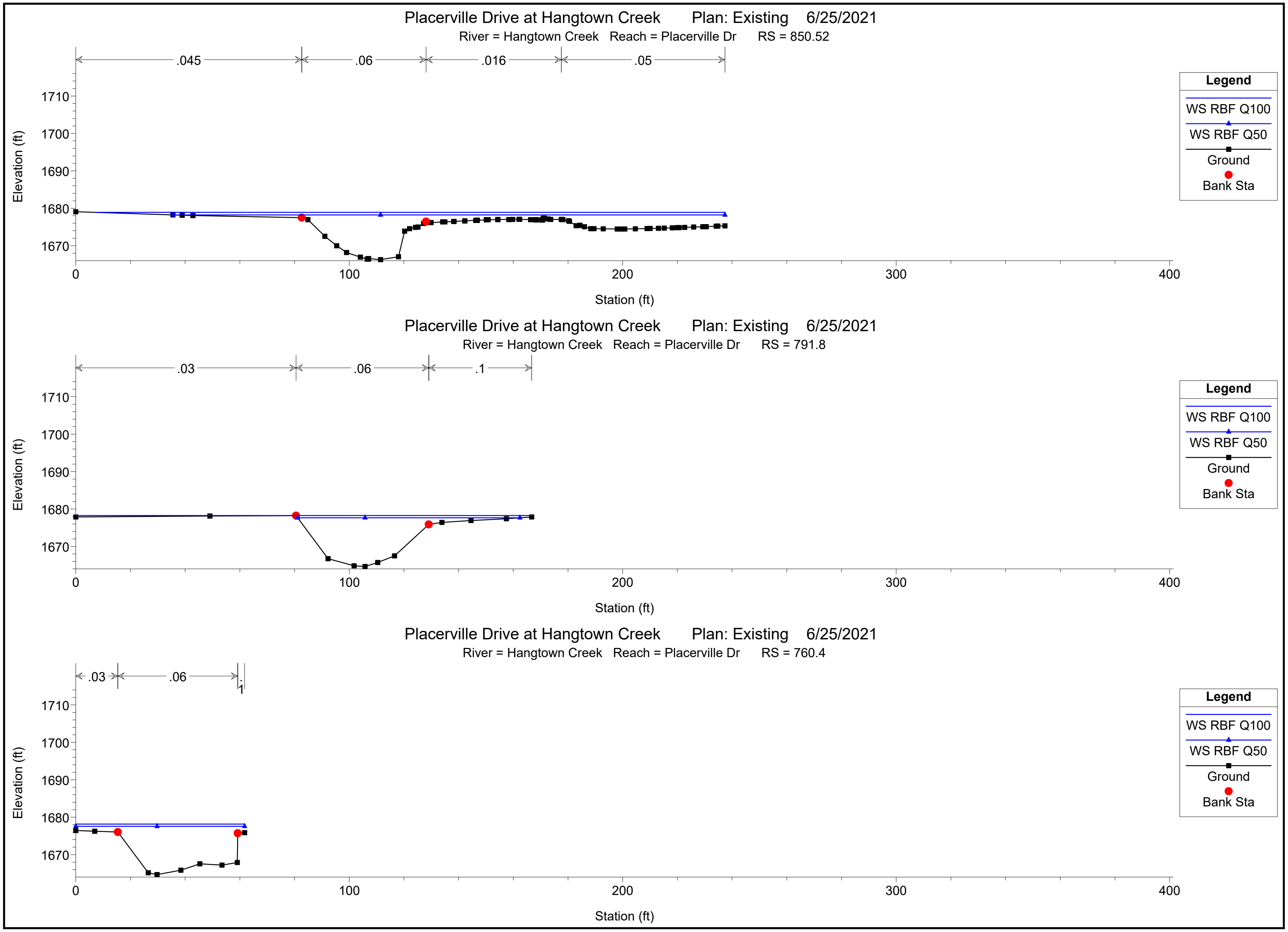
1 in Horiz. = 52 ft 1 in Vert. = 38 ft



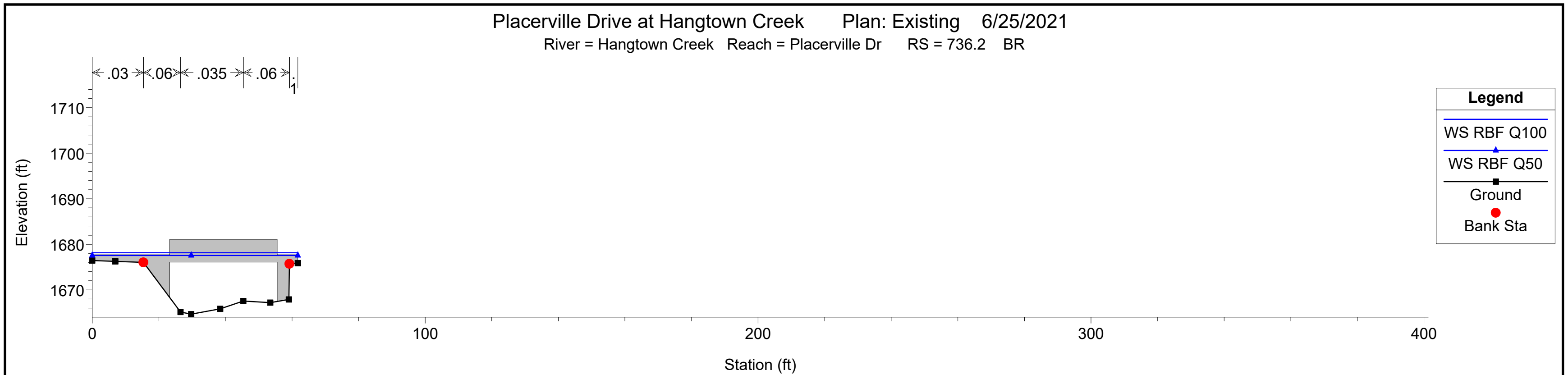


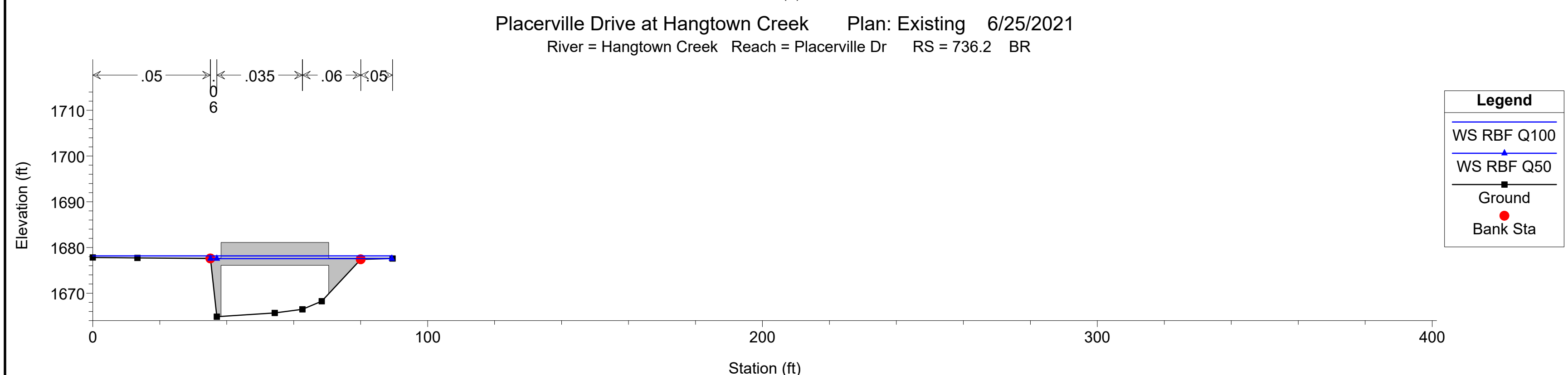


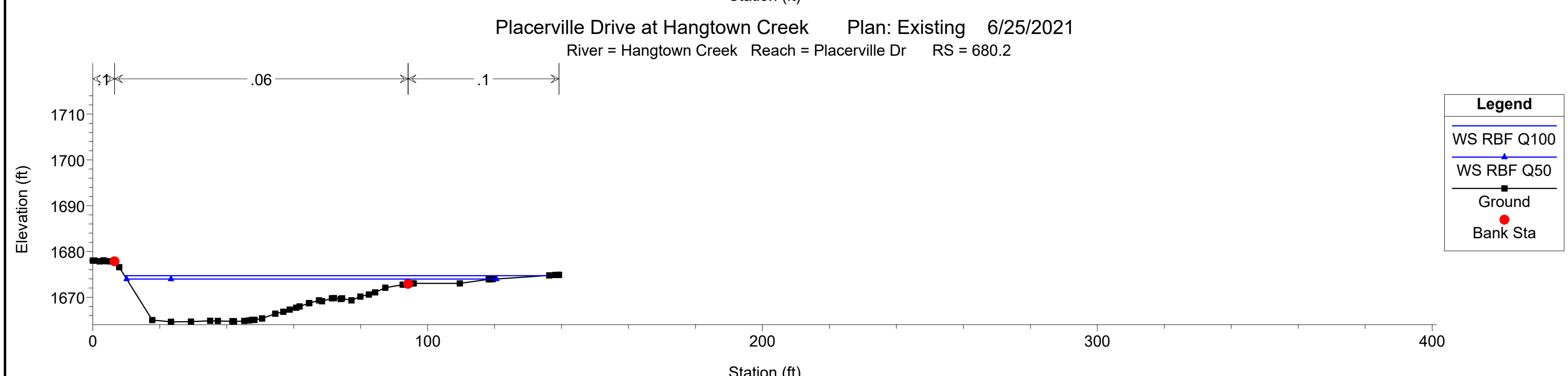
1 in Horiz. = 52 ft 1 in Vert. = 38 ft



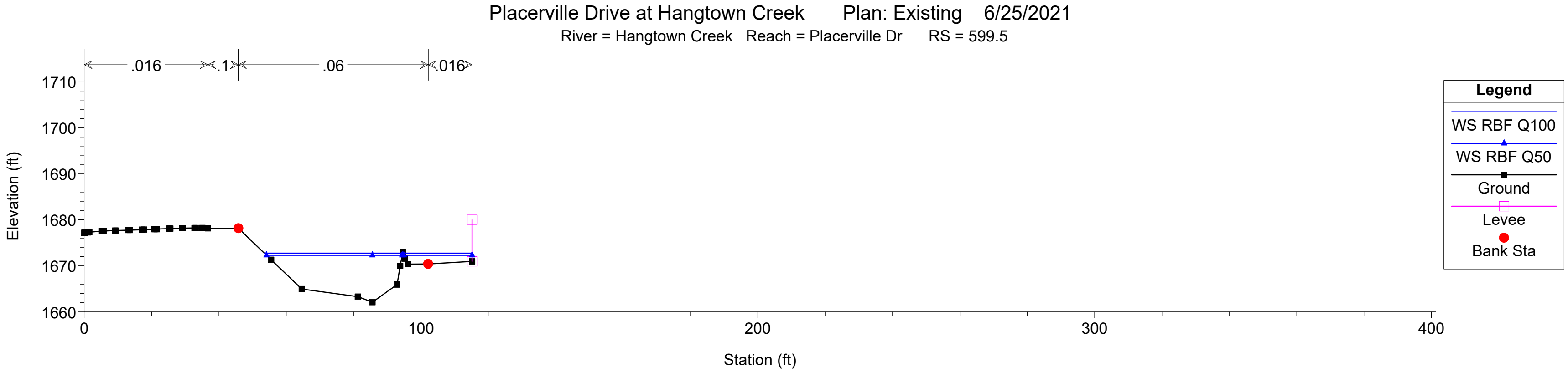
1 in Horiz. = 52 ft 1 in Vert. = 38 ft

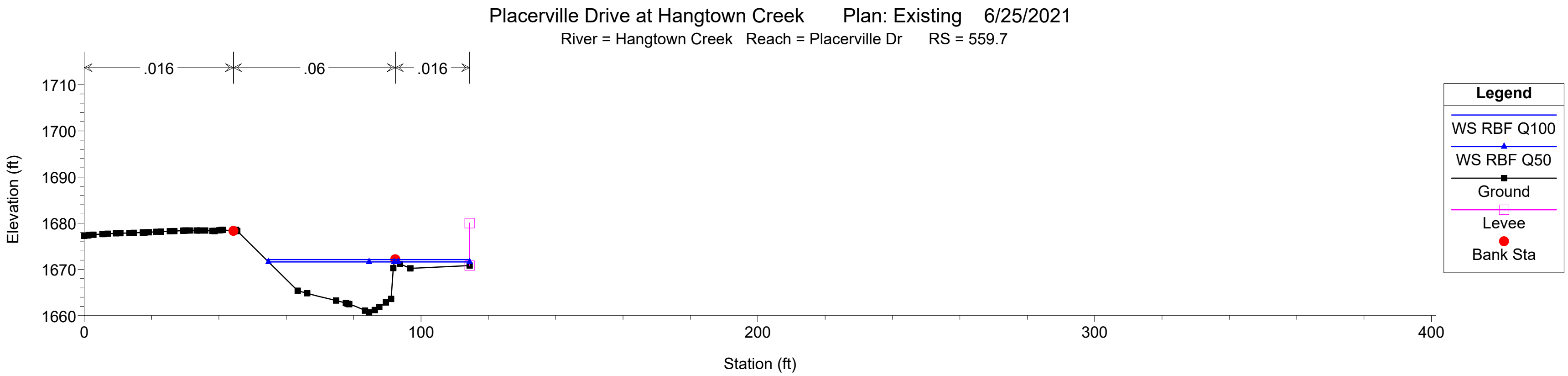


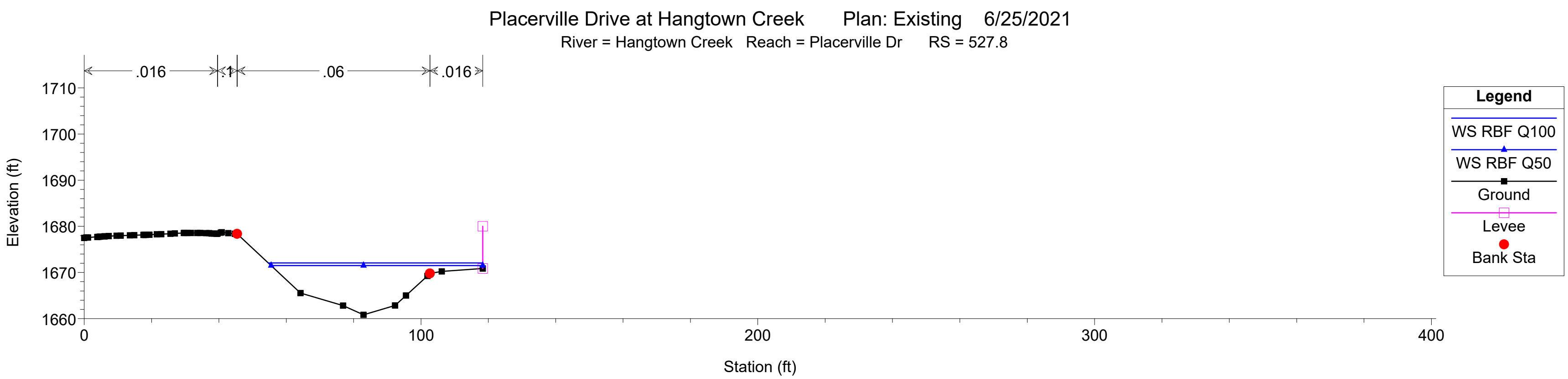




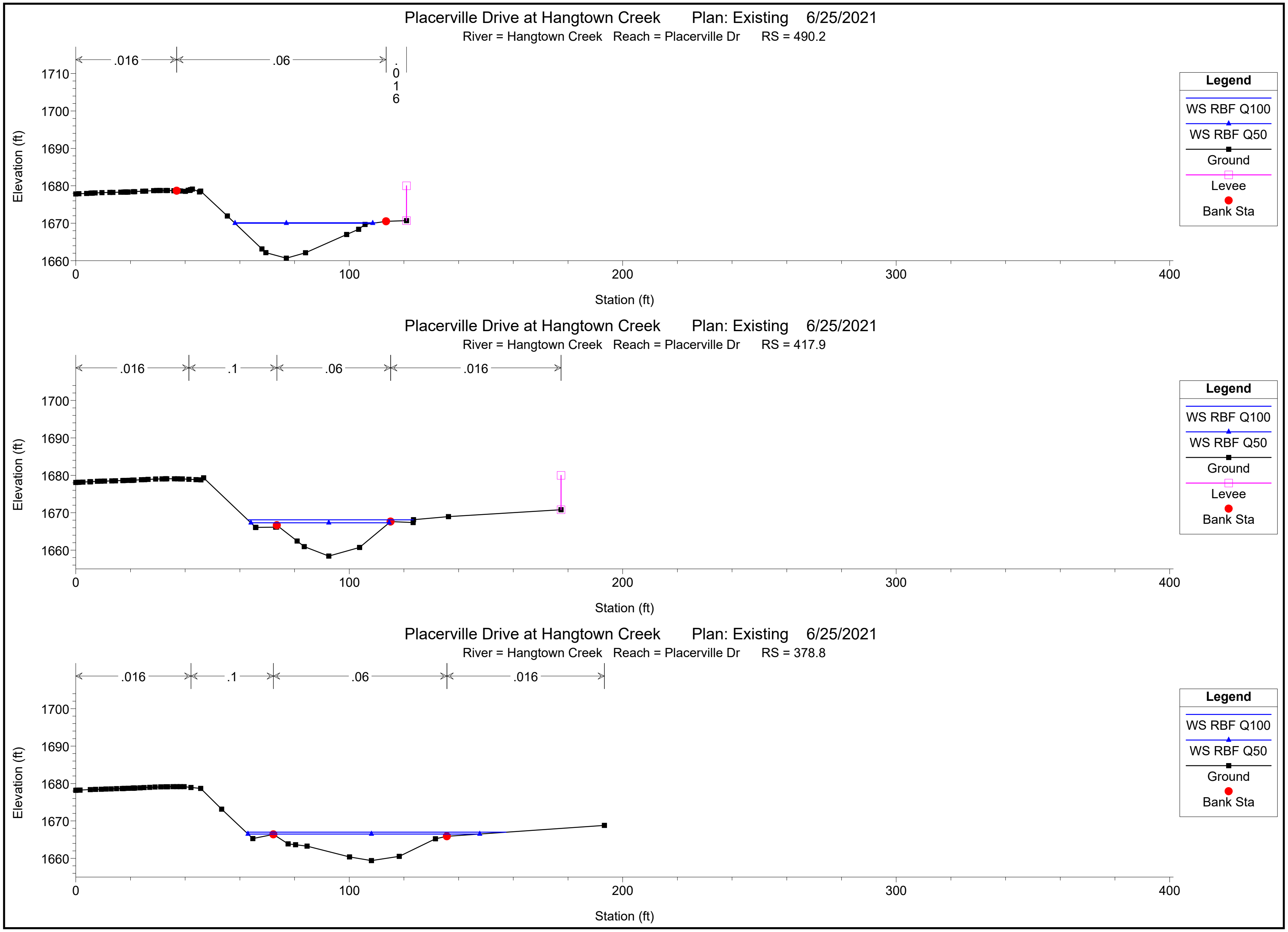
1 in Horiz. = 52 ft 1 in Vert. = 38 ft



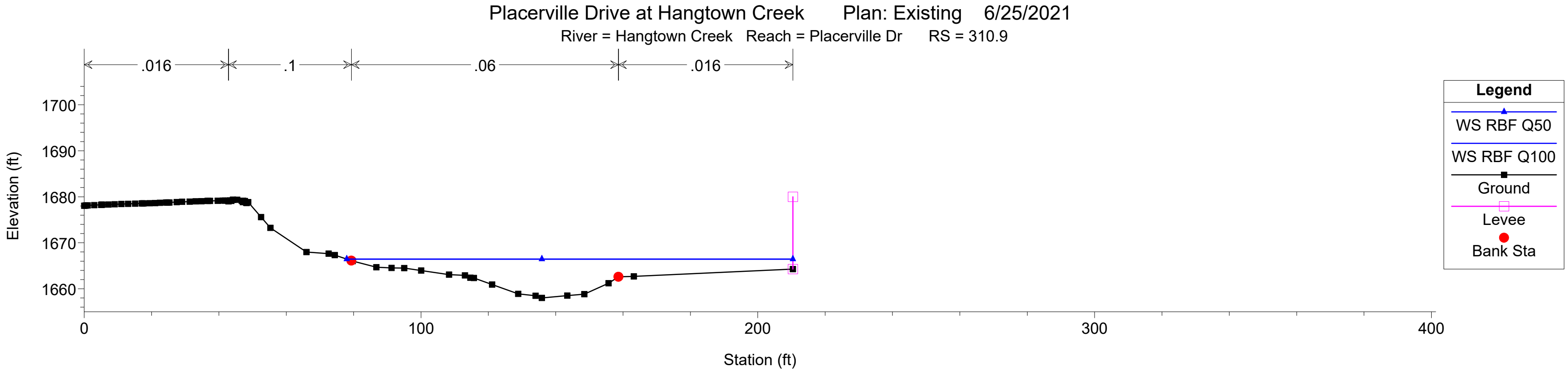


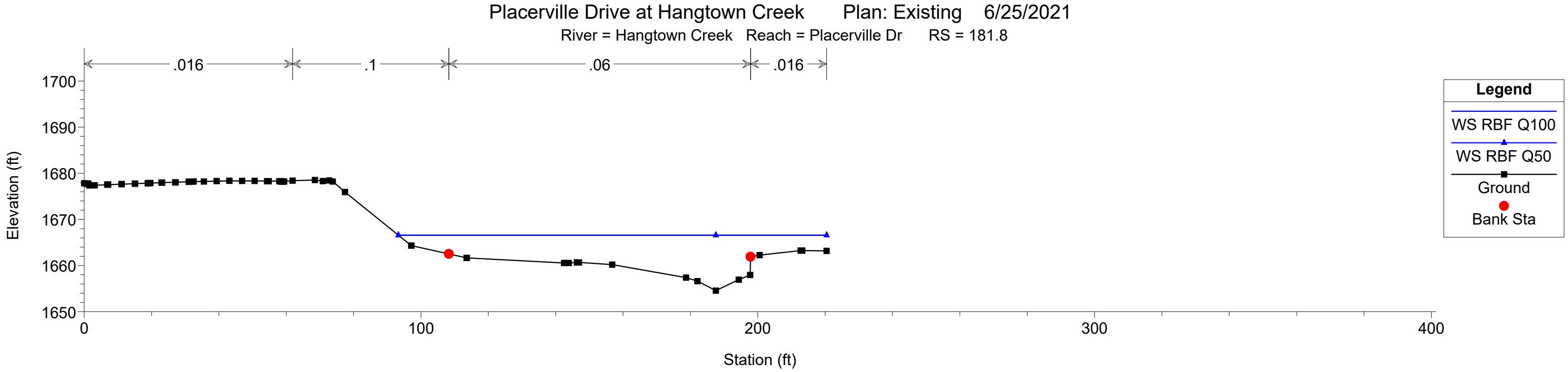


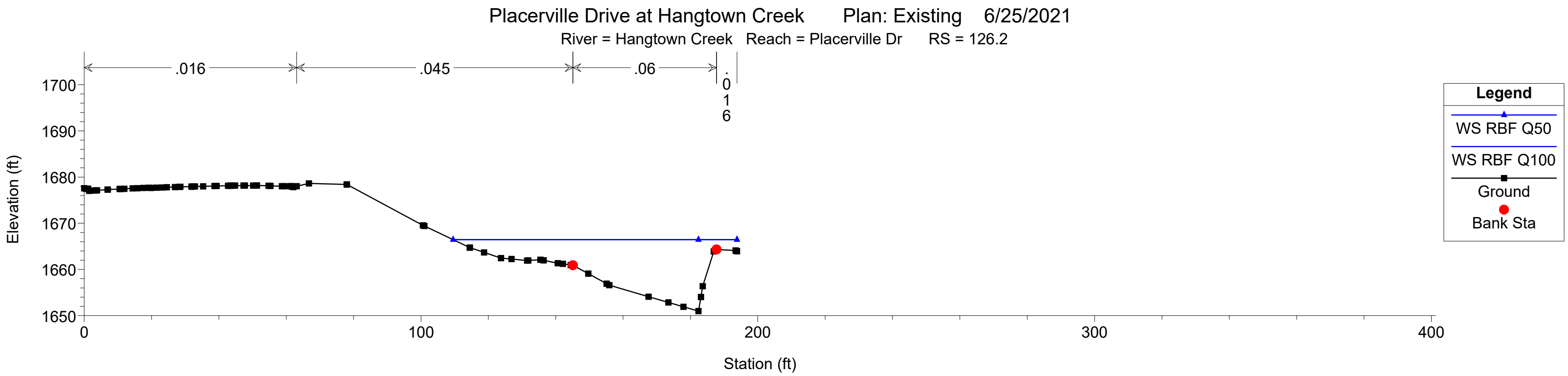
1 in Horiz. = 52 ft 1 in Vert. = 38 ft



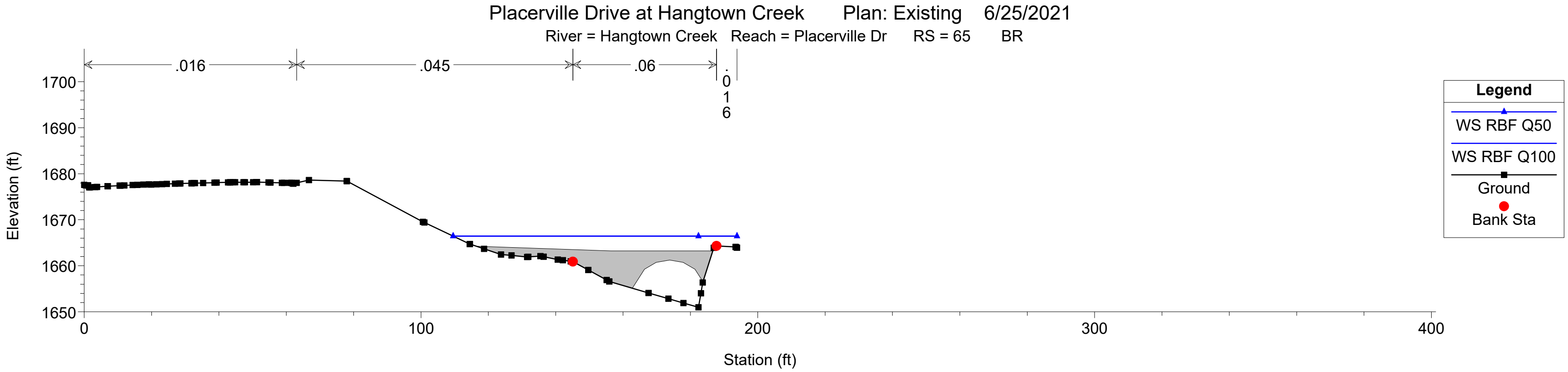
1 in Horiz. = 52 ft 1 in Vert. = 38 ft

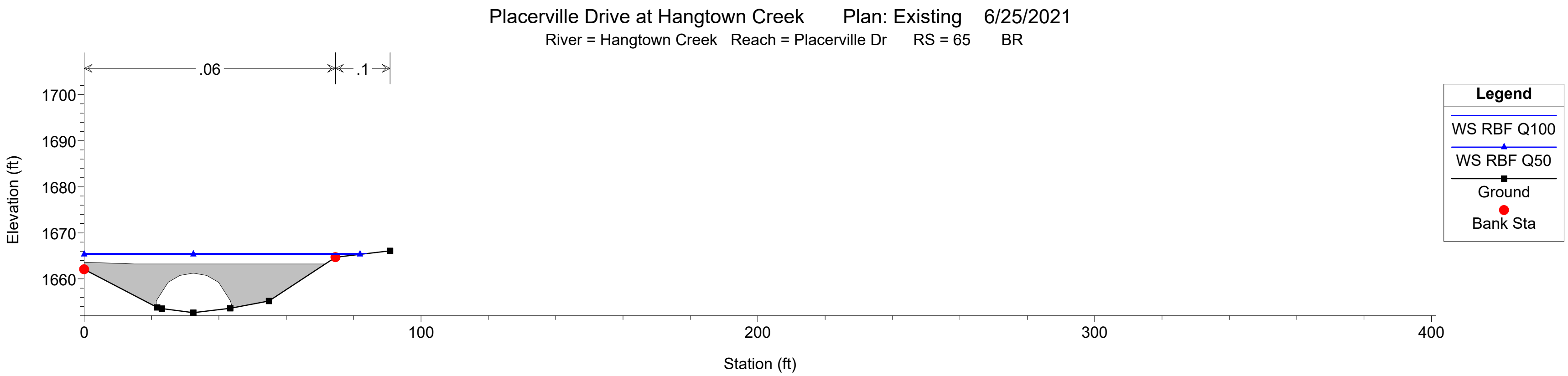


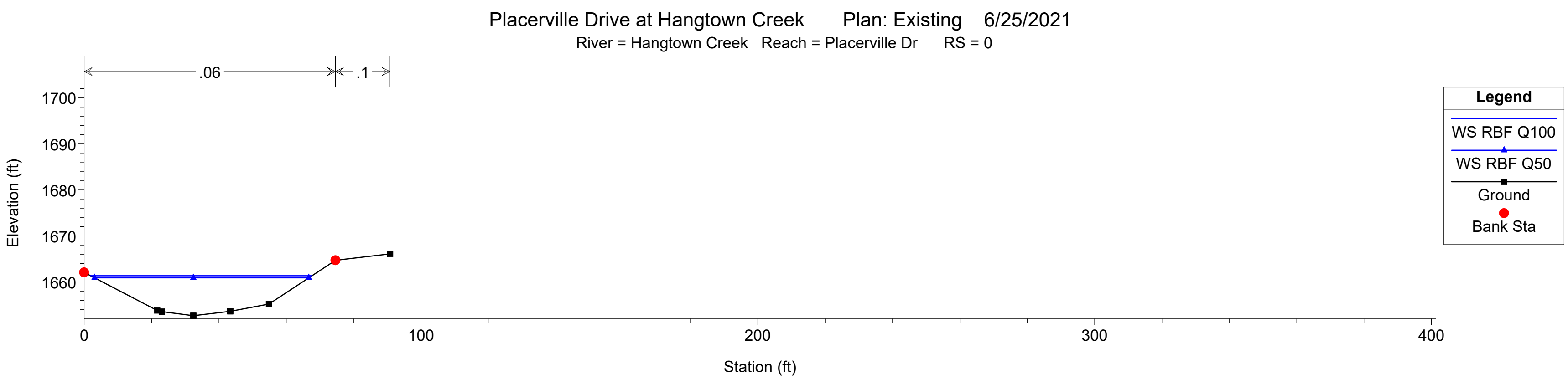




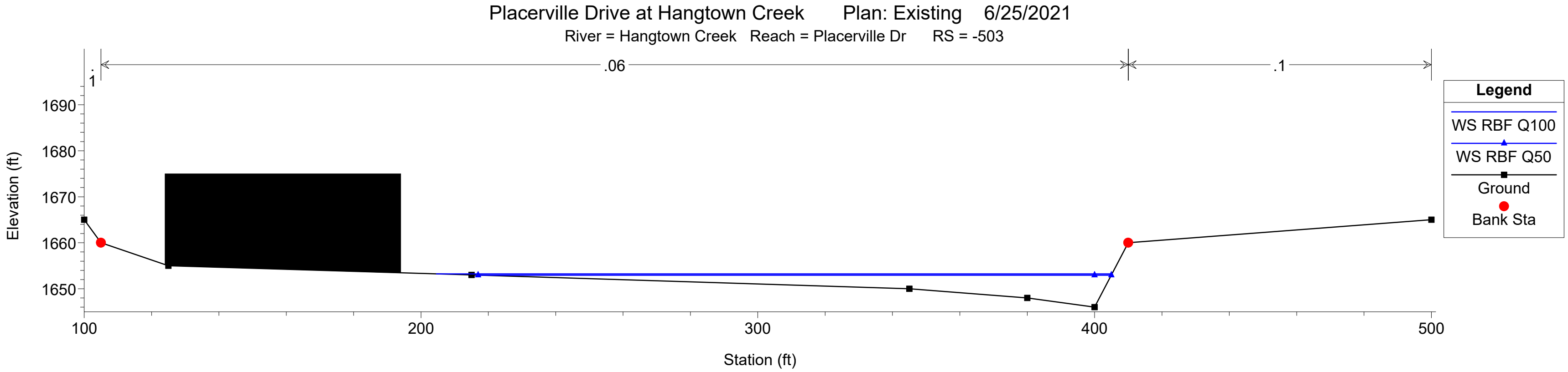
1 in Horiz. = 52 ft 1 in Vert. = 38 ft







1 in Horiz. = 52 ft 1 in Vert. = 38 ft



1 in Horiz. = 52 ft 1 in Vert. = 38 ft

Appendix B Proposed Condition HEC-RAS Results

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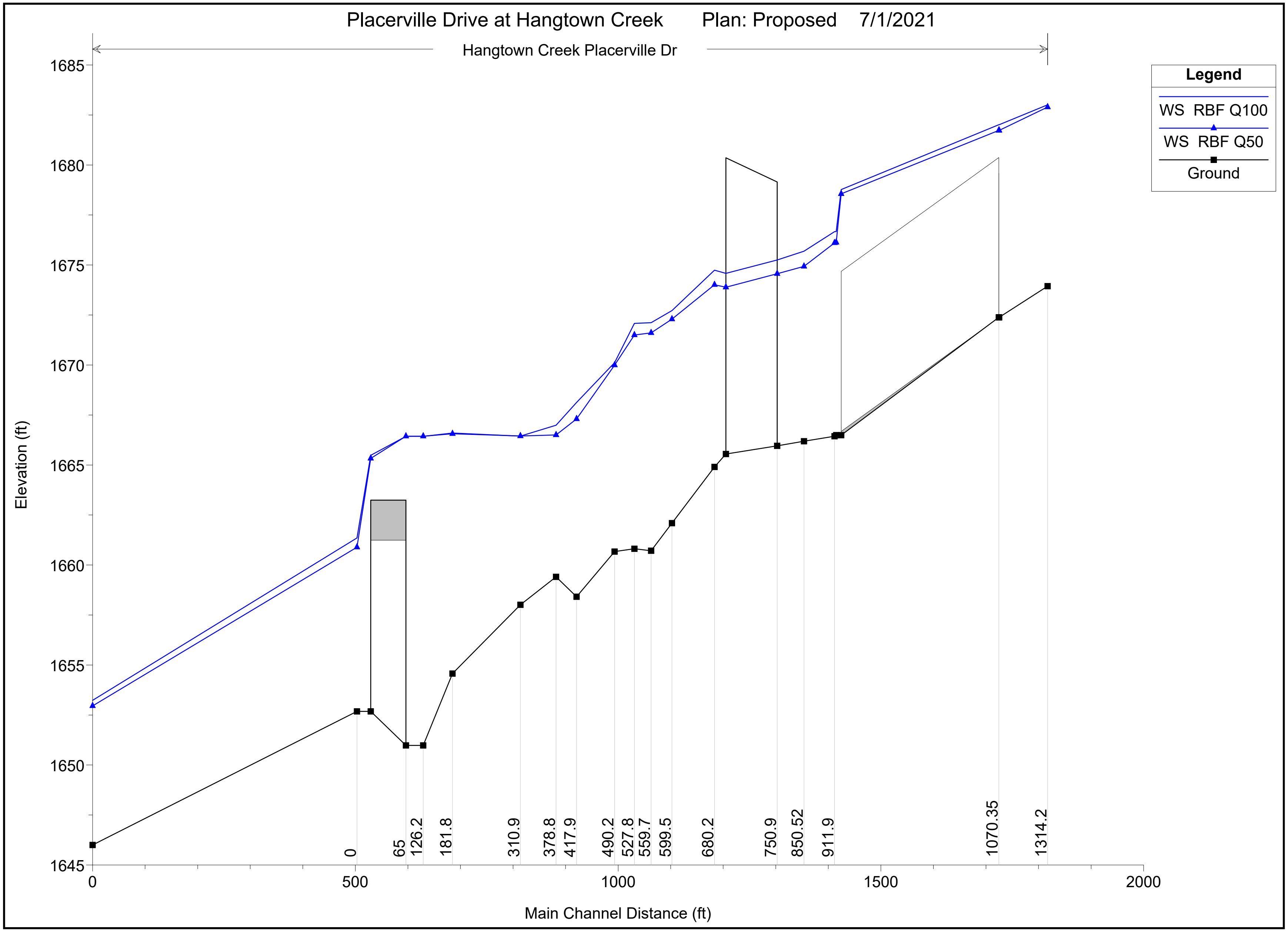


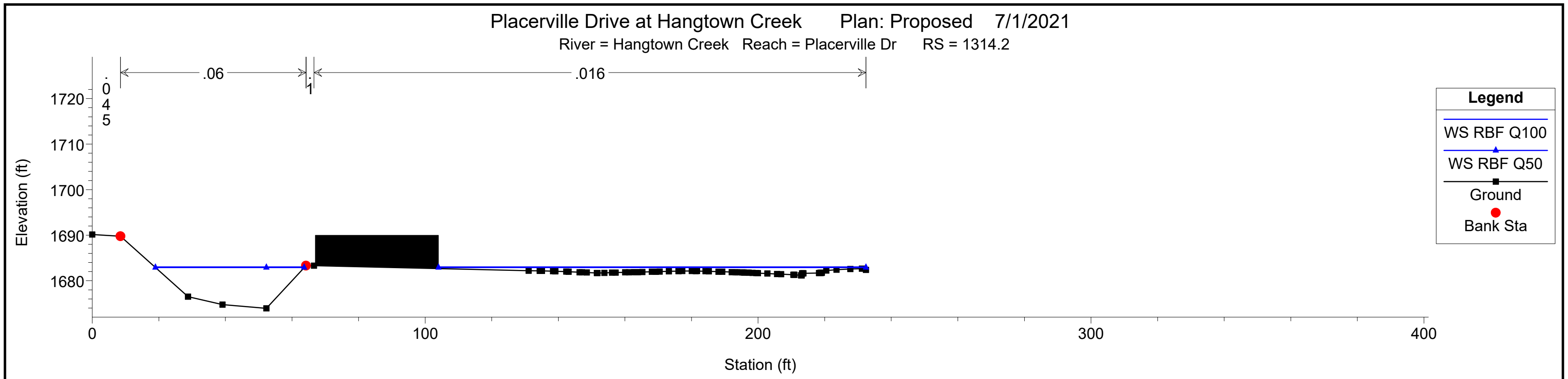
Placerville Drive at Hangtown Creek Plan: Proposed 7/1/2021
Hangtown Creek Placerville Dr
Legend
WS RBF Q100
WS RBF Q50
Ground
Elevation (ft)
1685
1680
1675
1670
1665
1660
1655
1650
1645
Main Channel Distance (ft)
0
500
1000
1500
2000
0
65
126.2
181.8
310.9
378.8
417.9
490.2
527.8
559.7
599.5
680.2
750.9
850.52
911.9
1070.35
1314.2

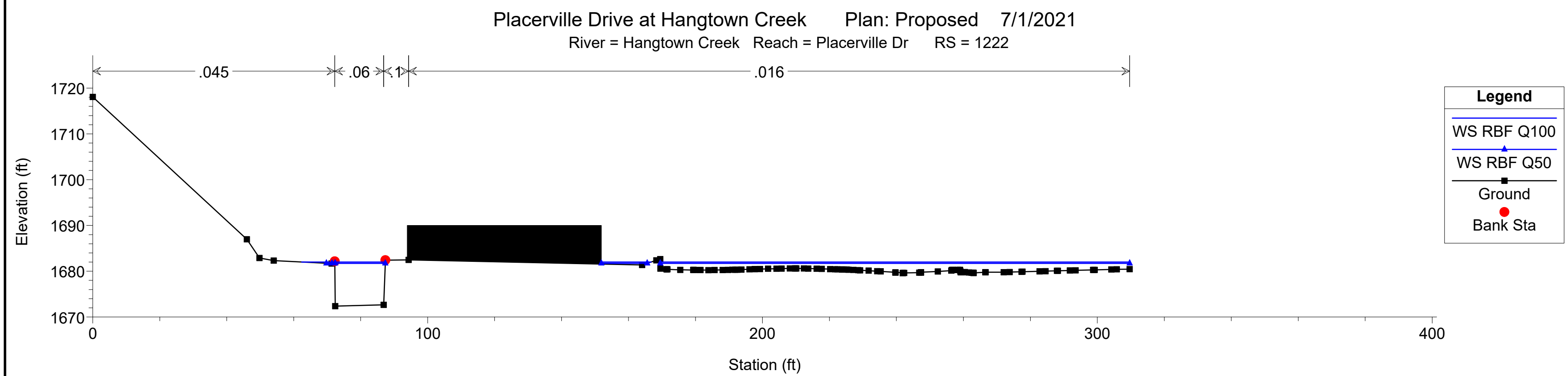
HEC-RAS Plan: Proposed River: Hangtown Creek Reach: Placerville Dr

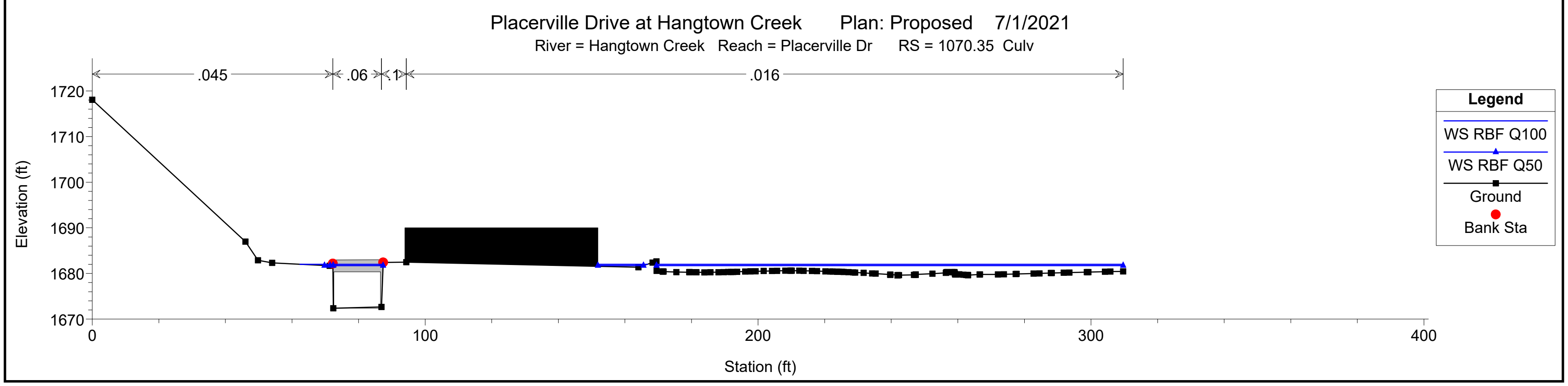
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Placerville Dr	1314.2	RBF Q100	3200.00	1673.94	1683.01	1682.85	1684.02	0.009784	7.61	399.19	173.37	0.54
Placerville Dr	1314.2	RBF Q50	2781.00	1673.94	1682.89	1682.72	1683.73	0.008747	7.14	379.68	173.06	0.51
Placerville Dr	1222	RBF Q100	3200.00	1672.38	1682.01	1682.01	1683.33	0.005462	3.95	407.16	179.72	0.23
Placerville Dr	1222	RBF Q50	2781.00	1672.38	1681.73	1681.73	1682.99	0.006594	4.31	358.00	170.62	0.25
Placerville Dr	1070.35		Culvert									
Placerville Dr	915.1	RBF Q100	3200.00	1666.49	1676.67		1677.63	0.003168	7.89	411.65	70.28	0.50
Placerville Dr	915.1	RBF Q50	2781.00	1666.49	1676.13		1676.99	0.003056	7.44	376.34	59.63	0.49
Placerville Dr	911.9	RBF Q100	3200.00	1666.44	1676.67		1677.61	0.007584	7.83	411.99	71.23	0.50
Placerville Dr	911.9	RBF Q50	2781.00	1666.44	1676.10		1676.97	0.007421	7.47	374.00	62.23	0.49
Placerville Dr	850.52	RBF Q100	3200.00	1666.19	1675.69	1673.58	1677.00	0.013394	9.38	374.11	99.98	0.61
Placerville Dr	850.52	RBF Q50	2781.00	1666.19	1674.93	1673.01	1676.32	0.014920	9.49	302.69	80.57	0.64
Placerville Dr	750.9		Bridge									
Placerville Dr	680.2	RBF Q100	3200.00	1664.90	1674.74		1675.20	0.004394	5.48	620.34	132.05	0.38
Placerville Dr	680.2	RBF Q50	2781.00	1664.90	1674.01		1674.47	0.005008	5.43	529.51	117.24	0.40
Placerville Dr	599.5	RBF Q100	3200.00	1662.09	1672.72	1671.42	1674.29	0.013477	8.80	341.77	61.46	0.61
Placerville Dr	599.5	RBF Q50	2781.00	1662.09	1672.30	1671.11	1673.60	0.013088	8.45	315.76	60.57	0.60
Placerville Dr	559.7	RBF Q100	3200.00	1660.71	1672.12	1671.22	1673.74	0.013986	9.54	321.07	60.02	0.61
Placerville Dr	559.7	RBF Q50	2781.00	1660.71	1671.61	1669.75	1673.04	0.014435	9.44	290.92	58.53	0.62
Placerville Dr	527.8	RBF Q100	3200.00	1660.81	1672.08	1669.77	1673.25	0.009058	8.29	375.20	63.70	0.54
Placerville Dr	527.8	RBF Q50	2781.00	1660.81	1671.50	1669.22	1672.55	0.009532	8.15	338.67	62.85	0.55
Placerville Dr	490.2	RBF Q100	3370.00	1660.67	1670.12	1669.92	1672.54	0.031082	12.48	270.06	51.75	0.96
Placerville Dr	490.2	RBF Q50	2929.00	1660.67	1670.00	1669.30	1671.91	0.024564	11.11	263.73	50.39	0.86
Placerville Dr	417.9	RBF Q100	3370.00	1658.41	1668.13	1668.13	1670.52	0.025183	12.54	283.52	60.63	0.88
Placerville Dr	417.9	RBF Q50	2929.00	1658.41	1667.30	1667.30	1669.85	0.031377	12.86	235.91	50.57	0.97
Placerville Dr	378.8	RBF Q100	3370.00	1659.41	1666.99	1666.99	1668.74	0.024253	10.71	324.32	95.03	0.87
Placerville Dr	378.8	RBF Q50	2929.00	1659.41	1666.50	1666.50	1668.26	0.027748	10.66	280.93	84.75	0.91

HEC-RAS Plan: Proposed River: Hangtown Creek Reach: Placerville Dr (Continued)

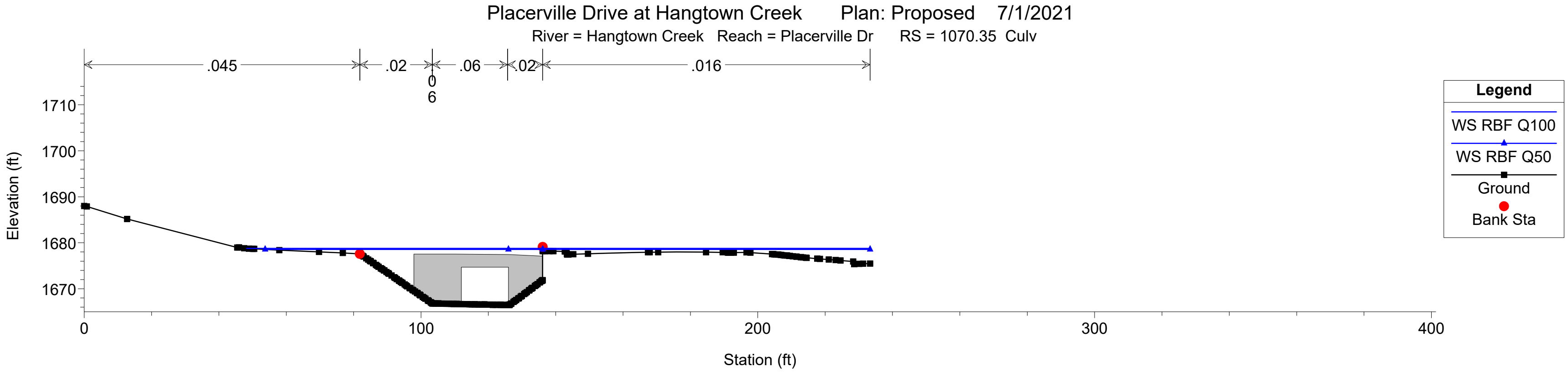
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Placerville Dr	310.9	RBF Q100	3370.00	1658.01	1666.44	1664.48	1667.65	0.003668	4.19	534.40	132.46	0.34
Placerville Dr	310.9	RBF Q50	2929.00	1658.01	1666.45	1664.31	1667.36	0.002756	3.64	535.18	132.48	0.29
Placerville Dr	181.8	RBF Q100	3370.00	1654.57	1666.60		1667.09	0.002012	3.97	765.04	127.21	0.26
Placerville Dr	181.8	RBF Q50	2929.00	1654.57	1666.56		1666.93	0.001553	3.47	760.20	127.15	0.23
Placerville Dr	126.2	RBF Q100	3370.00	1650.98	1666.43	1661.38	1666.94	0.003252	5.92	599.94	84.28	0.32
Placerville Dr	126.2	RBF Q50	2929.00	1650.98	1666.44	1660.76	1666.82	0.002451	5.14	600.40	84.29	0.28
Placerville Dr	65		Bridge									
Placerville Dr	0	RBF Q100	3370.00	1652.68	1661.35	1659.78	1662.63	0.014195	9.08	371.18	65.77	0.67
Placerville Dr	0	RBF Q50	2929.00	1652.68	1660.89		1662.03	0.013516	8.58	341.50	63.61	0.65
Placerville Dr	-503	RBF Q100	3370.00	1646.00	1653.23	1652.54	1653.89	0.020039	6.51	517.55	200.55	0.71
Placerville Dr	-503	RBF Q50	2929.00	1646.00	1652.95	1652.28	1653.57	0.020016	6.31	463.89	187.99	0.71

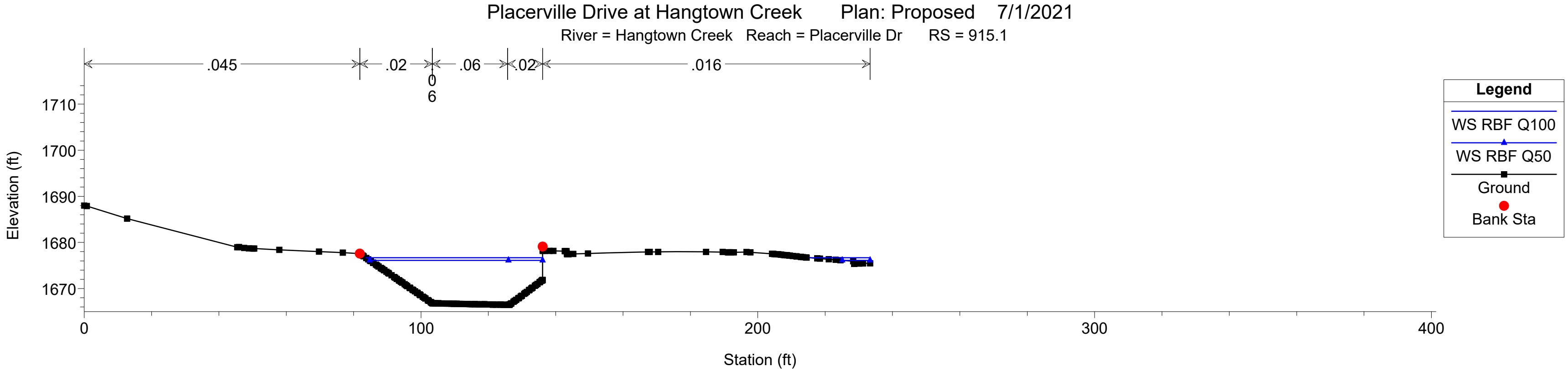


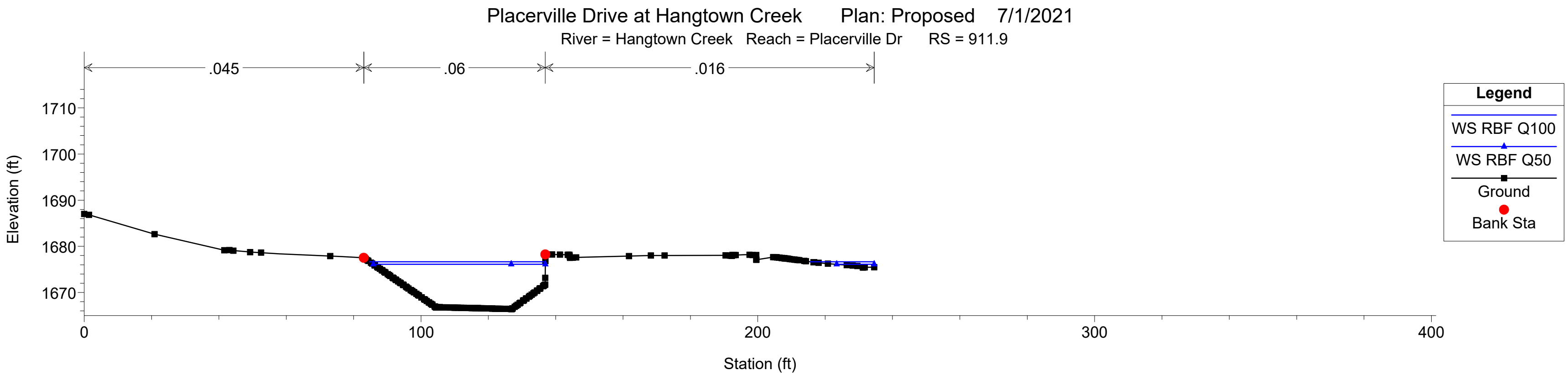




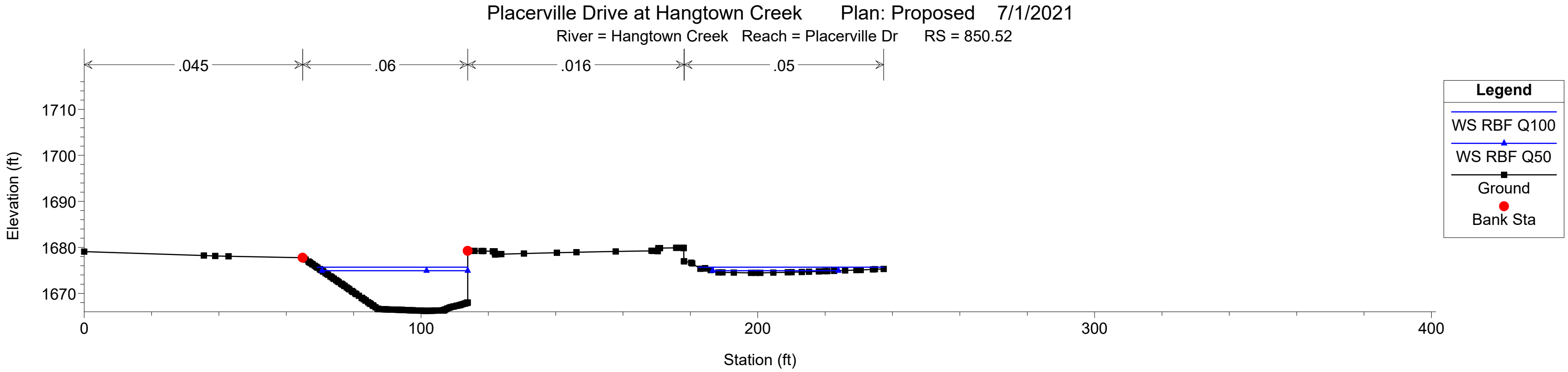
1 in Horiz. = 52 ft 1 in Vert. = 38 ft

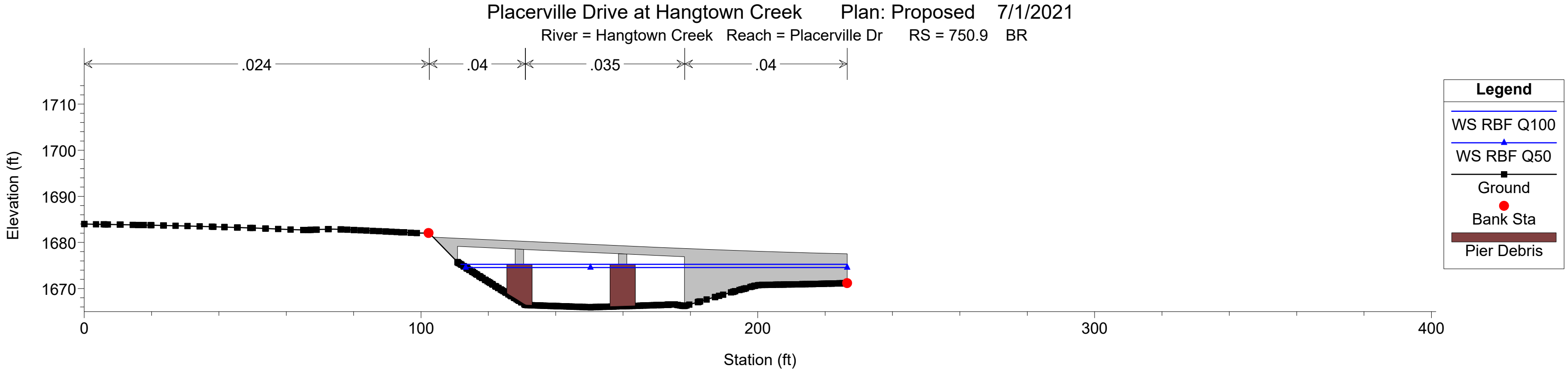


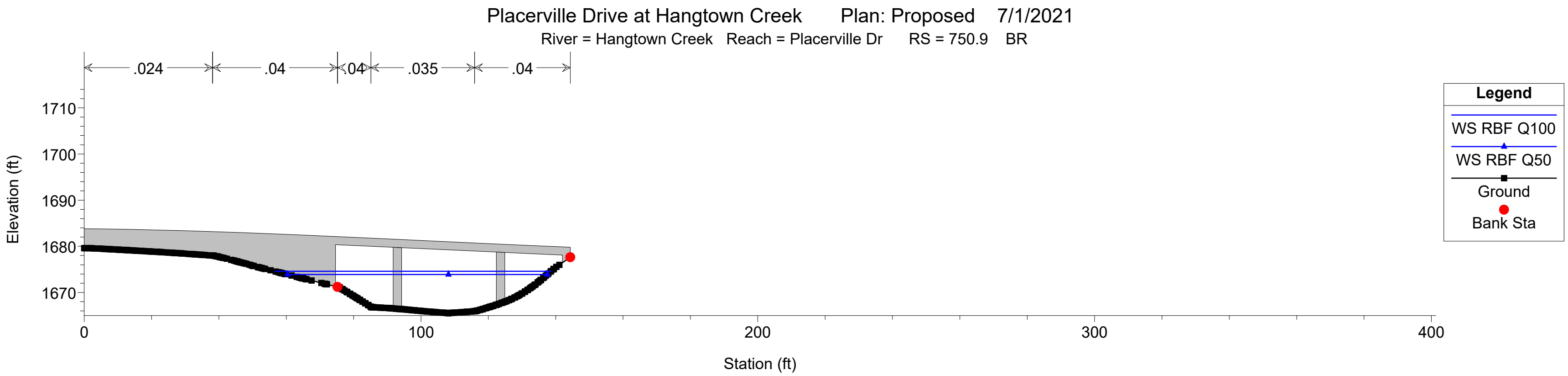




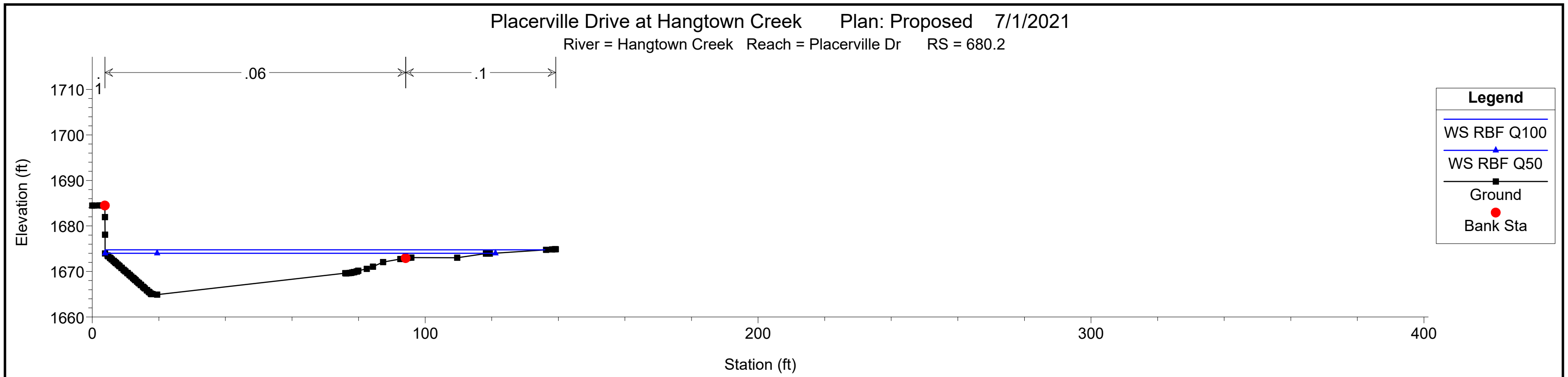
1 in Horiz. = 52 ft 1 in Vert. = 38 ft

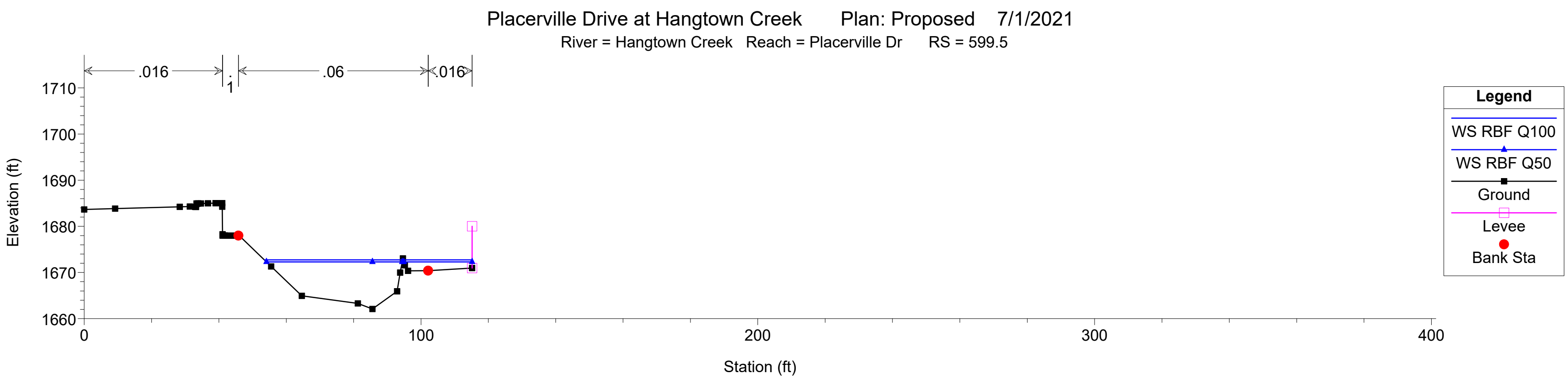


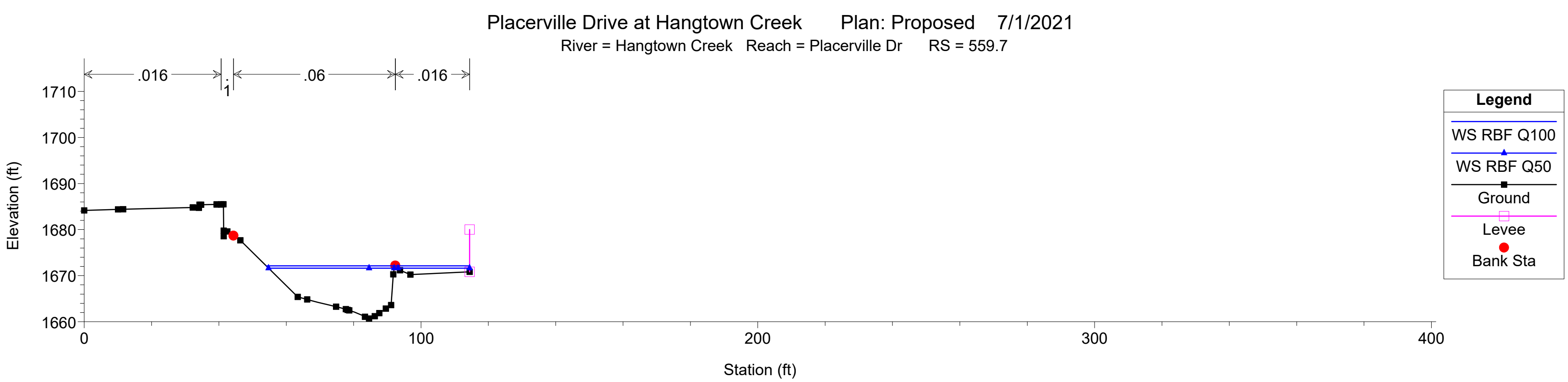




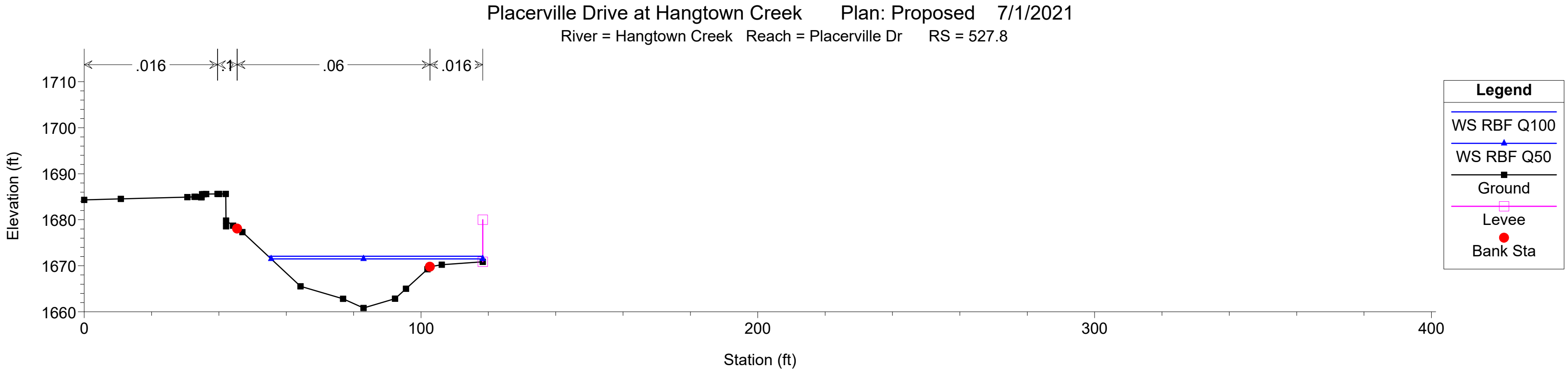
1 in Horiz. = 52 ft 1 in Vert. = 38 ft

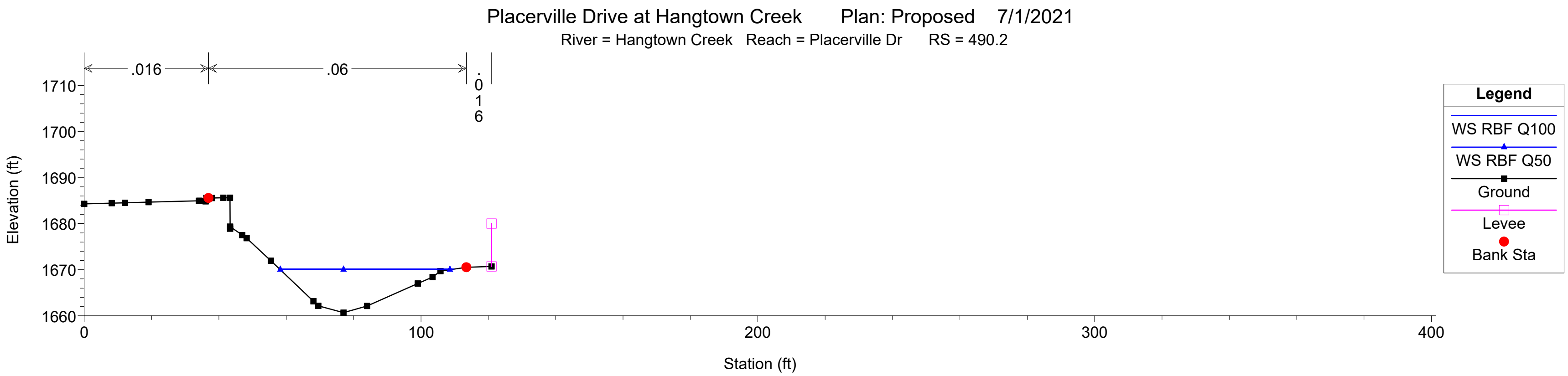


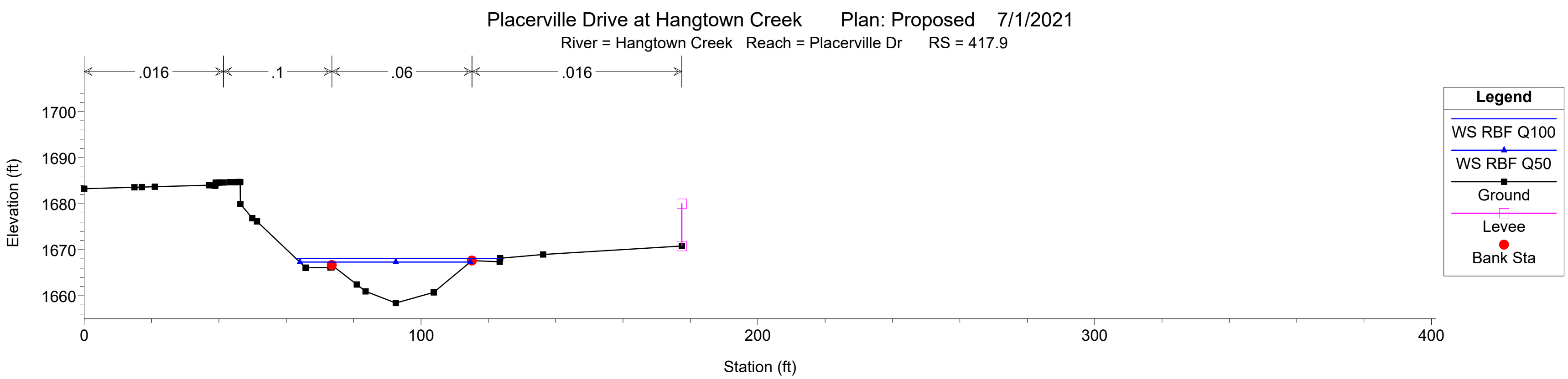




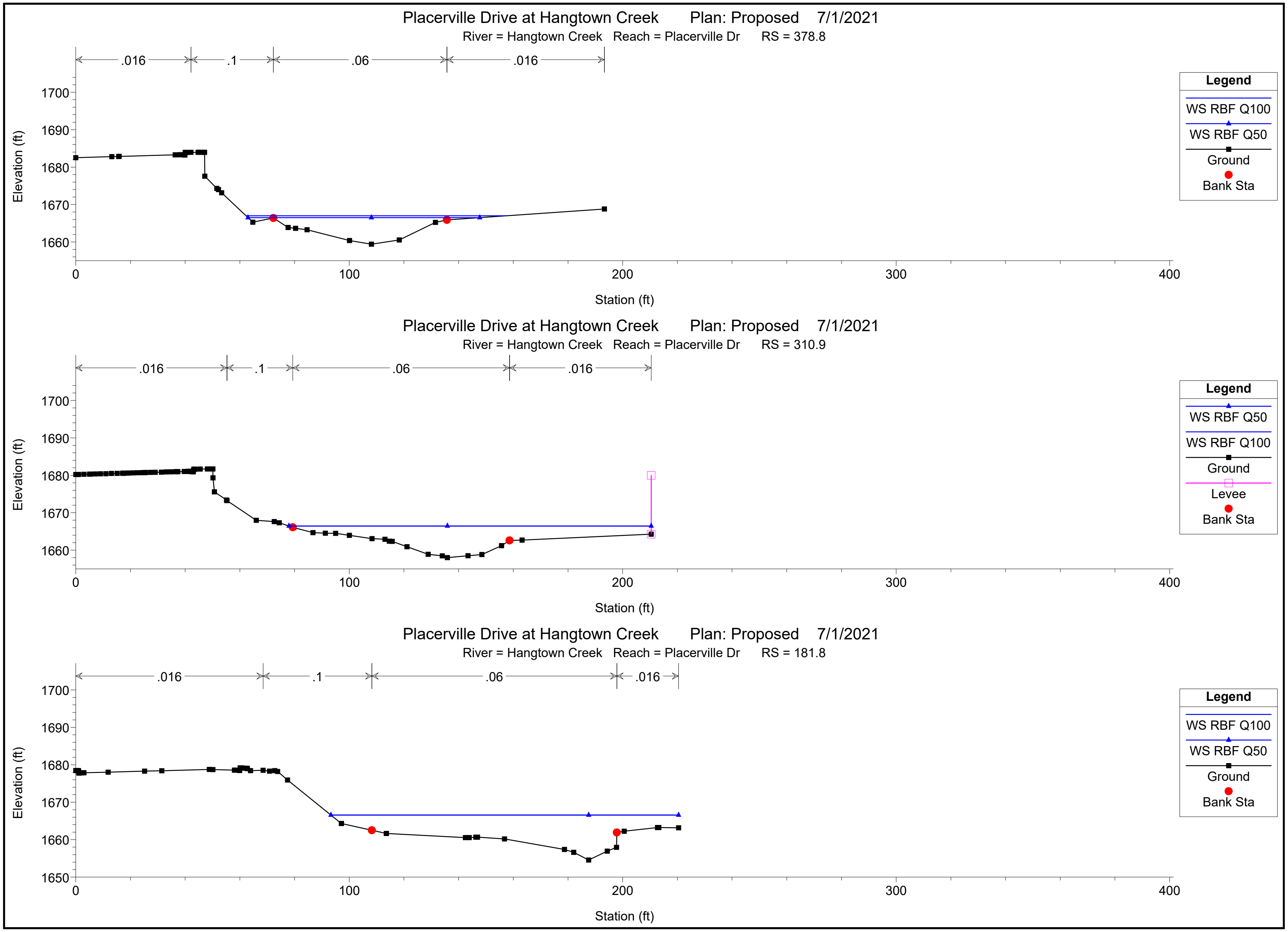
1 in Horiz. = 52 ft 1 in Vert. = 38 ft



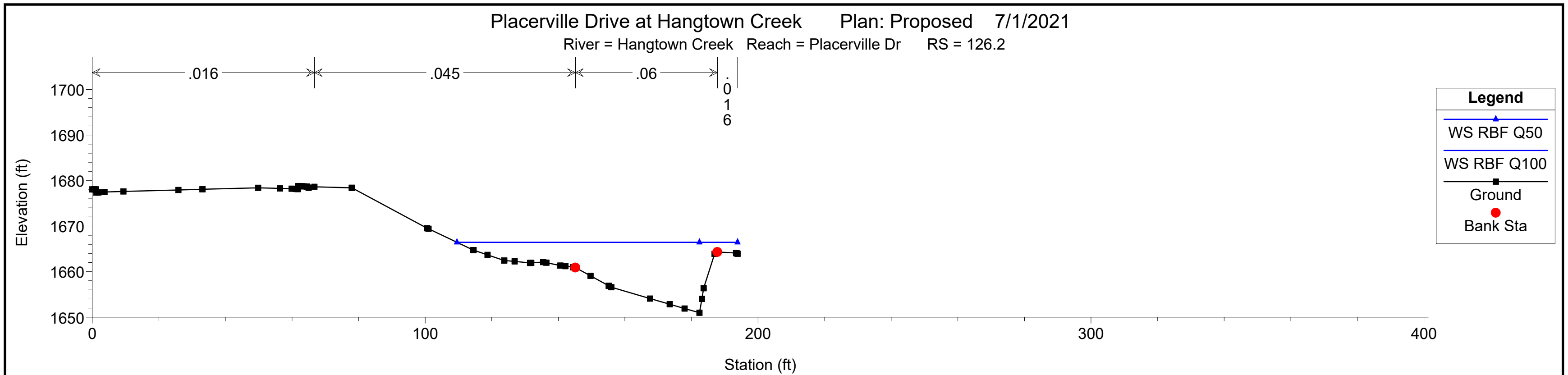


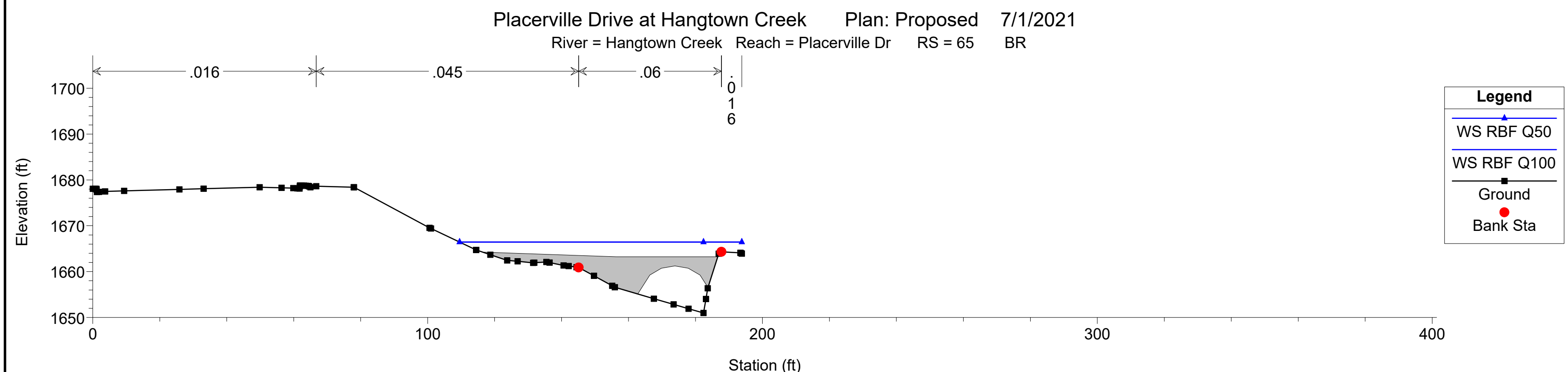


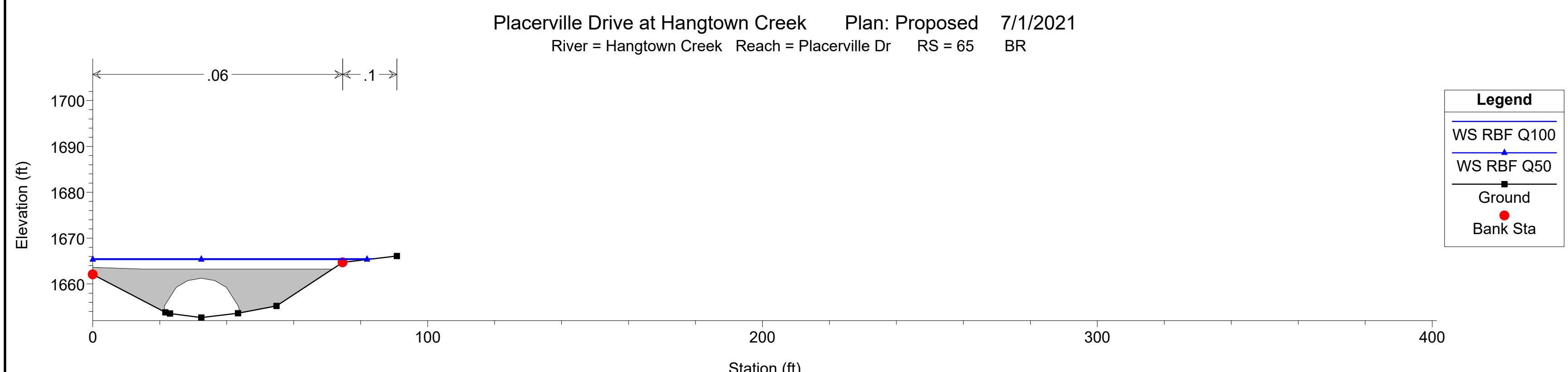
1 in Horiz. = 52 ft 1 in Vert. = 38 ft



1 in Horiz. = 52 ft 1 in Vert. = 38 ft

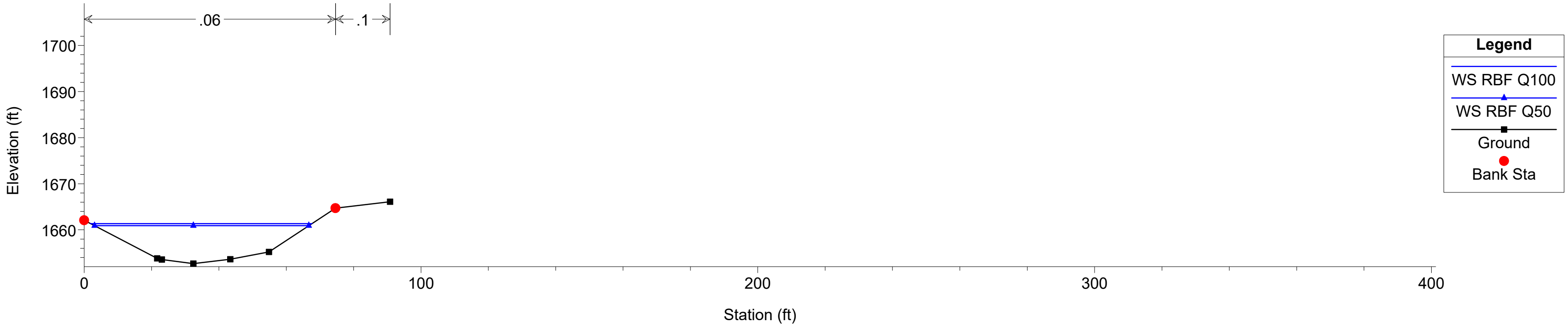




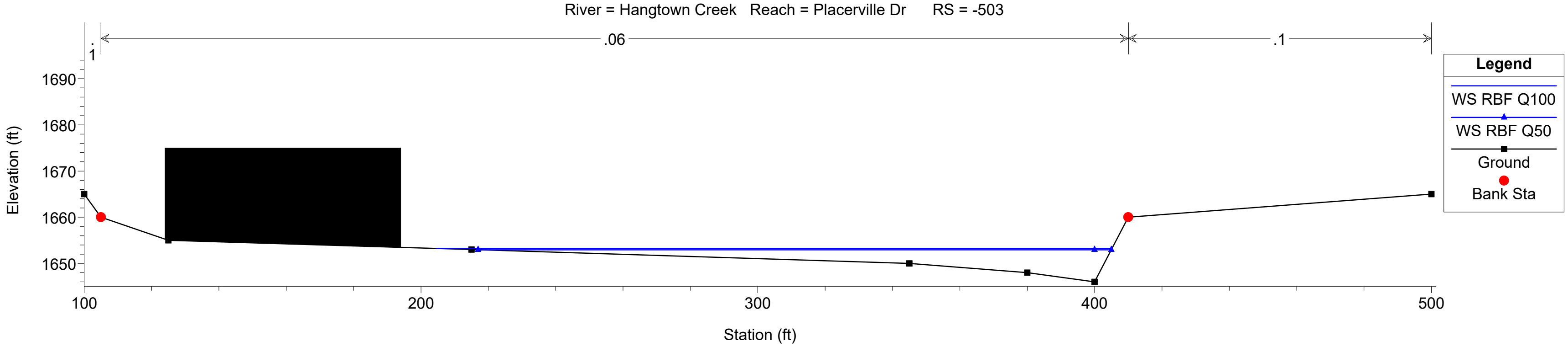


1 in Horiz. = 52 ft   1 in Vert. = 38 ft

Placerville Drive at Hangtown Creek Plan: Proposed 7/1/2021
River = Hangtown Creek Reach = Placerville Dr RS = 0



Placerville Drive at Hangtown Creek Plan: Proposed 7/1/2021
River = Hangtown Creek Reach = Placerville Dr RS = -503



1 in Horiz. = 52 ft 1 in Vert. = 38 ft

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Appendix C Scour Calculations

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Placerville Drive Bridge Replacement
City of Placerville, El Dorado County, California
Contraction Scour

100-year RBF
Calculation guideline from HEC-18 5th Edition
Proposed Three Span

Units = (SI or English)	English	
Ku = constant = 6.19 (SI) or 11.17 (English)	11.17	
g = acceleration due to gravity =	32.2	ft/s^2

Channel

Vchannel = Mean velocity of flow in main channel just upstream of bridge =	7.8	ft/s
D50channel = grain size in channel for which 50% of bed material is finer =	0.0007	ft
Yochannel = existing depth in the contracted channel section before scour =	7.5	ft
Ychannel = depth of flow just upstream of bridge in channel =	7.6	ft
VcD50channel = Ku*(Ychannel^(1/6))*(D50channel^(1/3))	1.4	ft/s
Contraction scour equation for channel =	**Live-Bed**	

Live-Bed

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1}$$ Equation 6.2

$$y_s = y_2 - y_o$$ Equation 6.3

Q1 channel = Flow in the upstream channel transporting sediment =	3200.0	ft^3/s
Q2 channel = Flow in the contracted channel = transporting sediment =	3200.0	ft^3/s
W1 channel = bottom or top width (consistent with W2 definition) of the upstream channel that is transporting bed material =	42.2	ft
W2 channel = bottom or top width (consistent with W1 definition) of the contracted channel section less pier widths =	62.4	ft
ω channel = fall velocity of bed material based on D50 =	0.06	ft/s
S channel = slope of energy grade line in main channel =	0.0	ft/ft
V* channel = shear velocity in the upstream channel section = (Ychannel*g*S channel)^.5 =	1.4	ft/s
V* channel/ω channel =	22.4	
k1 channel = (if V*/ω <0.5, 0.59, if(0.5<=V*/ω<=2,0.64,0.69)) =	0.69	
Y2channel = average depth in contracted section after scour = Ychannel*((Q2 channel/Q1 channel)^(6/7))*((W1 channel/W2 channel)^k1 channel) =	5.8	ft
Ys channel = Y2 channel - Yo channel =	**-1.7**	**ft**

Placerville Drive Bridge Replacement
City of Placerville, El Dorado County, California
Local Scour at Piers - Cohesionless

100-year RBF
Calculation guideline from HEC-18 5th Edition
Proposed Three Span

Units = (SI or English) =	English	English	
Complex Pier Scour? (Yes or No)	No	No	
Pier Number (Plan)	2	3	
Pier Number (HEC-RAS)	1	2	
Pier Scour component			
Water Surface Elevation	1675.3	1675.3	ft
Ground Elevation at Pier	1666.8	1666.2	ft
y1 = Approach flow depth at the beginning of computations =	8.5	9.0	ft
V1 = Approach velocity used at the beginning of computations =	10.4	10.5	ft/s
a = pier width =	2.5	2.5	ft
L = length of pier =	20.0	20.0	ft
L/a (if L/a is larger than 12, then use 12 as a maximum)	8	8	
Ө = angle of attack of flow =	0	0	degrees
Pier shape	Group of cylinders	Group of cylinders	
K1 = correction factor for pier nose shape =	1.0	1.0	
K2 = correction factor for angle of attack = (cosӨ+(L/a)*sinӨ)^0.65	1.0	1.0	
K3 = correction factor for bed condition =	1.1	1.1	
D50 = grain size for which 50% of bed material is finer =	0.0007	0.0007	ft
Yspier = scour component for the pier stem in the flow = y1*(Khpier*(2*K1*K2*K3*((a/y1)^0.65)*((V1/((g*y1)^0.5))^0.43)) =	**6.9**	**7.0**	**ft**

Placerville Drive Bridge Replacement
City of Placerville, El Dorado County, California
Local Scour at Abutments - NCHRP
100-year RBF
Calculation guideline from HEC-18 5th Edition

Proposed Three Span

Left Overbank = Abutment 1 (West)

y1 = Upstream flow depth	7.6	ft
Upstream velocity	7.8	ft/s
Upstream flow	3122.9	cfs
Upstream channel width	52.2	ft
q1 = Upstream unit discharge	59.8	ft2/s
Flow depth through bridge	7.5	ft
Velocity through bridge	8.3	ft/s
Flow through bridge	3200.0	cfs
Bridge opening width less pier widths	61.4	ft
q2 = Unit discharge in the constricted opening	62.3	ft2/s
Abutment scour equation =	**Live-Bed**	

Live-Bed Condition Abutment Scour

$$y_c = y_1 \left(\frac{q_{2c}}{q_1}\right)^{6/7}$$ Equation 8.5

yc = Flow depth including live-bed contraction scour	7.9	ft

$$y_{max} = \alpha_A y_c \ or \ y_{max} = \alpha_B y_c$$ Equation 8.3

$$y_s = y_{max} - y_0$$ Equation 8.4

yc = Flow depth including live-bed or clear-water contraction scour	7.9	ft
q2/q1	1.0	
Type of abutment (Wingwall vs Spill-through)	Spill-through	
αA = Amplification factor for live-bed conditions	1.46	
y max = Maximum flow depth resulting from abutment scour	11.6	ft
y0 = Flow depth prior to scour	7.5	ft
ys = Abutment scour depth	**4.1**	ft
Contraction scour component	**0.0**	ft
Local scour component	**4.1**	ft

Note: ys is a combination of local and contraction scour.

Placerville Drive Bridge Replacement
City of Placerville, El Dorado County, California
Local Scour at Abutments - NCHRP
100-year RBF

Calculation guideline from HEC-18 5th Edition

Proposed Three Span

Right Overbank = Abutment 4 (East)

y1 = Upstream flow depth	7.6	ft
Upstream velocity	7.8	ft/s
Upstream flow	3122.9	cfs
Upstream channel width	52.2	ft
q1 = Upstream unit discharge	59.8	ft2/s
Flow depth through bridge	7.5	ft
Velocity through bridge	8.3	ft/s
Flow through bridge	3200.0	cfs
Bridge opening width less pier widths	61.4	ft
q2 = Unit discharge in the constricted opening	62.3	ft2/s
Abutment scour equation =	**Live-Bed**	

Live-Bed Condition Abutment Scour

$$y_c = y_1 \left(\frac{q_{2c}}{q_1}\right)^{6/7}$$ Equation 8.5

yc = Flow depth including live-bed contraction scour	7.9	ft

$$y_{max} = \alpha_A y_c \; or \; y_{max} = \alpha_B y_c$$ Equation 8.3

$$y_s = y_{max} - y_0$$ Equation 8.4

yc = Flow depth including live-bed or clear-water contraction scour	7.9	ft
q2/q1	1.0	
Type of abutment (Wingwall vs Spill-through)	Spill-through	
αA = Amplification factor for live-bed conditions	1.46	
y max = Maximum flow depth resulting from abutment scour	11.6	ft
y0 = Flow depth prior to scour	7.5	ft
ys = Abutment scour depth	**4.1**	ft
Contraction scour component	**0.0**	ft
Local scour component	**4.1**	ft

Note: ys is a combination of local and contraction scour.

Appendix D Rock Slope Protection Calculations

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Placerville Drive Bridge Replacement
City of Placerville, El Dorado County, California
Streambank Rock Slope Protection
Calculation guideline from Caltrans Highway Design Manual

Input from HEC-RAS for Proposed Three Span

100-year RBF

Input

Location along stream:	Upstream	Upstream face	Downstream face	Downstream	
	Left/West/Abut 1	Left/West/Abut 1	Right/East/Abut 4	Right/East/Abut 4	
River Station	850.52	750.9 BR U	750.9 BR D	680.2	
V_{avg}	9.4	8.3	8.0	5.5	ft/s
g	32.2	32.2	32.2	32.2	ft/s^2
Depth based on	Average	Average	Average	Average	
y	7.3	7.5	6.8	6.4	ft
S_f	1.1	1.1	1.1	1.1	
C_s	0.3	0.3	0.3	0.3	
Cross section location:	Outside of bend	Straight channel	Straight channel	Outside of bend	
C_v	1.17	1.00	1.00	1.34	

For outside of bends, need R_c and W:

Note: these parameters also affect the V_{des}; for natural channels, $V_{des}=V_{avg}$ for $R_c/W>26$

Note: these parameters also affect the V_{des}; for trapezoidal channels, $V_{des}=V_{avg}$ for $R_c/W>8$

R_c	165	165	45	45	ft
W	45	66.4	64.0	90	ft
R_c/W	3.7	2.5	0.7	0.5	
C_t	1.0	1.0	1.0	1.0	
S_g	2.65	2.65	2.65	2.65	
Type of channel:	Natural	Natural	Natural	Natural	
V_{des}	13.6	12.7	14.6	10.4	ft/s
K_1	0.87	0.87	0.87	0.87	
θ	26.6	26.6	26.6	26.6	degrees
SS	2	2	2	2	
D_{30}	1.3	1.0	1.4	0.8	ft
D_{50}	1.6	1.1	1.7	1.0	ft
D_{50}	19.0	13.8	19.9	11.6	inches
	VI	IV	VI	III	RSP Class
	3/8 ton	**300 lb**	**3/8 ton**	**150 lb**	Median particle weight
	21	**15**	**21**	**12**	Median particle diameter (inches)

Placerville Drive Bridge Replacement

City of Placerville, El Dorado County, California

Streambank Rock Slope Protection

Calculation guideline from Caltrans Highway Design Manual

Input from HEC-RAS for Proposed Three Span

100-year RBF

Input

Location along stream:	Upstream	Upstream face	Downstream face	Downstream	
	Right/East/Abut 4	Right/East/Abut 4	Left/West/Abut 1	Left/West/Abut 1	
River Station	850.52	750.9 BR U	750.9 BR D	680.2	
V_{avg}	9.4	8.3	8.0	5.5	ft/s
g	32.2	32.2	32.2	32.2	ft/s^2
Depth based on	Average	Average	Average	Average	
y	7.3	7.5	6.8	6.4	ft
S_f	1.1	1.1	1.1	1.1	
C_s	0.3	0.3	0.3	0.3	
Cross section location:	Inside of bend	Inside of bend	Inside of bend	Inside of bend	
C_v	1.00	1.00	1.00	1.00	

For outside of bends, need R_c and W:

Note: these parameters also affect the V_{des}; for natural channels, $V_{des}=V_{avg}$ for $R_c/W>26$

Note: these parameters also affect the V_{des}; for trapezoidal channels, $V_{des}=V_{avg}$ for $R_c/W>8$

R_c	30	30	99	99	ft
W	45	66.4	64.0	90	ft
R_c/W	0.7	0.5	1.5	1.1	
C_t	1.0	1.0	1.0	1.0	
S_g	2.65	2.65	2.65	2.65	
Type of channel:	Natural	Natural	Natural	Natural	
V_{des}	17.1	15.9	13.2	9.4	ft/s
K_1	0.87	0.87	0.87	0.87	
θ	26.6	26.6	26.6	26.6	degrees
SS	2	2	2	2	
D_{30}	2.0	1.7	1.1	0.5	ft
D_{50}	2.4	2.0	1.3	0.6	ft
D_{50}	29.2	24.0	15.4	6.7	inches
	VIII	VII	V	II	RSP Class
	1 ton	**1/2 ton**	**1/4 ton**	**60 lb**	Median particle weight
	30	**24**	**18**	**9**	Median particle diameter (inches)

Placerville Drive Bridge Replacement
City of Placerville, El Dorado County, California
Rock Slope Protection Calculations for Abutments
Calculation guideline from HEC-23 3rd Edition

Input from HEC-RAS for Proposed Three Span

100-year RBF

Location	Upstream	Upstream Face	Downstream Face	Downstream	
V	9.4	8.3	8.0	5.5	ft/s
g	32.2	32.2	32.2	32.2	ft/s^2
y	7.3	7.5	6.8	6.4	ft
Fr	0.61	0.53	0.54	0.38	
Equation	**Isbash**	**Isbash**	**Isbash**	**Isbash**	

For Froude Numbers $(V/(gy)^{1/2})$<=0.80, Isbash relationship (Equation 14.1)

$$D_{50} = \frac{yK}{(S_s - 1)}\left[\frac{V^2}{gy}\right]$$

y	7.3	7.5	6.8	6.4	depth of flow in the contracted bridge opening, ft
K	1.02	1.02	1.02	1.02	1.02 for vertical wall abutment, 0.89 or for spill-through abutment
S_s	2.65	2.65	2.65	2.65	specific gravity of rock
V	9.4	8.3	8.0	5.5	average velocity in contracted section, ft/s
g	32.2	32.2	32.2	32.2	gravitational acceleration, ft/s^2
D_{50}	1.7	1.3	1.2	0.6	median stone diameter, ft
D_{50}	20.3	15.9	14.9	6.9	median stone diameter, inches

VI	V	IV	II	RSP Class
3/8 ton	1/4 ton	300 lb	60 lb	Median particle weight
21	18	15	9	Median particle diameter (inches)

Placerville Drive Bridge Replacement
City of Placerville, El Dorado County, California
Streambank Rock Slope Protection
Calculation guideline from Caltrans Highway Design Manual

Input from HEC-RAS for Proposed Three Span
100-year RBF

FHWA HEC-15
Page 46, 3-12, Section 3.4 Stability in Bends

	DS Rt	DS Rt		
	750.9 BR D	680.2		
t_b	3.16	3.34	lb/ft^2	**side** shear stress on the channel
K_b	2	2		
t_d	1.58	1.67	lb/ft^2	from HEC-RAS output
Rc	45	45	feet	
T	64	90	feet	
Rc/T	0.7	0.5	<2, therefore Kb=2	

From Caltrans HDM Table 865.2 Permissible Shear and Velocity for Selected Lining Materials
The channel is: Fine gravels with sand, silt, and cobbles mixed in.
For fine gravels:
The permissible shear is: 0.075 lb/ft2
The permissible velocity is: 2.5 ft/s

Therefore, the shear stress at the side of the channel exceeds the permissible shear.
And RSP is recommended.

a	0.6	for CU units
n	0.04	for RSP

R = hydraulic radius
Lp = length of protection from point of tangency

RS	R	Lp
850.52	5.92	119
750.9 BR U	3.12	57
750.9 BR D	4.37	84
680.2	6.1	124

Placerville Drive Bridge Replacement
City of Placerville, El Dorado County, California
Streambank Rock Slope Protection
Calculation guideline from Caltrans Highway Design Manual
Input from HEC-RAS for Proposed Three Span
100-year RBF

Input

RSP for area between upstream culvert and new bridge

Location along stream:	BxN culv & br	BxN culv & br	
River Station	915.1	911.9	
V_{avg}	7.9	7.8	ft/s
g	32.2	32.2	ft/s^2
Depth based on	Average	Average	
y	7.6	7.6	ft
S_f	1.1	1.1	
C_s	0.3	0.3	
Cross section location:	Straight channel	Straight channel	
C_v	1.00	1.00	
C_t	1.0	1.0	
S_g	2.65	2.65	
Type of channel:	Natural	Natural	
V_{des}	7.9	7.8	ft/s
K_1	0.87	0.87	
θ	26.6	26.6	degrees
SS	2	2	
D_{30}	0.3	0.3	ft
D_{50}	0.3	0.3	ft
D_{50}	4.1	4.1	inches
	I	I	RSP Class
	20 lb	**20 lb**	Median particle weight
	6	**6**	Median particle diameter (inches)