Appendix 5.5 (Continued)

Hydrology (Part 7 of 15)

Appendix 5.5c

Castaic Creek Drainage Concept

LAND DEVELOPMENT DIVISION
HYDROLOGY UNIT

TO:	PACE	Date: 09/24/2019
	17520 Newhope Street, Suite 200	
	Fountain Valley, CA 92708	
ATTN:	Jose Cruz	

REVIEW OF HYDROLOGY STUDY

PM NO. **18108**

DATE OF REPORT **09/10/2019**
PLAN CHECK NO. **2**
PLAN CASE NO. **ESTU2001000012**

We have reviewed your Hydrology Study.

[X] The Hydrology Study has been approved.

AYM

REVIEWED BY M.D Esfandi
DAVID ESFANDI (626) 458-7130

REGISTERED PROFESSIONAL ENGINEER
MANOUCHEHR DAVID ESFANDI
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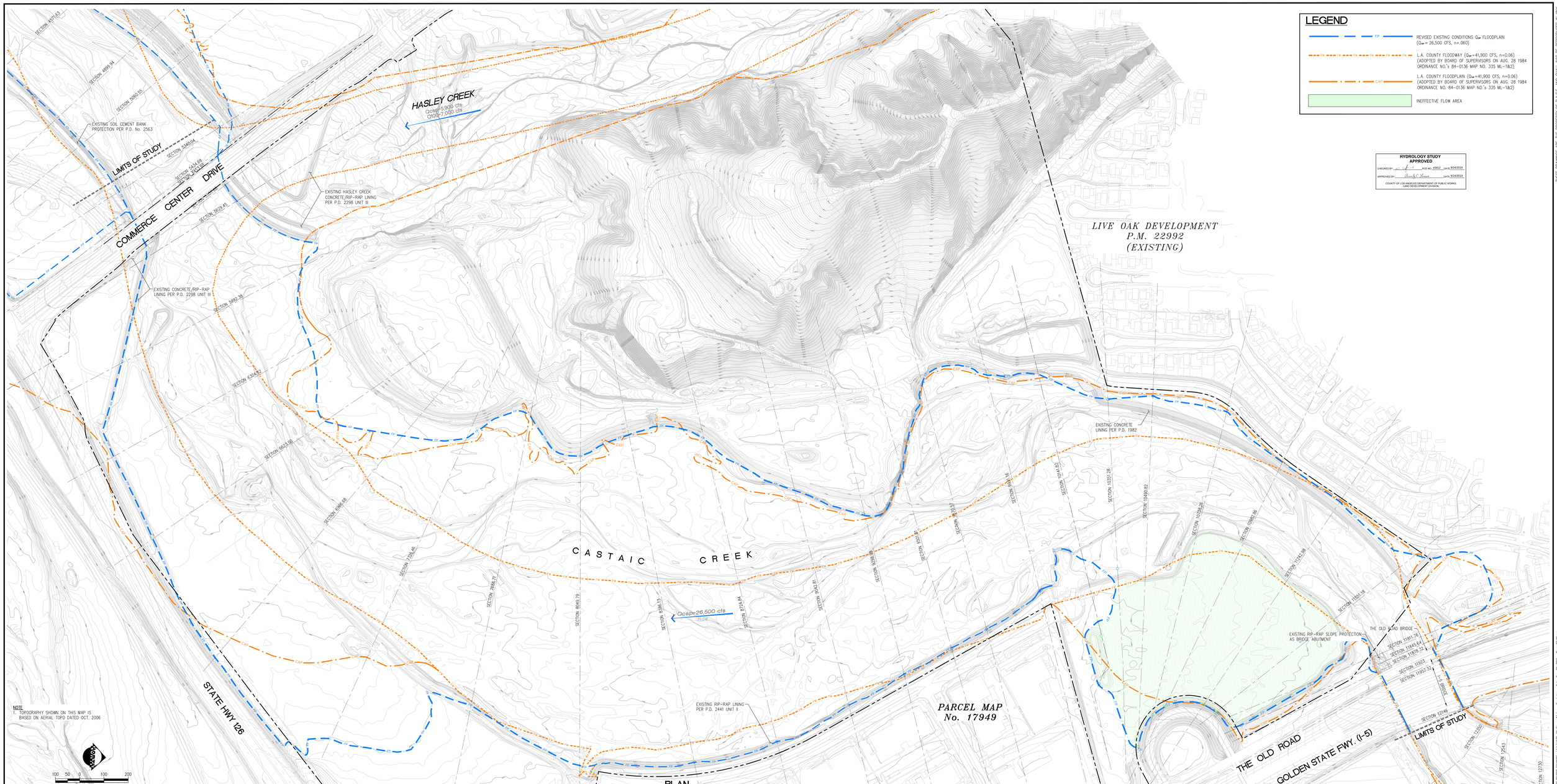

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DRAINAGE CONCEPT REPORT
VOLUME II OF V
CASTAIC CREEK SOIL CEMENT BANK PROTECTION
FOR T.P.M. NO. 18108

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DATE: Sep. 2019

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TITLE

EXISTING CONDITION FLOODPLAIN MAPPING

FIGURE 04

JOB NO. A535

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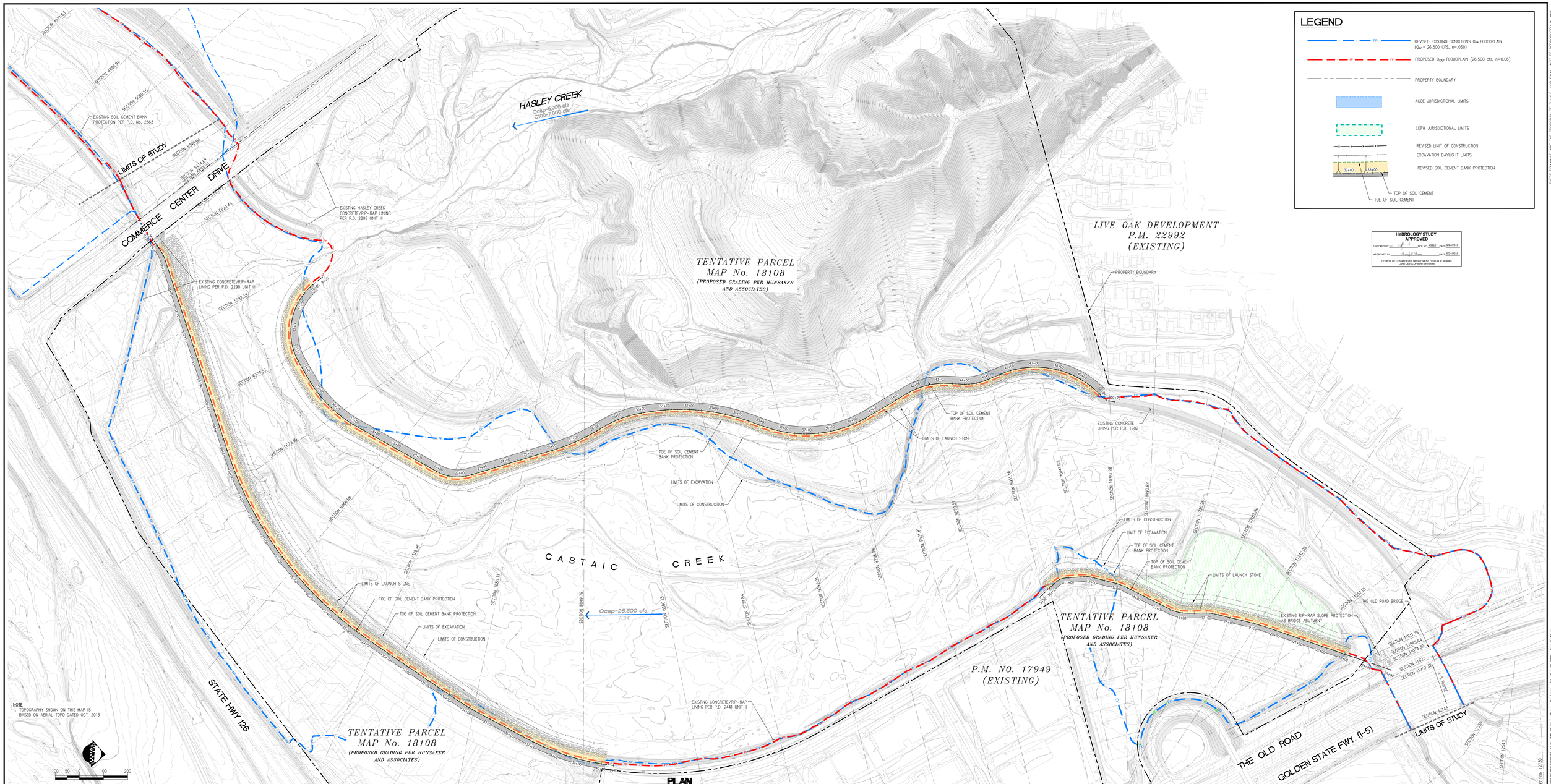

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TITLE

PROPOSED CONDITION FLOODPLAIN MAPPING

FIGURE 05

JOB NO. 4535

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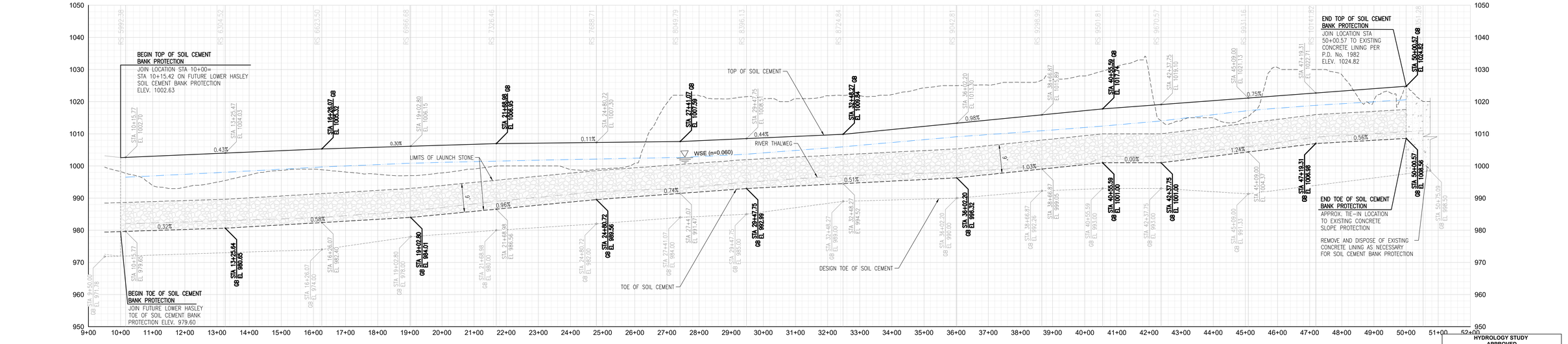

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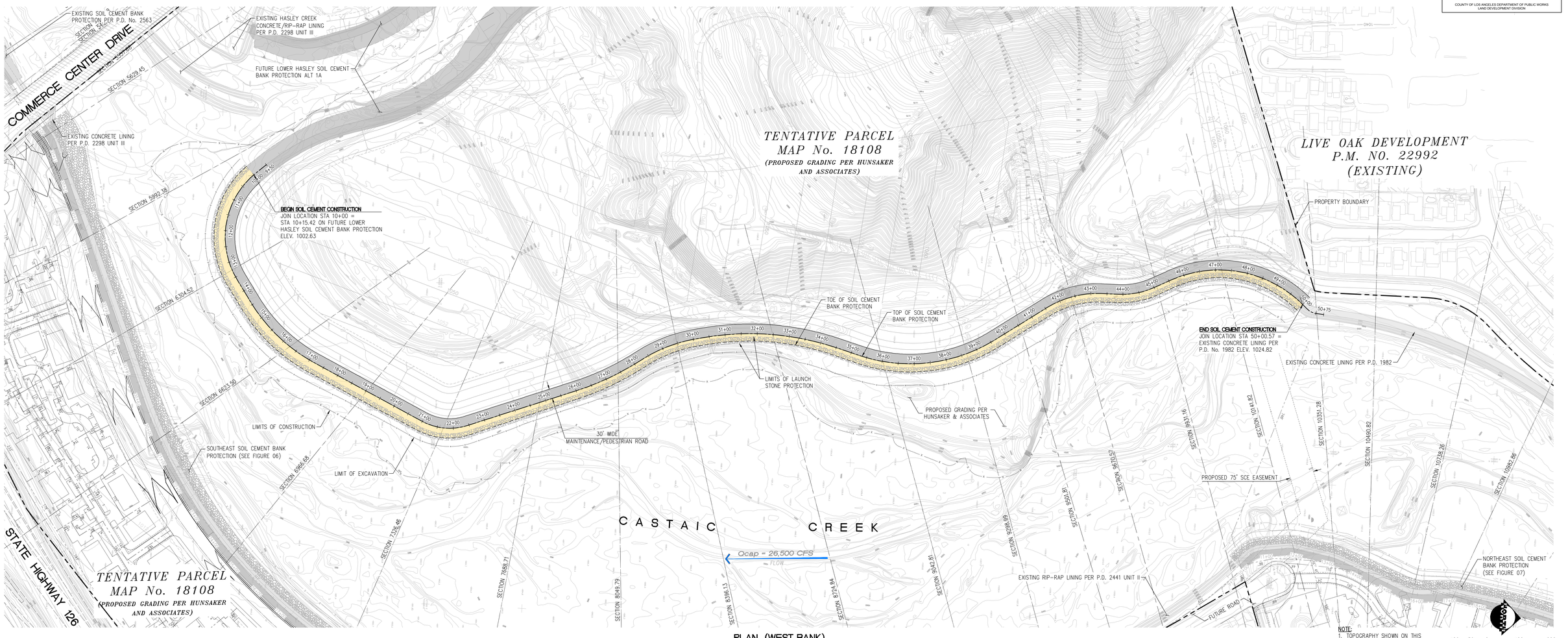

PLAN (WEST BANK)

NOTE:
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TITLE

WEST BANK SOIL CEMENT BANK PROTECTION PLAN & PROFILE

FIGURE 06

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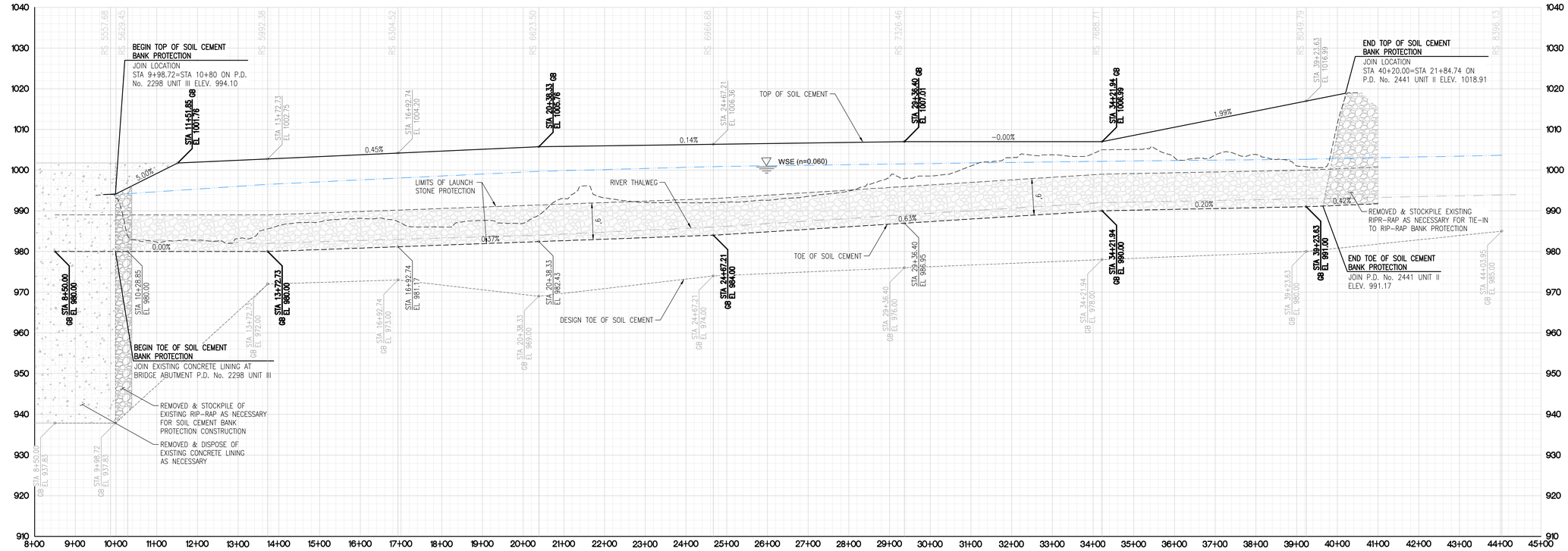
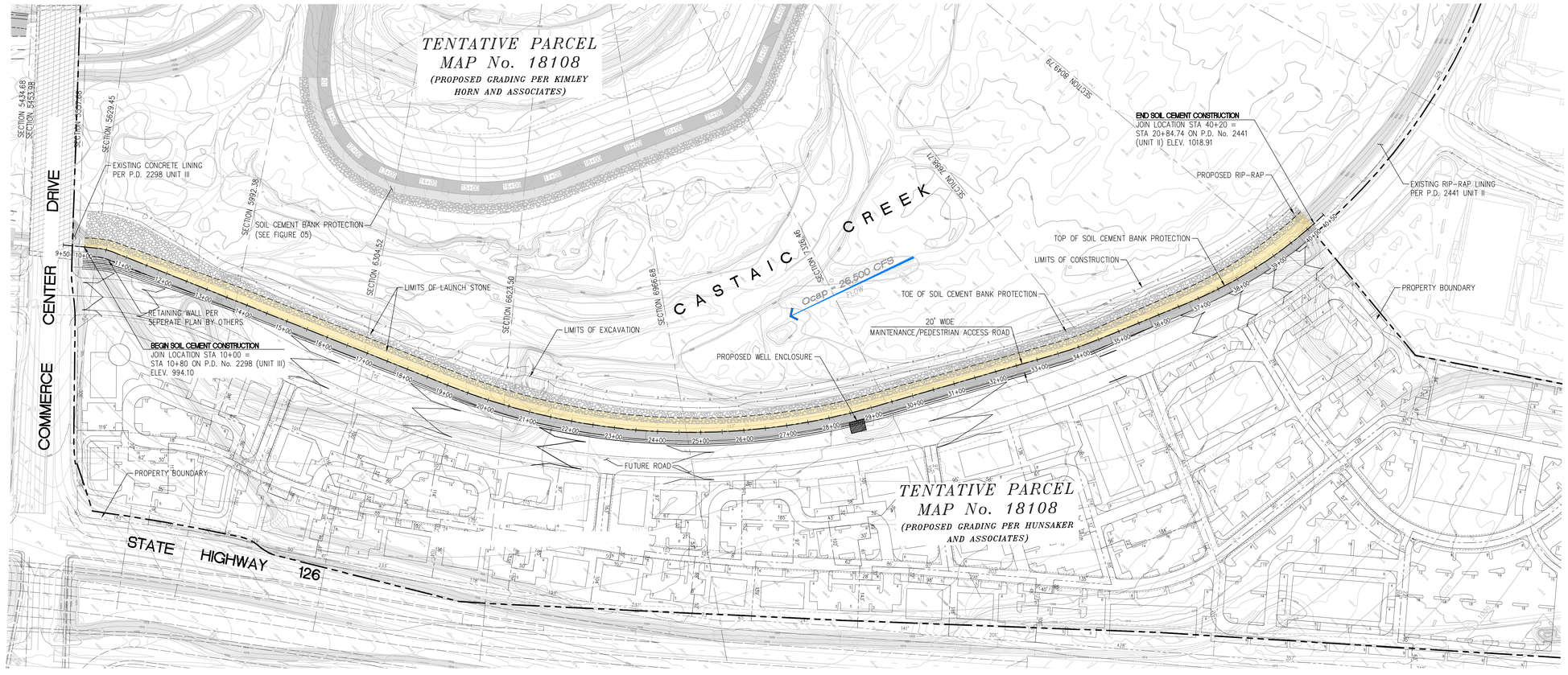


PROFILE (SOUTHEAST BANK)



PLAN (SOUTHEAST BANK)

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TITLE
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FIGURE 07
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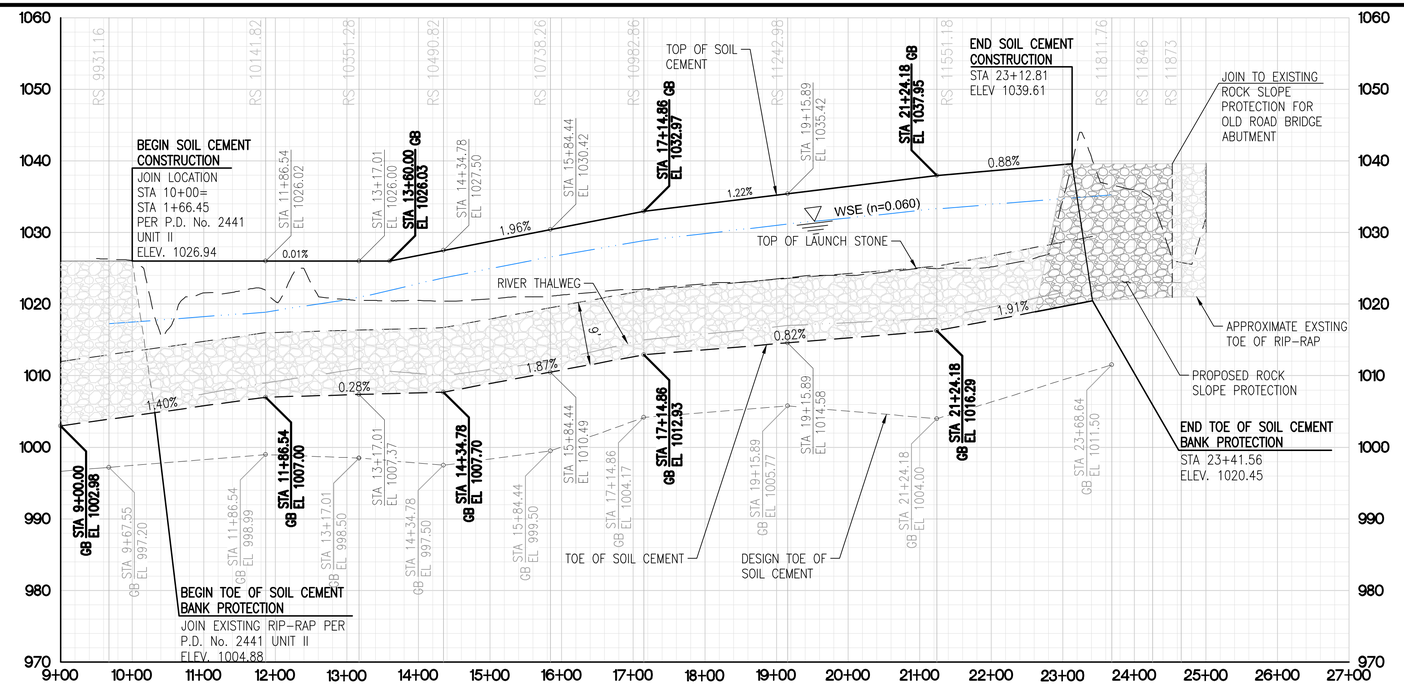
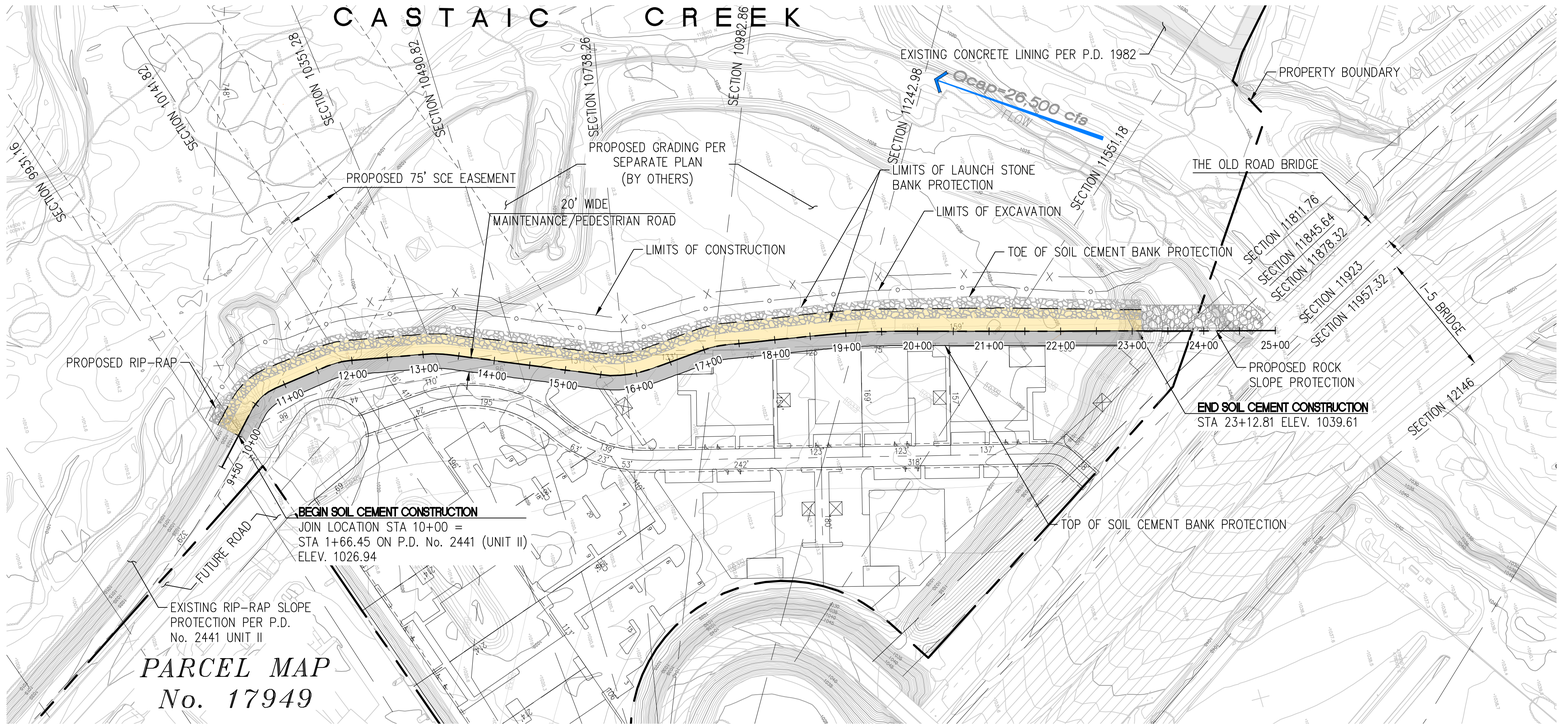


PROFILE (NORTHEAST BANK)



PLAN (NORTHEAST BANK)

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Valencia Commerce Center TPM No. 18108

Drainage Concept Report Volume II of V
Castaic Creek Bank Protection
ESTU No. 2001000012

September 2019
February 2015 (Revised)
March 2008 (Revised)
January 2008 (Revised)

Prepared For:

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PACE JN A535

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Appendix H	LACDPW Hydrology Data
Appendix I	Launch Stone Sizing Technical Memo
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1 Introduction

Pacific Advanced Civil Engineering, Inc. (PACE) has been retained by FivePoint Holdings to prepare a Drainage Concept Report (DCR Volume II of V) for the proposed creek improvements at Valencia Commerce Center (VCC) Industrial Park Development in Tentative Parcel Map (TPM) No. 18108. The proposed creek improvements associated with the VCC project include soil cement bank protection combined with launch stone rip-rap along both banks of Castaic Creek and Hasley Creek. The analysis presented in the following report will focus on the improvements located along Castaic Creek. A separate DCR will be prepared for the improvements along Hasley Creek (DCR Volume III of V).

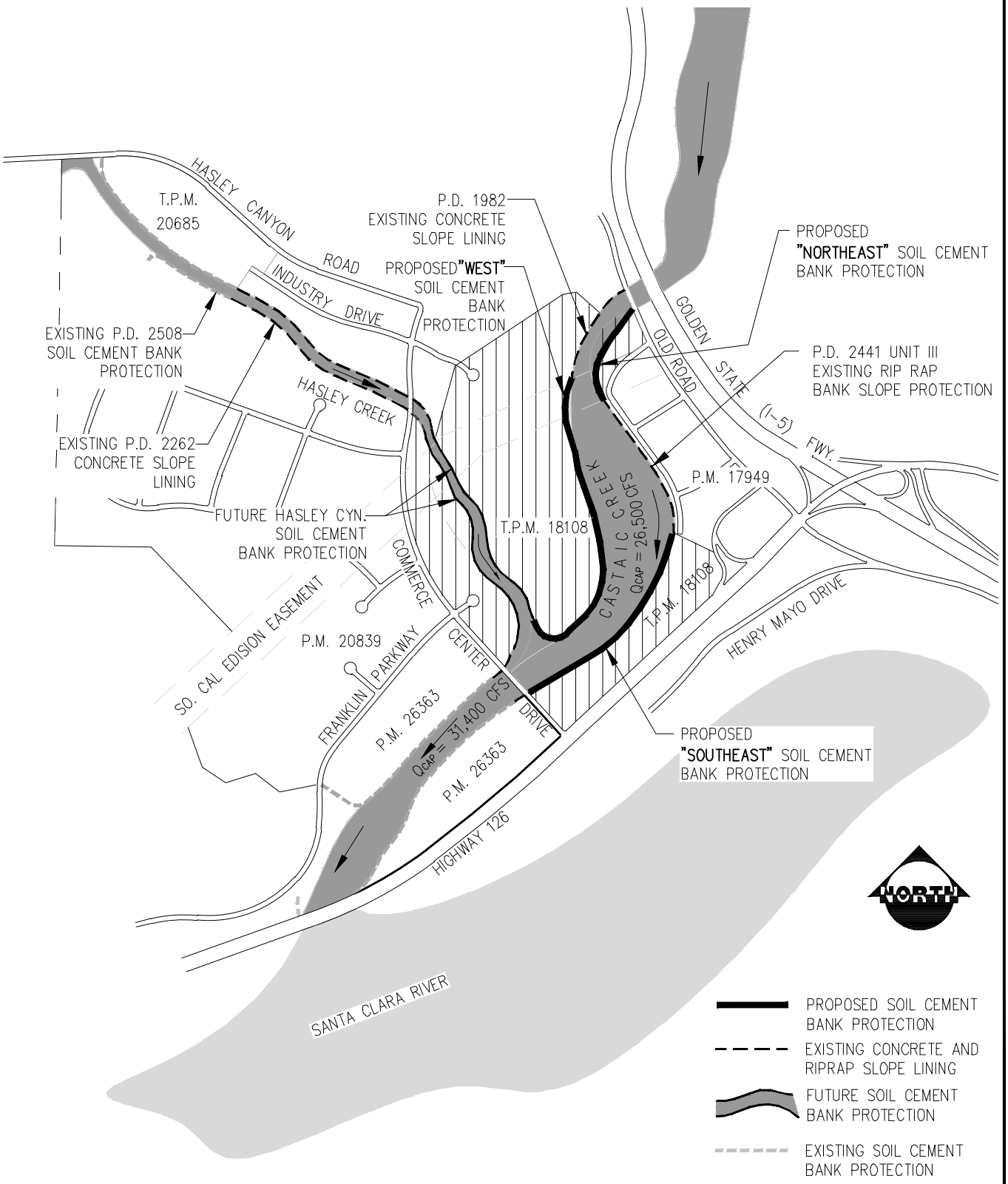
The VCC project site is situated in an unincorporated area of Los Angeles County, west of the Interstate 5 (I-5) and north of State Route 126 (SR-126). A location map for the project site is shown on **Figure 1**. The proposed improvements along Castaic Creek consist of approximately 8,400 lineal feet (LF) of soil cement bank protection. The bank protection proposed for the west bank will be constructed as a continuous section beginning near the confluence of Castaic Creek and Hasley Creek and extending roughly 4,000 LF upstream, where it will join the existing concrete slope lining (PD No. 1982) adjacent to Live Oak Road. The bank protection proposed for the east bank will be constructed in two separate sections due to an existing rip-rap slope protection (PD No. 2441-Unit II) located adjacent to Hancock Parkway. The southeast section of the proposed bank protection will be approximately 3,000 LF and will extend from the Commerce Center Drive (CCD) Bridge to the existing rip-rap slope protection (PD No. 2441). The northeast section of the proposed soil cement bank protection will be around 1,400 LF and extends from the existing rip-rap slope protection (PD No. 2441) to the Old Road Bridge.

As part of this DCR, PACE performed a hydraulic analysis of Castaic Creek in the vicinity of the VCC project, referred to herein as the "Project Specific" analysis. The purpose of the Project Specific hydraulic analysis is to provide design guidelines for the proposed bank protection and to evaluate potential floodplain impacts caused by the project. An analysis was performed for Castaic Creek as part of the Approved Castaic Creek Fluvial Study Phase I Final Draft Report, which will be referred to herein as the "Fluvial Study". The Fluvial Study established design guidelines for proposed bank protection measures along Castaic Creek within the reach between the Santa Clara River and the I-5 Freeway Bridge. This reach includes the reach of Castaic Creek adjacent to the VCC project. The bank protection alignments included in the Fluvial Study considered the full build-out condition within this reach of Castaic Creek, whereas the Project Specific analysis presented in this DCR only considers the improvements proposed within the TPM 18108 development. The results of the Project Specific analysis were compared to the results from the Fluvial Study to determine the most conservative design for the proposed bank protection.

Design of the proposed improvements along Castaic Creek is in accordance with current Los Angeles County Department of Public Works (LACDPW) hydrology and hydraulic design criteria. Revised Capital Flood flow rates (Q_{CAP}) were obtained from the LACDPW Water Resources Division for the reach of Castaic Creek between I-5 and SR-126 (see **Table 1** and **Appendix H**). All water surface elevations and topographic data presented in this report are referenced to the North American Vertical Datum of 1988 (NAVD 88). Topographic data shown on the figures was collected in 2013 and is the same topographic data used for the HEC-RAS modeling analysis.

Hydraulic models of Castaic Creek were prepared and analyzed for existing (pre-project) and proposed (post-project) conditions. **Section 3** describes these hydraulic models in detail. Results of the Project Specific post-project analysis were compared to results from the Fluvial Study. This comparison is described in **Section 6**. These analyses were also used to evaluate the potential offsite impacts and to determine the appropriate design top and toe elevations for the bank protection. **Table 4** presents the results of the post-project HEC-RAS model (Q_{CAP}, Manning's roughness value of 0.06) for this particular reach of Castaic Creek.

The Project Specific analysis evaluates impacts of the following: (1) fluvial modifications of the riverbed from the Capital Flood event and (2) changes in the floodplain fluvial mechanics over the long-term. General and long-term adjustment values were obtained from the Fluvial Study. The approved fluvial adjustment values were applied to determine the top and toe elevations of the Castaic Creek bank protection. A brief overview of the Fluvial Study methodology is provided in **Section 5**. The adopted final design top and toe elevations for the VCC bank protection will be based on the more conservative values between the Project Specific and Fluvial Study analyses.



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CASTAIC CREEK
SOIL CEMENT BANK PROTECTION
TPM # 18108 DCR VOL. II OF V

INDEX MAP

FIGURE 01

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2 Project Hydrology

2.1 Regional Castaic Creek Hydrology

The study reach is situated within the Castaic Creek Watershed and encompasses approximately 209 square miles. The boundaries of the upper and lower Castaic Creek basins are shown on **Figure 2**. The hydrology used for the hydraulic analysis was obtained from LACDPW Water Resources Division (WRD). The Los Angeles County Flood Control District (LACFCD) analyzes conditions based on the Capital Flood event, which assumes a burned watershed and resulting bulked peak flows. FEMA has published maps to designate flood hazard zones that are accepted as the floodplain boundaries for insurance purposes. The Q_{CAP} is in excess of the flow anticipated by the 100-year FEMA storm event (see **Table 1** for Q_{CAP} and 100-year flow rates). All proposed grading located behind the proposed bank protection for Castaic Creek is currently above, or will be filled to be above, the capital floodplain. Therefore, adjacent sites would not be subject to flood hazards from Castaic Creek.

The FEMA Flood Insurance Rate Map (FIRM) 100-year flood hazard area is accepted as the floodplain boundary for insurance purposes. LACDPW analyzes conditions based on the Capital Flood, which assumes a burned watershed and resulting bulked flows. The Q_{CAP} flow is in excess of the flow anticipated by the base flood or 100-year FEMA storm event. The design storm established by LACDPW is the Capital Flood which is defined as follows:

1. Rain falls on already saturated soils over a period of four days with the maximum rainfall on the fourth day. During the 24-hour period of maximum rainfall, the rainfall intensity typically increases during the first 70 to 90% of the period and decreases in the remaining time. Furthermore, approximately 80% of the amount of the 24-hour rainfall falls within the same 70 to 90% of the period.

2. When converting rainfall to runoff, rainfall that is not lost due to the hydrologic processes of interception, evaporation, transpiration, depression storage, infiltration, or percolation is assumed to be surface runoff.

3. The natural portions of the watershed have been burned by fire, which decreases soil infiltration.

4. A bulking factor is estimated. The runoff would carry a large layer of eroded topsoil, burned trees, and brush from burned areas. To account for the quantity of debris, the design flow rate is artificially increased by a percentage increase in flow rate, or bulking factor.

PACE has utilized the Q_{CAP} flow rate for the project hydraulic analysis and design. The Q_{CAP} and 100-year flow rates were obtained from LACDPW (see the Information Request Summary provided in **Appendix H**). Q_{CAP} and 100-year flow rates used are summarized in **Table 1**.

Table 1 – Design Hydrology Summary

Return Period	Design Flow (cfs)	Location within Castaic Creek
Revised Q_{CAP} (b&b) (1)	26,500	Commerce Center Drive Bridge to Interstate 5
100-Year (2)	11,805	Approx. 2,100-ft U/S of Charlie Canyon Confluence

Notes:
*(1) Source: LADPW burned & bulked flow – See **Appendix H**.*
(2) Source: FEMA FIS for Los Angeles County, CA Unincorporated Areas, September 26, 2008.

2.2 Local Hydrology

The on-site hydrology study is being prepared by another consultant and will be submitted as a separate report (TPM No. 18108, DCR Volume I of V).

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FIGURE 2

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CASTAIC CREEK
SOIL CEMENT BANK PROTECTION
TPM #18108 DCR VOL. II of V
VALENCIA CA

TITLE
CASTAIC CREEK WATERSHED

3 HEC-RAS Modeling

An extensive hydraulic analysis was performed to determine floodplain limits, as well as the top and toe elevations for the proposed bank protection along Castaic Creek associated with the VCC TPM 18108 project.

3.1 HEC-RAS Models Developed for VCC TPM 18108

Analyses and calculations for the Q_{CAP} event using a Manning's roughness value of 0.060 are presented to identify the changes in water surface elevation and velocity between the existing and proposed conditions. To establish the top and toe elevations for the soil cement bank protection per LACDPW design requirements, a proposed condition model was developed for two scenarios: (i) full vegetation (Manning's roughness value of 0.085) to estimate top of bank elevations and (ii) no vegetation (Manning's roughness value of 0.025) to estimate toe-down. The HEC-RAS models used in the Project Specific analysis are summarized in **Table 2**.

Table 2 – Castaic Creek HEC-RAS Model Descriptions

HEC-RAS File Name	Description	Purpose
Existing Condition	Existing vegetation, n=0.060	Determine existing condition Capital Flood water surface elevations and flow velocities within Castaic Creek.
Proposed Condition	Existing vegetation, n=0.060	Determine proposed condition Capital Flood water surface elevations and flow velocities to compare with existing conditions for hydraulic impacts.
Proposed Condition	**Full vegetation, n=0.085**	**Determine proposed condition Capital Flood maximum water surface elevations within Castaic Creek.**
Proposed Condition	**No vegetation, n=0.025**	**Determine proposed condition Capital Flood maximum flow velocities within Castaic Creek.**

Notes:
(1) HEC-RAS models for Castaic Creek included in this report are based on 2013 topography data.
(2) Items in bold are HEC-RAS models that are pertinent to the soil cement bank protection design.

The HEC-RAS models were assigned upstream and downstream boundary conditions. The upstream boundary conditions consist of a normal depth with a slope of 0.006. However, the downstream boundary conditions were established using known water surface elevations obtained from the Fluvial Study and the *Valencia Commerce Center – (I-5 to Hwy 126) Castaic Creek HEC-RAS Modeling Study*. **Table 3** lists the boundary conditions used in the HEC-RAS model plans.

Table 3 – HEC-RAS Model Boundary Conditions for Capital Flood (Q_{CAP})

HEC-RAS Model Plan	Downstream Boundary Condition (STA 1981.79)	Upstream Boundary Condition (STA 13371)
Existing Conditions n = 0.060	Known WSE - 969.53	Normal Depth S = 0.006
Proposed Conditions n = 0.060	Known WSE – 969.53	Normal Depth S = 0.006
Proposed Conditions n = 0.085	Known WSE – 971.62	Normal Depth S = 0.006
Proposed Conditions n = 0.025	Known WSE – 966.28	Normal Depth S = 0.006

Notes:

1. *For the Existing Conditions (n=0.06) Plan, the downstream boundary condition was established as a Known WSE at HEC-RAS River Station 1981.79, which was obtained from Appendix A of the Valencia Commerce Center – (I-5 to Hwy 126) Castaic Creek HEC-RAS Modeling Study (See summary output tables for HEC-RAS Plan Exist 0.060)*
2. *For the Proposed Conditions (n=0.06) Plan, the downstream boundary condition was established as a Known WSE value at HEC-RAS River Station 1981.71, which was obtained from Appendix B of the Valencia Commerce Center – (I-5 to Hwy 126) Castaic Creek HEC-RAS Modeling Study (See summary output tables for HEC-RAS Plan Prop 0.060).*
3. *For the Proposed Conditions (n=0.025 and n=0.085) Plans, the downstream boundary conditions were established as a Known WSE at HEC-RAS River Station 1981.71, which were obtained from Appendix 4.2 of the Fluvial Study (See summary output tables for HEC-RAS Plans Prop085 and Prop025nb).*

In an effort to verify that the boundary conditions chosen for the analysis did not have an impact on the area of interest (between the Old Road Bridge and the Commerce Center Drive Bridge as indicated by the red box in Table 4), a sensitivity analysis was performed using the following combinations of boundary conditions:

- *Condition 1* – Normal depth was selected as the upstream boundary condition, and normal depth was selected as the downstream boundary condition.
- *Condition 2* – Critical depth was selected as the upstream boundary condition, and normal depth was selected as the downstream boundary condition.
- *Condition 3* – Normal depth was selected as the upstream boundary condition, and critical depth was selected as the downstream boundary condition.
- *Condition 4* – Critical depth was selected as the upstream boundary condition, and critical depth was selected as the downstream boundary condition.

Table 4 lists the water surface elevations from the models run for the four conditions discussed above. Changing the boundary conditions does not change the water surface elevation within the area of interest. Therefore, the model has been extended far enough above and below the area of concern.

Table 4: Comparison of HEC-RAS Hydraulic Model Boundary Conditions (n=0.06)

HEC-RAS River Station	Proposed Condition WSE[1] (ft)	Condition 1 WSE[2] (ft)	Condition 2 WSE[3] (ft)	Condition 3 WSE[4] (ft)	Condition 4 WSE[5] (ft)
13371	1043.1	1043.1	1043.1	1043.1	1043.1
13141	1042.6	1042.6	1042.6	1042.6	1042.6
12937	1042.3	1042.3	1042.3	1042.3	1042.3
12730	1042.0	1042.0	1042.0	1042.0	1042.0
12543	1041.7	1041.7	1041.7	1041.7	1041.7
12350	1040.7	1040.7	1040.7	1040.7	1040.7
12146	1039.4	1039.4	1039.4	1039.4	1039.4
I-5 Freeway Bridge					
11934	1037.1	1037.1	1037.1	1037.1	1037.1
11923	1037.0	1037.0	1037.0	1037.0	1037.0
Old Road Bridge					
11873	1036.0	1036.0	1036.0	1036.0	1036.0
11846	1035.9	1035.9	1035.9	1035.9	1035.9
11811.76	1035.3	1035.3	1035.3	1035.3	1035.3
11551.18	1033.3	1033.3	1033.3	1033.3	1033.3
11242.98	1031.2	1031.2	1031.2	1031.2	1031.2
10982.86	1028.9	1028.9	1028.9	1028.9	1028.9
10738.26	1026.6	1026.6	1026.6	1026.6	1026.6
10490.82	1023.7	1023.7	1023.7	1023.7	1023.7
10351.28	1020.9	1020.9	1020.9	1020.9	1020.9
10141.82	1018.9	1018.9	1018.9	1018.9	1018.9
9931.16	1017.3	1017.3	1017.3	1017.3	1017.3
9670.57	1014.0	1014.0	1014.0	1014.0	1014.0
9501.81	1012.4	1012.4	1012.4	1012.4	1012.4
9298.99	1011.0	1011.0	1011.0	1011.0	1011.0
9042.81	1009.1	1009.1	1009.1	1009.1	1009.1
8724.84	1006.0	1006.0	1006.0	1006.0	1006.0
8396.13	1003.7	1003.7	1003.7	1003.7	1003.7
8049.79	1002.7	1002.7	1002.7	1002.7	1002.7
7688.71	1002.2	1002.2	1002.2	1002.2	1002.2
7326.46	1001.5	1001.5	1001.5	1001.5	1001.5
6966.68	1000.9	1000.9	1000.9	1000.9	1000.9
6623.50	999.7	999.7	999.7	999.7	999.7
6304.52	998.0	998.0	998.0	998.0	998.0
5992.38	996.5	996.5	996.5	996.5	996.5
5629.45	994.2	994.2	994.2	994.2	994.2
5557.68	994.1	994.1	994.1	994.1	994.1
Commerce Center Drive Bridge					
5453.98	993.8	993.8	993.8	993.8	993.8
5434.68	993.8	993.8	993.8	993.8	993.8
5340.04	992.5	992.5	992.5	992.5	992.5

HEC-RAS River Station	Proposed Condition WSE[1] (ft)	Condition 1 WSE[2] (ft)	Condition 2 WSE[3] (ft)	Condition 3 WSE[4] (ft)	Condition 4 WSE[5] (ft)
5060.55	990.4	990.4	990.4	990.4	990.4
4899.94	989.4	989.4	989.4	989.4	989.4
4571.63	987.9	987.9	987.9	987.9	987.9
4220.87	985.5	985.5	985.5	985.5	985.5
3865.39	982.5	982.5	982.5	982.5	982.5
3579.81	980.4	980.4	980.4	980.4	980.4
3289.64	978.3	978.3	978.3	978.3	978.3
2974.88	973.7	973.6	973.6	973.7	973.7
2627.11	972.6	972.4	972.4	972.6	972.6
2307.93	970.9	970.5	970.5	970.9	970.9
1981.79	969.5	968.2	968.2	965.2	965.2

Notes:

1. *Proposed Conditions n=0.06 WSE – Boundary Conditions shown in **Table 2***
2. *Condition 1 – U/S Boundary Condition: Normal Depth, D/S Boundary Condition: Normal Depth*
3. *Condition 2 – U/S Boundary Condition: Critical Depth, D/S Boundary Condition: Normal Depth*
4. *Condition 3 – U/S Boundary Condition: Normal Depth, D/S Boundary Condition: Critical Depth*
5. *Condition 4 – U/S Boundary Condition: Critical Depth, D/S Boundary Condition: Critical Depth*
6. *The limits of PD 2615 bank protection are indicated by the red box*

4 Summary of HEC-RAS Results

4.1 Existing vs. Proposed Condition (n=0.060)

Results of the hydraulic analysis for the existing and proposed conditions modeling are summarized in **Table 5** for the Q_{CAP} flow rate and Manning's roughness value of 0.060. Comparison of the existing and proposed hydraulic results indicate there are no off-site impacts due to the proposed Castaic Creek bank protection. In addition, the largest on-site impacts occur approximately 1,000 feet upstream of CCD between HEC-RAS River Station 6623.5 and HEC-RAS River Station 6304.52. These on-site impacts are quantified as 2.0 feet rise in water surface elevation and 2.9 feet per second increase in velocity.

The hydraulic impacts associated with the location of the existing concrete slope protection (P.D. No. 1982) and the existing rip-rap slope protection (P.D. No. 2441 Unit II) must also be assessed. For the existing concrete slope protection between HEC-RAS River Station 11551.18 and HEC-RAS River Station 10351.28, there are no hydraulic impacts. In contrast, for the existing rip-rap slope protection located between HEC-RAS River Station 9931.16 and HEC-RAS River Station 8396.13, there are hydraulic impacts, specifically 0.2 feet rise in water surface elevation and 1.9 feet per second increase in velocity. However, the proposed bank protection design provides enough freeboard and toe-down to offset these increases in water surface elevation and velocity.

The existing and proposed conditions HEC-RAS output data are included in **Appendices A and B**, respectively.

Table 5 – Existing vs. Proposed Hydraulic Analysis (Q_{CAP} and n=0.060)

HEC-RAS River Station	Existing Floodplain[1]		Proposed Floodplain[2]		ΔWSE (ft)	ΔVelocity (ft/s)
	WSE (ft)	Velocity (ft/s)	WSE (ft)	Velocity (ft/s)		
12350	1040.7	7.2	1040.7	7.2	0.0	0.0
12146	1039.4	9.0	1039.4	9.0	0.0	0.0
12059.65	**I-5 Freeway Bridge**					
11934	1037.1	10.4	1037.1	10.4	0.0	0.0
11923	1037.0	10.5	1037.0	10.5	0.0	0.0
11917.82	**Old Road Bridge**					
11873	1036.0	10.0	1036.0	10.0	0.0	0.0
11846	1035.9	9.4	1035.9	9.4	0.0	0.0
11811.76	1035.3	10.5	1035.3	10.5	0.0	0.0
11551.18	1033.3	8.5	1033.3	8.5	0.0	0.0
11242.98	1031.2	8.1	1031.2	8.1	0.0	0.0
10982.86	1028.9	9.3	1028.9	9.3	0.0	0.0
10738.26	1026.6	8.6	1026.6	8.5	0.0	-0.1
10490.82	1023.8	7.5	1023.7	7.7	-0.1	0.2
10351.28	1021.3	8.6	1020.9	9.4	-0.4	0.8
10141.82	1019.0	6.9	1018.9	6.8	-0.1	-0.1
9931.16	1017.1	6.3	1017.3	5.9	0.2	-0.4
9670.57	1015.1	5.7	1014.0	7.6	-1.1	1.9
9501.81	1013.8	6.2	1012.4	5.8	-1.4	-0.4
9298.99	1011.8	7.7	1011.0	5.7	-0.8	-2.0
9042.81	1009.4	6.5	1009.1	5.6	-0.3	-0.9
8724.84	1006.1	6.1	1006.0	5.9	-0.1	-0.2
8396.13	1003.5	4.9	1003.7	4.5	0.2	-0.4
8049.79	1002.1	3.9	1002.7	3.4	0.6	-0.5
7688.71	1001.2	3.6	1002.2	3.2	1.0	-0.4
7326.46	1000.1	4.7	1001.6	4.3	1.5	-0.4
6966.68	999.0	3.9	1000.9	4.6	1.9	0.7
6623.5	997.7	5.2	999.7	6.4	2.0	1.2
6304.52	996.5	4.8	998.0	7.7	1.5	2.9
5992.38	995.7	4.5	996.5	6.9	0.8	2.4
5629.45	994.4	6.5	994.2	7.9	-0.2	1.4
5557.68	994.1	6.8	994.1	6.8	0.0	0.0
5500	**Commerce Center Drive Bridge**					
5453.98	993.8	5.6	993.8	5.6	0.0	0.0
5434.68	993.8	5.4	993.8	5.4	0.0	0.0
5340.04	992.5	8.7	992.5	8.7	0.0	0.0
5060.55	990.4	8.6	990.4	8.6	0.0	0.0
4899.94	989.4	7.9	989.4	7.9	0.0	0.0

Notes:

1. *Existing floodplain WSEs and velocities are based on Q_{CAP} flow rate and n = 0.060 (see **Appendix A**)*
2. *Proposed floodplain WSEs and velocities are based on Q_{CAP} flowrate and n = 0.060 (see **Appendix B**)*
3. *Gray highlight indicates existing concrete slope protection (P.D. No. 1982) between RS 11551.18 and RS 10351.28*
4. *Gray highlight indicates existing riprap slope protection (P.D. No. 2441-Unit II) between RS 9931.16 and RS 8396.13*

4.2 Proposed Condition Model Results and Top of Bank Determination

The HEC-RAS analyses of the Q_{CAP} event applying a Manning's roughness value (n) of 0.085 was used to determine the proposed condition water surface elevations for the purpose of determining minimum freeboard based on LACDPW requirements. Summary output tables containing results of the n=0.085 HEC-RAS model is provided in **Appendix C**. Bank protection must be designed to contain the design flood and have adequate freeboard, which represents the additional height required to ensure overtopping does not occur due to factors not accounted for in the design water surface calculations. Calculated freeboard considers possible long-term aggradation, super-elevation at curved reaches of the channel, and bed forms, in addition to less identifiable components such as excessive turbulence, wave action and variation of loss coefficients. The calculated freeboard values and top of bank elevations are summarized in **Tables 6A and 6B**.

Table 6A – Calculated Top Elevations for Soil Cement Bank Protection - West Bank

HEC-RAS Section	Bank Station	WSE for n = 0.085[1] (Appendix C) (ft)	Minimum Required Freeboard[2] (Table E-1) (ft)	Calculated Top Elevation (ft)
10351.28	—	1022.5	2.5	—
10141.82	47+19.31	1020.0	2.5	1022.5
9931.16	45+09.00	1018.1	2.5	1020.6
9670.57	42+37.75	1015.1	2.5	1017.6
9501.81	40+55.59	1013.4	2.5	1015.9
9298.99	38+66.87	1011.9	2.5	1014.4
9042.81	36+02.20	1009.9	2.5	1012.4
8724.84	32+48.27	1007.0	2.5	1009.5
8396.13	29+47.75	1005.3	2.5	1007.8
8049.79	27+41.07	1004.6	2.5	1007.1
7688.71	24+80.72	1004.1	2.5	1006.6
7326.46	21+68.98	1003.5	3.1	1006.5
6966.68	19+02.80	1002.8	3.1	1005.9
6623.5	16+26.07	1001.6	3.3	1004.9
6304.52	13+25.47	999.9	3.4	1003.4
5992.38	—	998.3	3.3	1001.6

Notes:
*(1) Water Surface Elevation (WSE) is from the proposed condition HEC-RAS model with a Manning's roughness value (n) of 0.085 for entire reach. See **Appendix C** for detailed results*
*(2) Freeboard calculations are shown in **Table E-1**, included in **Appendix E**.*

Table 6B – Calculated Top Elevations for Soil Cement Bank Protection - Northeast and Southeast Banks

HEC-RAS Section	Bank Station	WSE for n = 0.085[1] (Appendix C) (ft)	Freeboard[2] (Table E-3) (ft)	Calculated Top Elevation (ft)
11811.76	—	1037.5	2.5	—
11551.18	21+24.18	1035.4	2.5	1037.9
11242.98	19+15.89	1032.8	2.5	1035.3
10982.86	17+14.86	1030.4	2.5	1032.9
10738.26	15+84.44	1027.8	2.5	1030.3
10490.82	14+34.78	1024.8	2.5	1027.3
10351.28	13+17.01	1022.5	2.5	1025.0
10141.82	11+86.54	1020.0	2.5	1022.5
9931.16	—	1018.1	2.5	—
8396.13	—	1005.3	2.5	—
8049.79	39+23.63	1004.6	2.5	1007.1
7688.71	34+21.94	1004.1	2.5	1006.6
7326.46	29+36.40	1003.5	3.1	1006.6
6966.68	24+67.21	1002.8	3.2	1005.9
6623.5	20+38.33	1001.6	3.4	1005.0
6304.52	16+92.74	999.9	3.4	1003.4
5992.38	13+72.73	998.3	3.3	1001.6
5629.45	10+28.85	996.1	3.5	999.6
5557.68	—	996.0	3.3	—

Notes:

*(1) Water Surface Elevation (WSE) is from the proposed condition HEC-RAS model with Manning roughness value (n) of 0.085 for entire reach and included in **Appendix C**.*

*(2) Freeboard calculations are shown in **Table E-3**, included in **Appendix E**.*

(3) The Northeast Bank is between HEC-RAS Sections 11811.76 and 9931.16.

(4) The Southeast Bank is between HEC-RAS Sections 8396.13 and 5557.68.

4.3 Proposed Condition Model Results and Toe of Bank Determination

The HEC-RAS analyses of the Q_{CAP} event applying a Manning's roughness value of 0.025 was used to determine proposed condition flow velocities for the purpose of calculating required toe-down. Summary output tables consisting results of the n=0.025 HEC-RAS model is provided in **Appendix D**. The primary failure mechanism of rigid bank protection revetments is generally scouring at the toe. The bank protection toe-down or cut-off depth must provide adequate scour protection below the earthen creek invert to account for dynamic changes in streambed elevations during storm events. The calculated toe-down accounts for the potential general bed degradation; bend scour, bedform height, abutment or contraction scour, and scour associated with hydraulic structures. Local scour is considered in addition to development of the potential long-term equilibrium channel slope that may develop from general degradation and sediment deposition. The calculated toe-down values and minimum toe of bank elevations are summarized in **Tables 7A and 7B**.

Table 7A – Calculated Toe Elevations for Soil Cement Bank Protection - West Bank

HEC-RAS Section	Bank Station	Minimum Channel Invert[1] (Appendix A) (ft)	Toe-Down[2] (Table E-2) (ft)	Calculated Toe Elevation (ft)
10351.28	—	1011.0	12.5	—
10141.82	47+19.31	1009.0	15.0	994.0
9931.16	45+09.00	1006.0	14.7	991.3
9670.57	42+37.75	1003.0	10.0	993.0
9501.81	40+55.59	1003.0	10.0	993.0
9298.99	38+66.87	1001.0	8.7	992.3
9042.81	36+02.20	998.0	8.0	990.0
8724.84	32+48.27	997.0	8.0	989.0
8396.13	29+47.75	995.0	10.0	985.0
8049.79	27+41.07	993.0	8.0	985.0
7688.71	24+80.72	992.0	6.4	985.6
7326.46	21+68.98	989.0	8.0	981.0
6966.68	19+02.80	986.0	8.0	978.0
6623.5	16+26.07	984.0	10.0	974.0
6304.52	13+25.47	983.0	10.0	973.0
5992.38	—	982.0	10.0	972.0

Notes:
*(1) Minimum Channel Invert is from the existing condition HEC-RAS model and is included in **Appendix A**.*
*(2) Toe-Down calculations are shown in **Table E-2**, included in **Appendix E**.*

Table 7B – Calculated Toe Elevations for Soil Cement Bank Protection – Northeast and Southeast Banks

HEC-RAS Section	Bank Station	Minimum Channel Invert[(1)] (Appendix A) (ft)	Toe-Down[(2)] (Table E-4) (ft)	Calculated Toe Elevation (ft)
11811.76	—	1023.0	11.3	—
11551.18	21+24.18	1018.0	14.0	1004.0
11242.98	19+15.89	1017.0	11.2	1005.8
10982.86	17+14.86	1015.0	10.8	1004.2
10738.26	15+84.44	1012.0	12.5	999.5
10490.82	14+34.78	1010.0	12.5	997.5
10351.28	13+17.01	1011.0	12.5	998.5
10141.82	11+86.54	1009.0	10.0	999.0
9931.16	—	1006.0	8.8	—
8396.13	—	995.0	10.0	—
8049.79	39+23.63	993.0	12.0	981.0
7688.71	34+21.94	992.0	9.7	982.3
7326.46	29+36.40	989.0	12.0	977.0
6966.68	24+67.21	986.0	12.0	974.0
6623.5	20+38.33	984.0	15.0	969.0
6304.52	16+92.74	983.0	10.0	973.0
5992.38	13+72.73	982.0	10.0	972.0
5629.45	10+28.85	982.0	10.0	972.0
5557.68	—	982.0	24.8	—

Notes:

*(1) Minimum Channel Invert is from the existing condition HEC-RAS model, and is included in **Appendix A**.*

*(2) Toe-Down calculations are shown in **Table E-4**, included in **Appendix E**.*

(3) The Northeast Bank is between HEC-RAS Sections 11811.76 and 9931.16.

(4) The Southeast Bank is between HEC-RAS Sections 8396.13 and 5557.68.

5 Fluvial Analysis

5.1 Introduction

The Approved Fluvial Study is a comprehensive analysis in which several methodologies such as the LACFCD Hydraulic Design, the LACDPW Hydrology and Sedimentation Manuals, SAM Hydraulic Design Package, and other long term/local degradation/aggradation methods were compiled and compared. Results from the Project Specific analysis and results from the Fluvial Study were compared, then the most conservative values for top and toe elevations were adopted for the bank protection design. The following is a brief introduction to the Fluvial Study.

The Castaic Creek study reach is located in western Los Angeles County, California, between I-5 and the Santa Clara River. The study reach is between the Santa Clara River (south) and I-5 (northeast) and is upstream of the confluence with Hasley Canyon Creek. Castaic Creek lies within the Castaic Creek Watershed, which has a contributing drainage area of approximately 209 square miles. The existing floodplain generally consists of a natural channel alluvial system extending from Castaic Lake (north) to the Santa Clara River, Castaic Creek's natural terminus (south). Adjacent development along Castaic Creek has been evaluated to determine potential changes in the runoff and reduction in the sediment supply from the developed areas. The proposed buried soil cement bank protection on both the west and east banks of Castaic Creek is intended to provide long-term erosion protection from lateral migration of the bank and flood protection for the adjacent proposed development areas. These modifications to the stream system may result in adjustment to the fluvial operation of the floodplain and changes to the stream mechanics. The intent of this analysis is to evaluate these impacts from (1) fluvial modifications to the river bed from single hypothetical storm events and from (2) changes in the floodplain fluvial operation over the long-term.

5.2 Types of Adjustments

Modifications to the Castaic Creek system are measured as bed adjustment in feet. Positive adjustment indicates bed aggradation while negative adjustment indicates bed degradation. Several types of adjustment are considered in the Fluvial Study including general, long-term, and other scour adjustments. General adjustment consists of scour that occurs in an individual discharge event and may be considered as the difference between sediment inflow and outflow. For example, aggradation describes a situation where sediment inflow is higher than sediment outflow for the same reach. In contrast, if sediment outflow exceeds inflow for a given reach, degradation in the form of scour will occur. Long-term adjustment consists of fluvial processes that occur over many rainy seasons and contribute to fluctuation of bed elevation of a river or creek. Other scour types include local scour, bend scour, low-flow incising, and bedform formation. These adjustments are discussed in **Section 4.3**.

5.3 Objectives

The primary objective of the Fluvial Study was to assess creek bed impacts from potential modifications of fluvial operation from the proposed TPM No. 18108 development. The intent was to provide a comprehensive assessment of short-term and long-term bed adjustment based on the level of information available. The Fluvial Study describes the following: (1) general adjustment, (2) long-term adjustment, (3) other scour, (4) study reach gradation, (5) SAM modeling and analysis, and (6) total scour potential for the purpose of determining soil cement bank protection toe-down. The objectives of the fluvial assessment for the proposed development project included:

1. Quantify fluvial parameters that are representative of the creek bed characteristics;
2. Model the existing and proposed conditions;
3. Conduct a preliminary assessment of streambed stability; and,
4. Estimate toe-down depth assessment.

The final design top and toe elevations from the Fluvial Study are included in **Appendix F**.

6 Proposed Bank Top and Bank Toe Elevations

In the previously approved Fluvial Study, the bank top and toe elevations were determined based on HEC-RAS modeling of the existing and proposed conditions. Excerpts of the Fluvial Study are provided in **Appendix F**.

The hydraulic analysis presented in this report is based on the approved HEC-RAS model from the Fluvial Study and has been updated to reflect the current alignment for the VCC Industrial Park Development in TPM No. 18108. The project specific analysis provided in this report involved determining the bank top and toe elevations based on the current project conditions.

6.1 Proposed Bank Top and Toe Elevations

Results of the analysis described in **Section 5** were compared to the Fluvial Study. In an effort to satisfy the minimum requirements established in this report, as well as the Fluvial Study, the more conservative values were selected as the design top and toe elevations. **Tables 8A and 8B** list the calculated top and toe elevations (see **Section 5**), the proposed top and toe elevations from the Fluvial Study, and the design elevations for each HEC-RAS section.

Table 8A – Proposed Top and Toe Elevations for Soil Cement Bank Protection - West Bank

HEC-RAS Section	Bank Station	Calculated Toe Elevation(1) (ft)	Proposed Toe Elevation(2) (ft)	Design Toe Elevation(3) (ft)	Calculated Top Elevation(1) (ft)	Proposed Top Elevation(2) (ft)	Design Top Elevation(4) (ft)	Proposed Min. Bank Height (ft)
10351.28	—	—	—	—	—	—	—	—
10141.82	47+19.31	994.0	1000.0	994.0	1022.5	1022.5	1022.5	28.5
9931.16	45+09.00	991.3	998.0	991.3	1020.6	1020.6	1020.6	29.3
9670.57	42+37.75	993.0	996.0	993.0	1017.6	1018.9	1018.9	25.9
9501.81	40+55.59	993.0	994.0	993.0	1015.9	1017.6	1017.6	24.6
9298.99	38+66.87	992.3	993.2	992.3	1014.4	1015.7	1015.7	23.5
9042.81	36+02.20	990.0	990.0	990.0	1012.4	1013.2	1013.2	23.2
8724.84	32+48.27	989.0	990.0	989.0	1009.5	1009.7	1009.7	20.7
8396.13	29+47.75	985.0	986.0	985.0	1007.8	1008.2	1008.2	23.2
8049.79	27+41.07	985.0	984.0	984.0	1007.1	1007.5	1007.5	23.5
7688.71	24+80.72	985.6	982.0	982.0	1006.6	1006.9	1006.9	24.9
7326.46	21+68.98	981.0	980.0	980.0	1006.5	1006.9	1006.9	26.9
6966.68	19+02.80	978.0	978.5	978.0	1005.9	1006.1	1006.1	28.1
6623.50	16+26.07	974.0	977.2	974.0	1004.9	1005.2	1005.2	31.2
6304.52	13+25.47	973.0	974.0	973.0	1003.4	1003.6	1003.6	30.6
5992.38	—	972.0	972.0	972.0	1001.6	1001.6	1001.6	29.6

Notes:

*(1) The Calculated Toe and Top Elevations are from "Project Specific" analysis. Refer to **Table 6A and 7A**.*

*(2) The Proposed Toe and Proposed Top Elevations are from LACDPW approved report, "Creek Fluvial Study Phase 1 Final Draft: Castaic Creek," dated January 2006. See **Appendix F**.*

(3) The Design Toe Elevation is the lesser of the Calculated Toe Elevation and the Proposed Toe Elevation.

(4) The Design Top Elevation is the greater of the Calculated Top Elevation and the Proposed Top Elevation.

Table 8B – Proposed Top and Toe Elevations for Soil Cement Bank Protection – Northeast and Southeast Banks

HEC-RAS Section	Bank Station	Calculated Min. Toe Elevation[1] (ft)	Proposed Toe Elevation[2] (ft)	Design Toe Elevation[3] (ft)	Calculated Top Elevation[1] (ft)	Proposed Top Elevation[2] (ft)	Design Top Elevation[4] (ft)	Proposed Min. Bank Height (ft)
11811.76	—	—	—	—	—	—	—	—
11551.18	21+24.18	1004.0	1008.0	1004.0	1037.9	1036.2	1037.9	33.8
11242.98	19+15.89	1005.8	1007.5	1005.8	1035.3	1033.6	1035.3	29.6
10982.86	17+14.86	1004.2	1006.0	1004.2	1032.9	1031.4	1032.9	28.7
10738.26	15+84.44	999.5	1003.5	999.5	1030.3	1029.4	1030.3	30.8
10490.82	14+34.78	997.5	1002.0	997.5	1027.3	1027.1	1027.3	29.8
10351.28	13+17.01	998.5	999.5	998.5	1025.0	1025.1	1025.1	26.6
10141.82	11+86.54	999.0	1000.0	999.0	1022.5	1022.5	1022.5	23.6
9931.16	—	—	—	—	—	—	—	—
8396.13	—	—	—	—	—	—	—	—
8049.79	39+23.63	981.0	**980.0**	980.0	1007.1	1007.5	1007.5	27.5
7688.71	34+21.94	982.3	**978.0**	978.0	1006.6	1006.9	1006.9	28.9
7326.46	29+36.40	977.0	**976.0**	976.0	1006.6	1006.9	1006.9	30.9
6966.68	24+67.21	974.0	974.5	974.0	1005.9	1006.2	1006.2	32.2
6623.50	20+38.33	969.0	973.2	969.0	1005.0	1005.3	1005.3	36.3
6304.52	16+92.74	973.0	974.0	973.0	1003.4	1003.6	1003.6	30.6
5992.38	13+72.73	972.0	972.0	972.0	1001.6	1001.6	1001.6	29.6
5629.45	10+28.85	972.0	972.1	972.0	999.6	1000.2	1000.2	28.2
5557.68	—	—	—	—	—	—	—	—

Notes:

*(1) The Calculated Toe and Top Elevations are from "Project Specific" analysis. Refer to **Tables 6B and 7B**.*

*(2) The Proposed Toe and Proposed Top Elevations are from LACDPW approved report, "Creek Fluvial Study Phase 1 Final Draft: Castaic Creek," dated January 2006. Refer to **Appendix F**.*

(3) The Design Toe Elevation is the lesser of the Calculated Toe Elevation and the Proposed Toe Elevation.

(4) The Design Top Elevation is the greater of the Calculated Top Elevation and the Proposed Top Elevation.

The profiles for the bank top and toe shown on **Figures 6, 7 and 8** were initially created using the design elevations. Then the bank top elevations were modified to minimize the changes in grade along the top of the proposed bank protection. Locations of grade breaks were selected to ensure the top of bank met or exceeded the freeboard requirements. The design bank top and toe elevations as well as the actual top elevations are presented in **Tables 9A, 9B and 9C**.

Table 9A – Castaic Creek Soil Cement Bank Protection Top and Toe Elevations – West Bank

HEC-RAS River Station	Bank Station (ft)	Design Top Elevation[1] (ft)	Actual Top Elevation Per Plan[2] (ft)	Design Min. Toe Elevation[3] (ft)	Bank Height (ft)
10351.28	50+75.09	1025.0	—	994.0	—
End SC Top	50+00.57	—	**1024.8**	—	—
10141.82	47+19.31	1022.5	**1022.7**	994.0	28.7
9931.16	45+09.00	1020.6	1021.1	991.3	29.8
9670.57	42+37.75	1018.9	1019.1	993.0	26.1
9501.81	40+55.59	1017.6	**1017.7**	993.0	24.7
9298.99	38+66.87	1015.7	1015.9	992.3	23.6
9042.81	36+02.20	1013.2	1013.3	990.0	23.3
8724.84	32+48.27	1009.7	**1009.8**	989.0	20.8
8396.13	29+47.75	1008.2	1008.5	985.0	23.5
8049.79	27+41.07	1007.5	**1007.6**	984.0	23.6
7688.71	24+80.72	1006.9	1007.3	982.0	25.3
7326.46	21+68.98	1006.9	**1007.0**	980.0	27.0
6966.68	19+02.80	1006.1	1006.2	978.0	28.2
6623.50	16+26.07	1005.2	**1005.3**	974.0	31.3
6304.52	13+25.47	1003.6	1004.0	973.0	31.0
5992.38	10+15.77	1001.6	1002.7	972.0	30.7
Begin SC Top	10+00.00	—	**1002.6**	—	—

Notes:

1. *Design Top Elevation values are obtained from **Table 8A**.*
2. *Actual Top Elevation values shown in **bold** font indicate Grade Break locations shown on **Figure 6**.*
3. *Design Toe Elevations are obtained from **Table 8A**. Actual Toe Elevation values are determined based on the launch stone design discussed in **Section 7.4**. See **Table 11A** for Actual Toe Elevation values.*
4. *Bank Height is the Actual Top Elevation – Design Min. Toe Elevation*

Table 9B – Castaic Creek Soil Cement Bank Protection Top and Toe Elevations – Northeast Bank

HEC-RAS River Station	Bank Station (ft)	Design Top Elevation[1] (ft)	Actual Top Elevation Per Plan[2] (ft)	Design Min. Toe Elevation[3] (ft)	Minimum Bank Height (ft)
11811.76	23+68.64	1040.0	—	1011.7	—
End SC Top	23+12.81	—	**1039.6**	—	—
11551.18	21+24.18	1037.9	**1038.0**	1004.0	33.8
11242.98	19+15.89	1035.3	1035.4	1005.8	29.6
10982.86	17+14.86	1032.9	**1033.0**	1004.2	28.7
10738.26	15+84.44	1030.3	1030.4	999.5	30.8
10490.82	14+34.78	1027.3	1027.5	997.5	29.8
10351.28	13+17.01	1025.1	**1026.0**	998.5	26.6
10141.82	11+86.54	1022.5	1026.0	999.0	23.6
Begin SC Top	10+00	—	**1026.9**	—	—
9931.16	9+67.55	1020.6	—	997.2	—

Notes:

1. *Design Top Elevation values are obtained from **Table 8B**.*
2. *Actual Top Elevation values shown in **bold** font indicate Grade Break locations shown on **Figure 8**.*
3. *Design Toe Elevations are obtained from **Table 8B**. Actual Toe Elevation values are determined based on the launch stone design discussed in **Section 7.4**. See **Table 11B** for Actual Toe Elevation values.*

Table 9C – Castaic Creek Soil Cement Bank Protection Top and Toe Elevations – Southeast Bank

HEC-RAS River Station	Bank Station (ft)	Design Top Elevation[1] (ft)	Actual Top Elevation Per Plan[2] (ft)	Design Min. Toe Elevation[3] (ft)	Minimum Bank Height (ft)
8396.13	44+03.95	1007.8	—	985.0	—
End SC Top	40+20.00	—	**1018.9**	—	—
8049.79	39+23.63	1007.5	1017.0	980.0	27.5
7688.71	34+21.94	1006.9	**1007.0**	978.0	28.9
7326.46	29+36.40	1006.9	**1007.0**	976.0	30.9
6966.68	24+67.21	1006.2	1006.4	974.0	32.2
6623.50	20+38.33	1005.3	**1005.8**	969.0	36.3
6304.52	16+92.74	1003.6	1004.2	973.0	30.6
5992.38	13+72.73	1001.6	**1002.8**	972.0	29.6
5629.45	10+28.85	1000.2	995.6	972.0	28.2
Begin SC Top	9+98.72	—	**994.10**	—	—
5557.68	9+86.53	999.3	—	957.2	—

Notes:

1. *Design Top Elevation values are obtained from **Table 8B**.*
2. *Actual Top Elevation values shown in **bold** font indicate Grade Break locations shown on **Figure 7**.*
3. *Design Toe Elevations are obtained from **Table 8B**. Actual Toe Elevation values are determined based on the launch stone design discussed in **Section 7.4**. See **Table 11C** for Actual Toe Elevation values.*

7 Soil Cement Bank Protection

7.1 Bank Protection Project Description

The proposed bank protection will consist of conventional soil cement bank protection combined with launch stone to provide the appropriate level of freeboard and scour protection for all storm events up to the Capital Flood (Q_{CAP}). The following criteria was considered when designing the bank protection:

1. Flood control stability;
2. Durability of bank protection;
3. Safety concerns regarding access to and from the channel;
4. Bank protection maintenance;
5. Environmental compatibility with the native area and aesthetics;
6. Constructability; and
7. Cost of construction.

The purpose of the proposed bank protection is to provide erosion and flood control protection. The soil cement bank protection will be completely buried with soil backfill sloped at 3H:1V over the soil cement face. The excavation required to construct the bank protection will be backfilled and returned to existing grade or slightly lower to facilitate vegetation regrowth. Soil cement bank protection is constructed as a monolithic and homogenous structure consisting of approximately 90% native soils and 10% cement. The typical section consists of 8-foot wide and 6- to 12-inch thick layers of soil cement. Each layer of soil cement is set back from the edge of the previous layer, at a 1.5H:1V slope. The entire section varies in total height based on varying freeboard, flow depth and toe-down requirements.

The proposed soil cement bank protection addresses the above design criteria as follows:

1. Soil cement provides a stable riverbank protection material, in terms of both surface erosion and structural stability. Preliminary geotechnical analyses indicate that locally available native soils are considered acceptable for use in soil cement.

2. The soil cement bank protection will be completely buried with a 3:1 slope soil backfill. Due to transitions from proposed soil cement bank protection (buried condition) to existing concrete and rip-rap bank protection (exposed conditions), some soil cement areas will be exposed.

7.2 Soil Cement Bank Protection Design Elements

This section describes design elements related to the proposed soil cement bank protection system along Castaic Creek associated with the VCC development project.

The proposed improvements along Castaic Creek consist of a total of approximately 8,400 lineal feet (LF) of bank protection. The west bank will consist of approximately 4,000 LF of bank protection. The proposed bank protection for the east bank will be constructed in two separate sections. The northeast section will be approximately 1,400 LF, and the southeast section will be approximately 3,000 LF.

7.2.1 West Bank Soil Cement Connections

The west bank soil cement protection will tie into the existing concrete bank protection per P.D. No. 1982. Soil cement placed behind the existing concrete bank protection shall be hand compacted using a vibrator plate to ensure compaction requirements are met and to achieve a good seal. Approximately 550 feet upstream of CCD Bridge, the west bank soil cement protection will join to the future Lower Hasley soil cement bank protection per TPM No. 18108 DCR Volume III of V.

7.2.2 Northeast Bank Soil Cement Connections

The northeast bank soil cement protection will tie into existing un-grouted rip-rap slope protection at the Old Road Bridge. This un-grouted rip-rap section was designed to protect the bridge abutment from being undermined. Approximately 2,000 feet downstream of the Old Road Bridge, the northeast bank protection will join with existing un-grouted rip-rap bank protection per P.D No. 2441 Unit II. The un-grouted rip-rap section will be extended to overlap 20 feet onto the soil cement for additional protection at the join location.

7.2.3 Southeast Bank Soil Cement Connections

At the downstream end, the southeast bank will tie into the existing concrete slope protection at CCD Bridge per P.D. No 2298 Unit III. Portions of the existing concrete slope protection for the bridge will be removed or demolished as necessary to install soil cement end connections. The soil cement will overlap behind the existing concrete slope protection by two feet. Hand compaction of soil cement with mechanical vibrator will be required at join locations. Some existing rip-rap will be removed and stockpiled to allow for soil cement bank protection construction. The rip-rap may be used in other locations within the project, if rock specifications are met. The southeast bank soil cement protection will extend from the CCD Bridge and will join the existing un-grouted rip-rap bank protection per P.D No. 2441 Unit II.

7.2.4 Storm Drain Penetrations

There will be several storm drains from the proposed on-site development penetrating through the soil cement bank protection. The storm drain lines will be penetrating through the bank protection on both east and west banks. Storm drain penetrations are typically constructed after the soil cement bank protection has been completed. A section of the soil cement bank protection is removed in order to install storm drain pipe and outlet. The area around the pipe is then backfilled with concrete slurry up to the top of bank. Storm drain pipes can also be installed concurrently with the soil cement bank protection, but may not be practical in some instances, as it slows down the soil cement construction operation.

7.2.5 Maintenance Access Road

The bank protection incorporates a 16-foot wide multi-purpose maintenance/pedestrian access road that follows the alignment of the bank protection. The maintenance road is located directly above the uppermost layer of soil cement. The finished surface will be sloped away from the creek into an earthen swale.

7.2.6 Horizontal and Vertical Scour Gauges

In order to measure abrasion of the soil cement, several scour gauges have been incorporated into the bank protection system. There are two ways that scour gauges are installed, either they are core drilled horizontally into the soil cement banks after completion of the project or installed concurrently during soil cement construction. If scour gauges are core drilled, they will be filled in with grout to secure scour gauge inside soil cement bank. In addition, vertical scour gauges are also installed adjacent to the toe alignment along the river bottom. The vertical scour gauges will be used to measure scour or degradation of the river bottom in the event of any significant storm event.

7.2.7 Retaining Wall Connection at Commerce Center Drive Bridge and Live Oak Road

To provide adequate flood protection at the location where the 16-foot multipurpose trail ramps down to join the existing 12-foot wide bike path under the bridge, a Type-1 reinforced concrete retaining wall is proposed on the southeast side to join the existing concrete wall at CCD Bridge. The retaining wall will provide adequate flood protection and complete the bank protection and bridge connection.

7.3 Material Suitability

The following soil cement material suitability analysis excerpts were taken from various ACI and PCA publications referenced in **Section 11.**

The erosion and overtopping of river banks by floodwaters is a significant flood concern in the southwest. In order to protect valuable land, bridges, and buildings from such erosion, many communities have found that stabilized soil, in the form of soil cement, meets the criteria of cost-effectiveness, performance, functional life, and aesthetics.

The American Concrete Institute (ACI 116R) defines soil cement as "a mixture of soil and measured amounts of Portland Cement and water compacted to a high density." Soil cement can be further defined as a material produced by blending, compacting, and curing a mixture of soil/aggregate, Portland Cement, possibly admixtures including pozzolans, and water, to form a hardened material with specific engineering properties. The soil/aggregate particles are bonded by cement paste, but unlike concrete, the individual particle is not completely coated with cement paste.

A wide variety of soils can be used to make durable soil cement. In fact, most soils in Southern California are suitable for use with soil cement. Ideally, sand to silty sand with the highest dry unit weight possible should be used to make soil cement, as this material is more likely to be well graded. For maximum economy and most efficient construction, the Portland Cement Association (PCA) recommends the following soil gradation:

Sieve Size	% Passing
3/4"	80% - 100%
#4	60% - 90%
#40	30% - 50%
#200	5% - 25%

The Plasticity Index (PI) of the fines should not exceed 8.

The following is the soil classification, cement content for the borrow area/earthwork and groundwater levels as stated in the geotechnical report for TPM No. 18108 dated May 31, 2007, prepared by GeoLabs:

7.3.1 Soil Classification

Most of the native on-site soils are expected to be suitable as aggregate for the soil cement mix. However, the existing fill soils and stockpiled soils from Tentative Parcel Map 18108 are not expected to be suitable for use as the aggregate base for soil cement.

The ideal material calls for a well-graded sand with some gravels and fine sands, and no or minimal clays. The gradation test data of various borings, provided in the geotechnical report, were used to determine the location of the soils that contain suitable material for use in soil cement mix.

7.3.2 Cement Content

An efficient mix utilizes the minimum cement content necessary to reach the specified design strengths. Based on strength and durability relationships developed by the Portland Cement Association, the minimum requirement for compressive strength at 7-day is 750 psi. A cement content of 8% to 10%, by weight, is anticipated based on several projects within the vicinity. The final cement content will be based on testing results from the actual base material stockpile.

7.3.3 Aggregate Location

The project Geotechnical Engineer has performed several borings along the existing creek banks to analyze soil gradations and determine if they are suitable for soil cement base. Based on a myriad of tests performed by PCA, the ideal base material for soil cement, in general, is well-graded sand with

some gravel and some fines. The soils that are extracted from the trench excavations typically are well graded sands and therefore used for the soil cement mix.

7.3.4 Groundwater Levels

All dewatering activities will be limited to the area from the soil cement grading daylight line to a maximum of 15 feet towards the creek. The contractor may request in writing an extension of the area for dewatering, except that in no case shall any dewatering or construction activities take place within the United States Army Corps of Engineers (ACOE) or California Department of Fish and Wildlife (CDFW) resource area without prior authorization.

Any groundwater data should not be assumed to be accurate during the period of construction. The contractor is responsible for obtaining and reviewing copies of the project geotechnical report and should independently determine groundwater levels and appropriate dewatering efforts necessary to prepare excavation for soil cement construction.

7.4 Design of Launch Stone Protection

Launch stone (rip-rap apron) will be utilized in conjunction with soil cement to provide the necessary level of flood and erosion protection for the VCC project site because future groundwater levels could be higher than the toe of the proposed bank protection. Launch stone provides an alternative to traditional river bank protection construction in cases where the presence of high groundwater may prevent soil cement from being installed to the required cut off depth without dewatering. An apron of launch stone will be placed adjacent to the face of soil cement bank and is designed to replace the volume of soil cement not placed due to high groundwater encountered during construction.

7.4.1 Required Volume of Launch Stone

The procedure for designing the launch stone apron begins with determining the minimum required volume of rip-rap material. The required volume of rip-rap is calculated based on the volume of soil cement not placed due to high groundwater encountered during construction. Groundwater levels are assumed to be approximately 2-ft below the river thalweg elevation. The unit volume of rip rap is calculated at each cross-section using the groundwater elevation and the design toe of bank elevation. The required volume of launch stone is equivalent to 2.2 times the unit volume of soil cement not placed below the groundwater level. In accordance with LACDPW requirements, the design toe of bank protection is based on the required cut off depth, which is calculated using velocity results from the HEC-RAS model applying a the Q_{CAP} discharge, a Manning's roughness value of 0.025, and a mixed flow regime.

Dimensions of the launch stone apron are determined based on physical constraints in the vicinity of the proposed bank protection. The bottom width of the launch stone apron is calculated by taking the required unit volume (cross-sectional area) and rearranging the area equation of a square piece plus a triangle piece to solve for bottom width. The equation to calculate the minimum bottom width of the launch stone apron is:

$$b = \frac{A - \frac{1}{2}zh^2}{h}$$

where,

b = bottom width (ft);
A = minimum required area (ft^2);
z = side slope ratio of soil cement (H:V); and,
h = height of launch stone apron (ft).

A typical section of launch stone combined with soil cement bank protection is shown on **Figure 3**. The configuration of the launch stone apron is constrained both vertically and horizontally. Vertically, it is preferable to have the launch stone installed in a buried condition, and horizontally, the launch stone

apron must minimize, if not avoid, encroachment of jurisdictional boundaries set by the United States Army Corps of Engineers (ACOE) and the California Department of Fish and Wildlife (CDFW). A constant height (thickness) of nine feet was selected for the launch stone rip-rap apron. This height allows the apron to be buried while minimizing, and in some cases completely avoiding, encroachment of jurisdictional boundaries within the Santa Clara River. **Tables 9A, 9B, and 9C** summarize the calculations to determine the volume and bottom width of the launch stone apron. The minimum required bottom width of the launch stone apron ranges from 10.8 to 33.9 feet. The actual bottom width provided either meets or exceeds the minimum required quantity of launch stone. **Tables 10A**, **10B, and 10C** list the actual toe of soil cement/bottom of launch stone elevations based on the launch stone apron configuration.

Table 10A – Launch Stone Volume Calculation (West Bank)

Bank Station	Design Toe Elevation[1] (ft)	Revised Toe of Bank Elevation[2] (ft)	Unit Volume of Soil Cement Below Groundwater (ft^3/ft)	Min. Required Launch Stone Unit Volume (ft^3/ft)	Min. Required Bottom Width of Launch Stone (ft)	Actual Bottom Width of Launch Stone (ft)
50+00	997.57	1008.56	87.9	193.4	14.7	15.0
48+00	995.03	1007.43	99.3	218.4	17.5	18.0
46+00	992.48	1005.49	104.1	228.9	18.7	19.0
44+00	992.00	1003.00	88.0	193.6	14.8	15.0
42+00	993.00	1001.00	64.0	140.8	8.9	9.0
40+00	992.78	1000.43	61.2	134.6	8.2	9.0
38+00	991.69	998.36	53.4	117.5	6.3	7.0
36+00	989.99	996.30	50.4	111.0	5.6	6.0
34+00	989.43	995.29	46.9	103.2	4.7	5.0
32+00	988.36	994.27	47.3	104.1	4.8	5.0
30+00	985.70	993.26	60.5	133.1	8.0	9.0
28+00	984.29	991.90	60.9	134.0	8.1	9.0
26+00	982.92	990.43	60.1	132.2	7.9	8.0
24+00	981.48	988.78	58.4	128.4	7.5	8.0
22+00	980.20	986.86	53.3	117.2	6.3	7.0
20+00	978.73	984.94	49.7	109.3	5.4	6.0
18+00	976.51	983.41	55.2	121.4	6.7	7.0
16+00	973.91	982.25	66.7	146.7	9.6	10.0
14+00	973.25	981.08	62.7	137.8	8.6	9.0
12+00	972.88	980.25	59.0	129.7	7.7	8.0
10+00	972.93	979.60	53.4	117.4	6.3	7.0

Notes:

*1. Design Toe Elevation values for full depth soil cement bank protection construction are interpolated from values in **Table 8A**.*

2. Revised Toe of Bank Elevation is the groundwater elevation (assumed as approximately 2 feet below the minimum channel invert (thalweg) elevation).

3. Launch stone sizing to be confirmed with improvement plans.

Table 10B – Launch Stone Volume Calculation (Northeast Bank)

Bank Station	Design Toe Elevation[1] (ft)	Revised Toe of Bank Elevation[2] (ft)	Unit Volume of Soil Cement Below Groundwater (ft^3/ft)	Min. Required Launch Stone Unit Volume (ft^3/ft)	Min. Required Bottom Width of Launch Stone (ft)	Actual Bottom Width of Launch Stone (ft)
23+00	1009.39	1019.65	82.0	180.5	13.3	14.0
22+00	1006.33	1017.74	91.3	200.9	15.6	16.0
21+00	1004.21	1016.09	95.1	209.2	16.5	17.0
20+00	1005.05	1015.27	81.7	179.8	13.2	14.0
19+00	1005.64	1014.45	70.5	155.0	10.5	11.0
18+00	1004.84	1013.63	70.3	154.6	10.4	11.0
17+00	1003.63	1012.65	72.1	158.7	10.9	11.0
16+00	1000.06	1010.79	85.9	188.9	14.2	15.0
15+00	998.37	1008.92	84.4	185.7	13.9	14.0
14+00	997.80	1007.60	78.4	172.6	12.4	13.0
13+00	998.56	1007.32	70.1	187.4	10.4	11.0
12+00	998.94	1007.04	64.8	142.6	9.1	10.0
11+00	998.28	1005.81	60.2	132.5	8.0	8.0

Notes:

*1. Design Toe Elevation values for full depth soil cement bank protection construction are interpolated from values in **Table 8B**.*
2. Revised Toe of Bank Elevation is the groundwater elevation (assumed as approximately 2 feet below the minimum channel invert (thalweg) elevation).
3. Launch stone sizing to be confirmed with improvement plans.

Table 10C – Launch Stone Volume Calculation (Southeast Bank)

Bank Station	Design Toe Elevation[1] (ft)	Revised Toe of Bank Elevation[2] (ft)	Unit Volume of Soil Cement Below Groundwater (ft^3/ft)	Min. Required Launch Stone Unit Volume (ft^3/ft)	Min. Required Bottom Width of Launch Stone (ft)	Actual Bottom Width of Launch Stone (ft)
39+00	979.91	990.95	88.4	194.4	14.8	15.0
37+00	979.11	990.55	91.5	201.4	15.6	16.0
35+00	978.31	990.16	94.8	208.5	16.4	17.0
33+00	977.50	989.23	93.9	206.5	16.2	17.0
31+00	976.67	987.98	90.4	199.0	15.4	16.0
29+00	975.84	986.72	87.0	191.4	14.5	15.0
27+00	974.99	985.46	83.7	184.2	13.7	14.0
25+00	974.14	984.21	80.6	177.2	12.9	13.0
23+00	972.05	983.39	90.7	199.6	15.4	16.0
21+00	969.72	982.66	103.5	227.8	18.6	19.0
19+00	970.60	981.93	90.6	199.4	15.4	16.0
17+00	972.92	981.20	66.3	145.8	9.4	10.0
15+00	972.40	980.47	64.6	142.1	9.0	10.0
13+00	965.35	979.99	117.1	257.6	21.9	22.0
11+00	947.08	979.97	263.1	578.8	57.6	58.0

Notes:

*1. Design Toe Elevation values for full depth soil cement bank protection construction are interpolated from values in **Table 8B**.*
2. Revised Toe of Bank Elevation is the groundwater elevation (assumed as approximately 2 feet below the minimum channel invert (thalweg) elevation).
3. Launch stone sizing to be confirmed with improvement plans.

Table 11A – Launch Stone Toe Elevations (West Bank)

HEC-RAS River Station	Bank Station (ft)	Minimum Channel Invert Elevation (ft)	Toe of Launch Stone Elevation[1] (ft)
End LS Toe	50+17.00	1010.7	**1008.66**
10141.82	47+19.31	1009.0	**1006.98**
9931.16	45+09.00	1006.0	1004.33
9670.57	42+37.75	1003.0	**1001.00**
9501.81	40+55.59	1003.0	**1001.00**
9298.99	38+66.87	1001.0	999.05
9042.81	36+02.20	998.0	**996.32**
8724.84	32+48.27	997.0	994.52
8396.13	29+47.75	995.0	**992.99**
8049.79	27+41.07	993.0	991.47
7688.71	24+80.72	992.0	**989.56**
7326.46	21+68.98	989.0	986.56
6966.68	19+02.80	986.0	**984.01**
6623.5	16+26.07	984.0	982.40
6304.52	13+25.47	983.0	**980.65**
Begin LS Toe	10+00.00	982.0	**979.60**

Notes:

1. *Revised Toe of Bank Elevation is equal to the Minimum Channel Invert (Thalweg) Elevation minus approximately 2 feet. Values shown in bold font indicate Grade Break locations shown on **Figure 6**. Revised Toe of Bank Elevations were interpolated between grade breaks.*

Table 11B – Launch Stone Toe Elevations (Northeast Bank)

HEC-RAS River Station	Bank Station (ft)	Minimum Channel Invert Elevation (ft)	Toe of Launch Stone Elevation[1] (ft)
End LS Toe	23+41.56	1022.4	**1020.45**
11551.18	21+24.18	1018.0	**1016.29**
11242.98	19+15.89	1017.0	1014.58
10982.86	17+14.86	1015.0	**1012.93**
10738.26	15+84.44	1012.0	1010.49
10490.82	14+34.78	1010.0	**1007.70**
10351.28	13+17.01	1011.0	1007.37
10141.82	11+86.54	1009.0	**1007.00**
Begin LS Toe	10+31.68	1006.9	**1004.88**

Notes:

1. *Revised Toe of Bank Elevation is equal to the Minimum Channel Invert (Thalweg) Elevation minus approximately 2 feet. Values shown in bold font indicate Grade Break locations shown on **Figure 8**. Revised Toe of Bank Elevations were interpolated between grade breaks.*

Table 11C – Launch Stone Toe Elevations (Southeast Bank)

HEC-RAS River Station	Bank Station (ft)	Minimum Channel Invert Elevation (ft)	Toe of Launch Stone Elevation[1] (ft)
End LS Toe	39+64.53	993.2	**991.17**
8049.79	39+23.63	993.0	**991.00**
7688.71	34+21.94	992.0	**990.00**
7326.46	29+36.40	989.0	986.95
6966.68	24+67.21	986.0	**984.00**
6623.5	20+38.33	984.0	982.43
6304.52	16+92.74	983.0	981.17
5992.38	13+72.73	982.0	**980.00**
5629.45	10+28.85	983.0	980.00
Begin LS Toe	9+98.72	937.8	**980.00**

Notes:

1. *Revised Toe of Bank Elevation is equal to the Minimum Channel Invert (Thalweg) Elevation minus approximately 2 feet. Values shown in bold font indicate Grade Break locations shown on* ***Figure 7****. Revised Toe of Bank Elevations were interpolated between grade breaks.*

7.4.2 Launch Stone Sizing

The launch stone apron was sized using methodology outlined in the memorandum presented in **Appendix I**. In summary, the memorandum discusses how Cowan's method was used for selecting the appropriate Manning's roughness value to apply for launch stone design calculations. A sensitivity analysis indicated that the appropriate Manning's roughness value was 0.035 for the launch stone apron design for the San Jose Flats project, which is a similar bank protection project along the Santa Clara River. Therefore, a Manning's roughness value of 0.035 was applied to the hydraulic model used to perform the launch stone design calculations for this project.

Then the rip-rap sizing chart on page F-32 of the LACFCD Hydraulic Design Manual was used to size rip-rap based on the velocities at each HEC-RAS cross-section. The rip-rap sizes determined for the study reach were then adjusted for consistency and to provide smooth transitions between varying rip-rap sizes. **Tables 12A, 12B, and 12C** summarize the rip-rap sizing for the soil cement bank protection.

Table 12A – Launch Stone Sizing Calculated (Q_{cap} and n=0.035) for the West Bank

HEC-RAS River Station	Bank Station (ft)	Velocity[1] (fps)	Launch Stone Size[2] (D_{50})	Actual Launch Stone Size[3] (D_{50})
10141.82	47+19.31	9.9	150 lb. (15")	1/2-Ton (27")
9931.16	45+09.00	7.8	100 lb. (12")	1/2-Ton (27")
9670.57	42+37.75	9.5	150 lb. (15")	1/2-Ton (27")
9501.81	40+55.59	8.1	100 lb. (12")	1/2-Ton (27")
9298.99	38+66.87	7.8	100 lb. (12")	1/2-Ton (27")
9042.81	36+02.20	7.6	100 lb. (12")	1/2-Ton (27")
8724.84	32+48.27	7.9	100 lb. (12")	1/2-Ton (27")
8396.13	29+47.75	7.7	100 lb. (12")	1/2-Ton (27")
8049.79	27+41.07	5.6	50 lb. (10")	1/2-Ton (27")
7688.71	24+80.72	4.4	50 lb. (10")	1/2-Ton (27")
7326.46	21+68.98	5.9	50 lb. (10")	1/2-Ton (27")
6966.68	19+02.80	5.8	50 lb. (10")	1/2-Ton (27")
6623.5	16+26.07	8.8	100 lb. (12")	1/2-Ton (27")
6304.52	13+25.47	10.4	300 lb. (18")	1/2-Ton (27")

Notes:

1. *Velocity values are obtained from Proposed Condition (n=0.035) HEC-RAS Model Output included in **Appendix E**.*
2. *Launch Stone Size is determined from the rip-rap sizing chart on page F-32 in the LACFCD Hydraulic Design Manual.*
3. *Actual Launch Stone Size will be a minimum of 1/2-Ton. Additionally, larger rock sizes are provided for smoother transitions between gradations.*

Table 12B – Launch Stone Sizing Calculated (Q_{cap} and n=0.035) for the Northeast Bank

HEC-RAS River Station	Bank Station (ft)	Velocity[1] (fps)	Launch Stone Size[2] (D_{50})	Actual Launch Stone Size[3] (D_{50})
11551.18	21+24.18	14.7	1-Ton (34")	1-Ton (34")
11242.98	19+15.89	10.5	300 lb. (18")	1/2-Ton (27")
10982.86	17+14.86	12.5	1/2-Ton (27")	1/2-Ton (27")
10738.26	15+84.44	12.4	1/2-Ton (27")	1/2-Ton (27")
10490.82	14+34.78	11.8	1/4-Ton (21")	1/2-Ton (27")
10351.28	13+17.01	12.8	1/2-Ton (27")	1/2-Ton (27")
10141.82	11+86.54	9.9	150 lb. (15")	1/2-Ton (27")

Notes:

1. *Velocity values are obtained from Proposed Condition (n=0.035) HEC-RAS Model Output included in **Appendix E**.*
2. *Launch Stone Size is determined from the rip-rap sizing chart on page F-32 in the LACFCD Hydraulic Design Manual.*
3. *Actual Launch Stone Size will be a minimum of 1/2-Ton. Additionally, larger rock sizes are provided for smoother transitions between gradations.*

Table 12C – Launch Stone Sizing Calculated (Q_{cap} and n=0.035) for the Southeast Bank

HEC-RAS River Station	Bank Station (ft)	Velocity[1] (fps)	Launch Stone Size[2] (D_{50})	Actual Launch Stone Size[3] (D_{50})
8049.79	39+23.63	5.6	50 lb. (10")	1/2-Ton (27")
7688.71	34+21.94	4.4	50 lb. (10")	1/2-Ton (27")
7326.46	29+36.40	5.9	50 lb. (10")	1/2-Ton (27")
6966.68	24+67.21	5.8	50 lb. (10")	1/2-Ton (27")
6623.5	20+38.33	8.8	100 lb. (12")	1/2-Ton (27")
6304.52	16+92.74	10.4	300 lb. (18")	1/2-Ton (27")
5992.38	13+72.73	10.2	300 lb. (18")	1/2-Ton (27")
5629.45	10+29.55	10.1	300 lb. (18")	1/2-Ton (27")

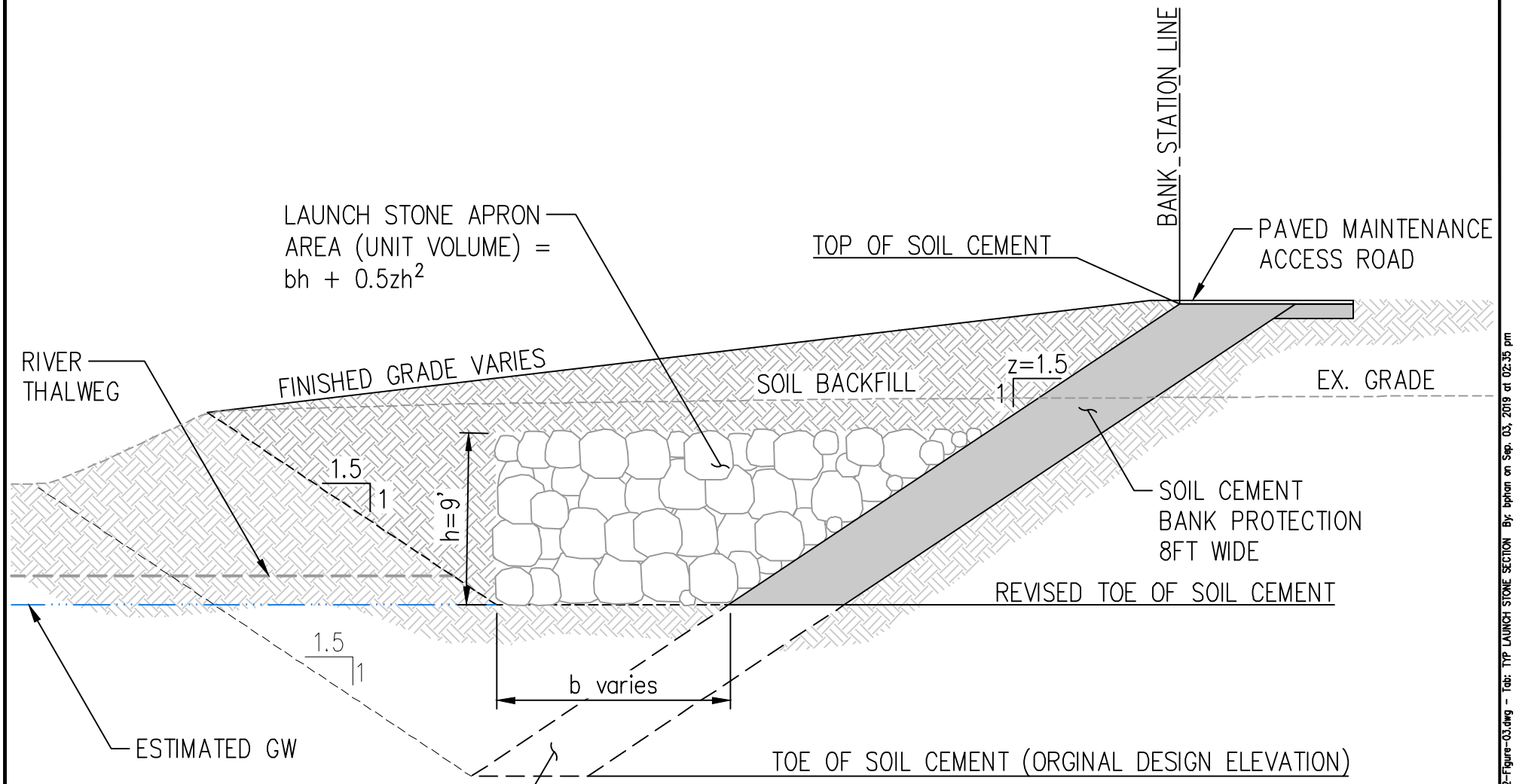
Notes:

1. *Velocity values are obtained from Proposed Condition (n=0.035) HEC-RAS Model Output included in **Appendix E**.*
2. *Launch Stone Size is determined from the rip-rap sizing chart on page F-32 in the LACFCD Hydraulic Design Manual.*
3. *Actual Launch Stone Size will be a minimum of 1/2-Ton. Additionally, larger rock sizes are provided for smoother transitions between gradations.*

Although the rip-rap sizing chart from the LACFCD Hydraulic Design Manual indicates smaller minimum required launch stone sizes, the minimum launch stone size will be 1/2-Ton. Additionally, larger rock sizes are provided to allow for smoother transitions between gradations. The corresponding range of launch stone rip-rap sizing along the bank protection are provided in the **Table 13**.

Table 13 – Launch Stone Sizing along Castaic Creek

Bank	Bank Station (ft)	Launch Stone Size (D_{50})
Northeast Bank	10+00 to 20+00	1/2-Ton (27”)
	20+00 to 23+41.56	1-Ton (34”)
Southeast Bank	9+98.72 to 40+20.00	1/2-Ton (27”)
West Bank	10+00 to 50+17	1/2-Ton (27”)



NOTE:
h = LAUNCH STONE HEIGHT
b = LAUNCH STONE BOTTOM WIDTH
(WIDTH VARIES BASED ON UNIT VOLUME CALCULATION)

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FIGURE 03

SOIL CEMENT / LAUNCH STONE BANK PROTECTION TYPICAL SECTION

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8 Existing and Proposed Floodplain Mapping

8.1 FEMA Floodplain Mapping

The 100-year floodplain for Castaic Creek was mapped by FEMA as shown on Flood Insurance Rate Map (FIRM) No's. 06037C0805F and 06037C0815F, dated September 26, 2008. The proposed project is located in flood hazard area Zone A and is within an unincorporated area of Los Angeles County. Additionally, a Conditional Letter of Map Revision (CLOMR), prepared by PACE, was submitted to FEMA and was approved on July 25, 2006 (Case No. 06-09-B577R). Upon completion of the bank protection construction, a Letter of Map Revision (LOMR) will be prepared and submitted to FEMA proposing that the 100-year floodplain be revised on the FIRM.

8.2 LACDPW Floodplain Mapping

LACDPW has mapped the floodway and floodplain along Castaic Creek per Floodway Maps 335-ML-1 and 335-ML-2. These maps were adopted by the Board of Supervisors in August 1984 by Ordinance No. 84-0136. Due to the changes that will occur to the currently mapped floodplain maps shown on the ML Maps as a result of the VCC project, PACE will be preparing a ML Map Revision Report, titled "TPM No. 18108 DCR Volume V of V*".* This report will propose that LACDPW revise Floodway Map 335-ML-2 after the bank protection construction has been completed. The current LACDPW floodplain and floodway lines, as well as the updated existing condition floodplain mapping for Castaic Creek applying Q_{CAP} flow rates and a Manning's roughness value of 0.060, is shown on **Figure 3**. The proposed condition floodplain mapping for Castaic Creek is shown on **Figure 5**.

9 Planning and Environmental Issues

The major environmental and planning issues impacting the proposed bank protection are the presence of aquatic resources subject to state and federal regulation. At a few locations, the proposed limit of construction disturbance encroaches upon areas delineated as waters of the United States and/or waters of the State. No discharge of dredged or fill material, or activities that may substantially divert, change, or obstruct the natural flow of the stream channel, may occur within these jurisdictional limits without prior approval from the United States Army Corps of Engineers (ACOE), California Department of Fish and Wildlife (CDFW) and/or the Regional Water Quality Control Board (RWQCB). Required approvals may include a Clean Water Act (CWA) Section 404 Permit from the ACOE, CWA Section 401 water quality certification and/or waste discharge requirements from the RWQCB, and a Fish and Game Code Section 1602 streambed alteration agreement from the CDFW. In addition, any potential impacts to species listed under the California Endangered Species Act (CESA) or Federal ESA, or their designated critical habitat, associated with these activities may require additional agency review by CDFW and/or the U.S. Fish and Wildlife Service to ensure compliance with the CESA and the ESA. The project proponent will submit applications to the relevant regulatory agencies to obtain the necessary approvals prior to commencing work within jurisdictional areas.

Jurisdictional areas impacted by project construction will be revegetated with native plant species. The excavation required to construct the bank protection will be backfilled to facilitate revegetation. For the majority of the reach, the upper portion of the bank protection above the Creek invert will be backfilled at a slope of 3:1 (H:V) or flatter. At the location of the tie-in with the existing concrete slope protection, the top portion of the bank protection above the creek invert will be left exposed. However, the banks will blend in with native material, since 90% of soil cement bank is composed of native soils.

Following the completion of project construction, LADPW will require periodic access into the channel along the lateral edges for flood control maintenance purposes. A General Maintenance Plan (GMP) for Castaic Creek and Hasley Canyon, dated March 2002 and approved by LACDPW in April 2002, was reviewed and adopted by ACOE and CDFW as part of other development activities on Castaic Creek and the Hasley Creek within the Valencia Commerce Center development. The GMP defines the allowable routine channel maintenance and repair activities within the creek. The GMP is included in Appendix G.

10 Conclusion

The purpose of this report is to present the results of the Project Specific hydraulic analysis in order to provide design guidelines for the proposed bank protection measures and to evaluate potential floodplain impacts caused by the project. The Project Specific analysis evaluates impacts from (1) fluvial modifications of the riverbed from the Capital Flood event and (2) changes in the floodplain fluvial mechanics over the long-term. General and long-term adjustment values were obtained from the Approved Fluvial Study. The approved fluvial adjustment values are used in the hydraulic analyses and determined the final top and toe design of the bank protection.

PACE has already prepared a CLOMR application which was approved by FEMA on July 25, 2006 (Case No. 06-09-B577R). Upon completion of the project, a Letter of Map Revision (LOMR) application will also be prepared and submitted to FEMA, requesting that the FIRM be revised to reflect the proposed channel improvements. In addition, PACE proposes that Newhall Land revise the Los Angeles County Capital Floodway Map No. 335-ML-2 upon completion of the VCC TPM 18108 Development Project.

11 Bibliography

"Valencia Commerce Center (I-5 to Hwy. 126) Castaic Creek HEC-RAS Modeling", Pacific Advanced Civil Engineering, Inc., October 2005.

"Castaic Creek Fluvial Study," Pacific Advanced Civil Engineering, Inc., November 22, 2005.

"Fluvial Study Proposed Channelization of Castaic Creek between Henry Mayo Drive and the Golden State Freeway," Simons-Li & Associates, January 1992.

"Report of Geotechnical Plan Review Proposed Soil Cement Bank Protection P.D. 2563 of Parcel Map No. 26363, dated October 24, 2005 Castaic Creek, California," R.T. Frankian & Associates.

"General Maintenance Plan Castaic Creek and Hasley Canyon Drain," Aquatic Consulting Services, Inc., March 2002.

Los Angeles Department of Public Works – Hydrology/Sedimentation Manual, Hydraulic/Water Conservation Division, December 1991/June 1993.

Los Angeles Department of Public Works – Hydraulic Design Manual, March 1982.

Los Angeles Department of Public Works – Project Preparation Instruction Manual for Drainage Facilities, February 1988.

Kenneth D. Hansen and John B. Lynch, "Controlling Floods in the Desert with Soil-Cement," authorized reprint from: *Second CANMET/ACI International Symposium on Advances in Concrete Technology,* Las Vegas, NV. June 11-14, 1995.

"Soil Cement Slope Protection for Embankments: Construction" (S167.03W), Portland Cement Association, 1988.

Soil Cement Construction Handbook (EB003.10S), Portland Cement Association, 1995.

"Soil-Cement Slope Protection for Embankments: Planning and Design" (IS173.03W), Portland Cement Association, 1991.

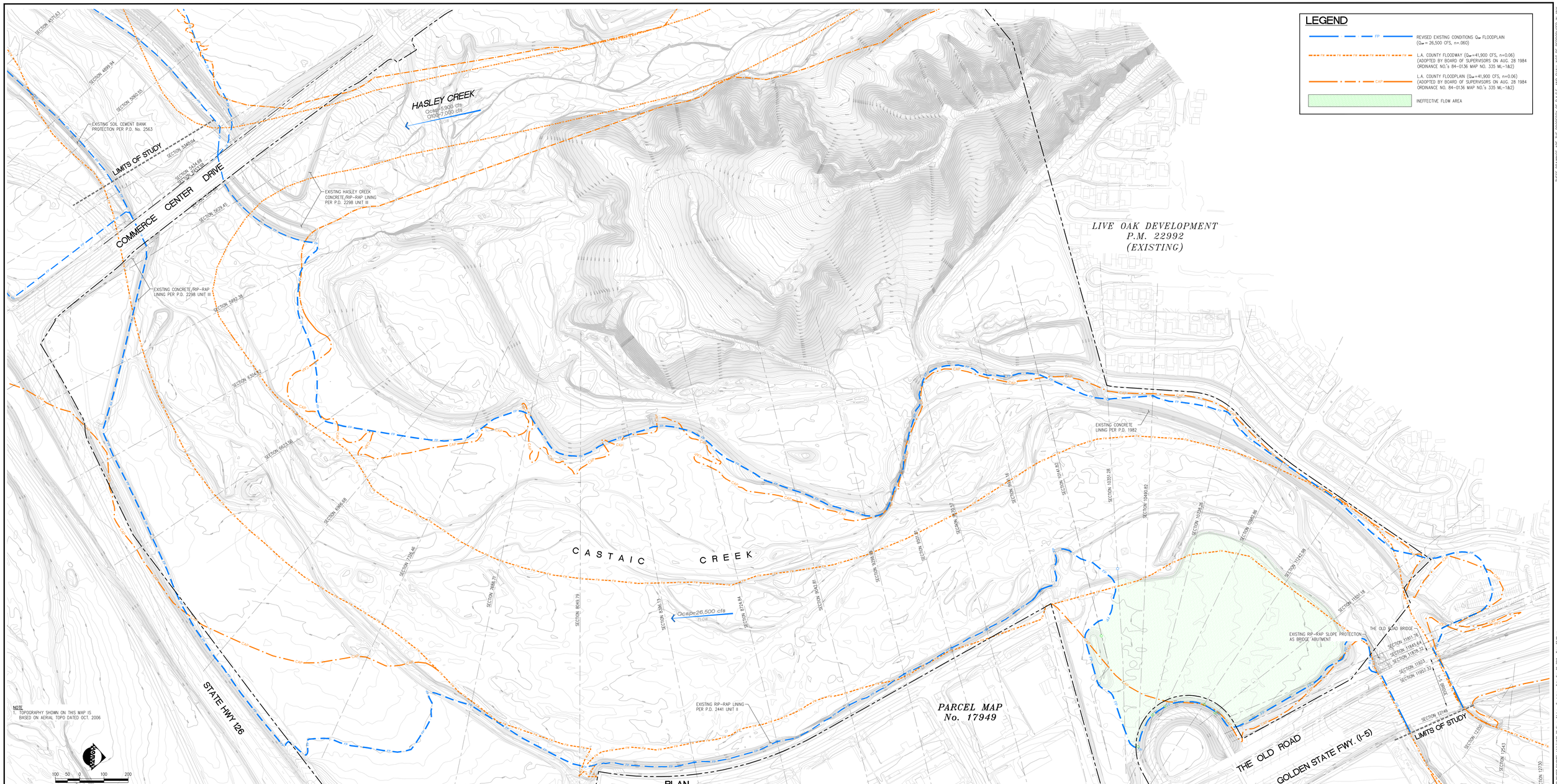
Soil-Cement Inspectors Manual (PA050.02S), Portland Cement Association, 1980. Reprinted in 1984.

Francis A. Omoregie, Richard A. Gutschow and Mark L. Russel, "Cement-Hardened Materials for Abrasion-Erosion in Hydraulic Structures," Journal of Concrete International, page 47-50, July 1994.

"Suggested Specifications for Soil-Cement Slope Protection for Embankments" (Central-Plant-Mixing Method), IS052.03W, Portland Cement Association, 1976.

"Soil-Cement for Water Control: Laboratory Tests" (IS166.02W), Portland Cement Association, 1976.

"State-of-the-Art Report on Soil Cement" (230.1R-90), American Concrete Institute.



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DRAINAGE CONCEPT REPORT
VOLUME II OF V
CASTAIC CREEK SOIL CEMENT BANK PROTECTION
FOR T.P.M. NO. 18108

LOS ANGELES COUNTY — GA.

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DESIGNED	JC
CHECKED	JC
DATE	Sep. 2019

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TITLE

TITLE: EXISTING CONDITION FLOODPLAIN MAPPING

FIGURE: 04

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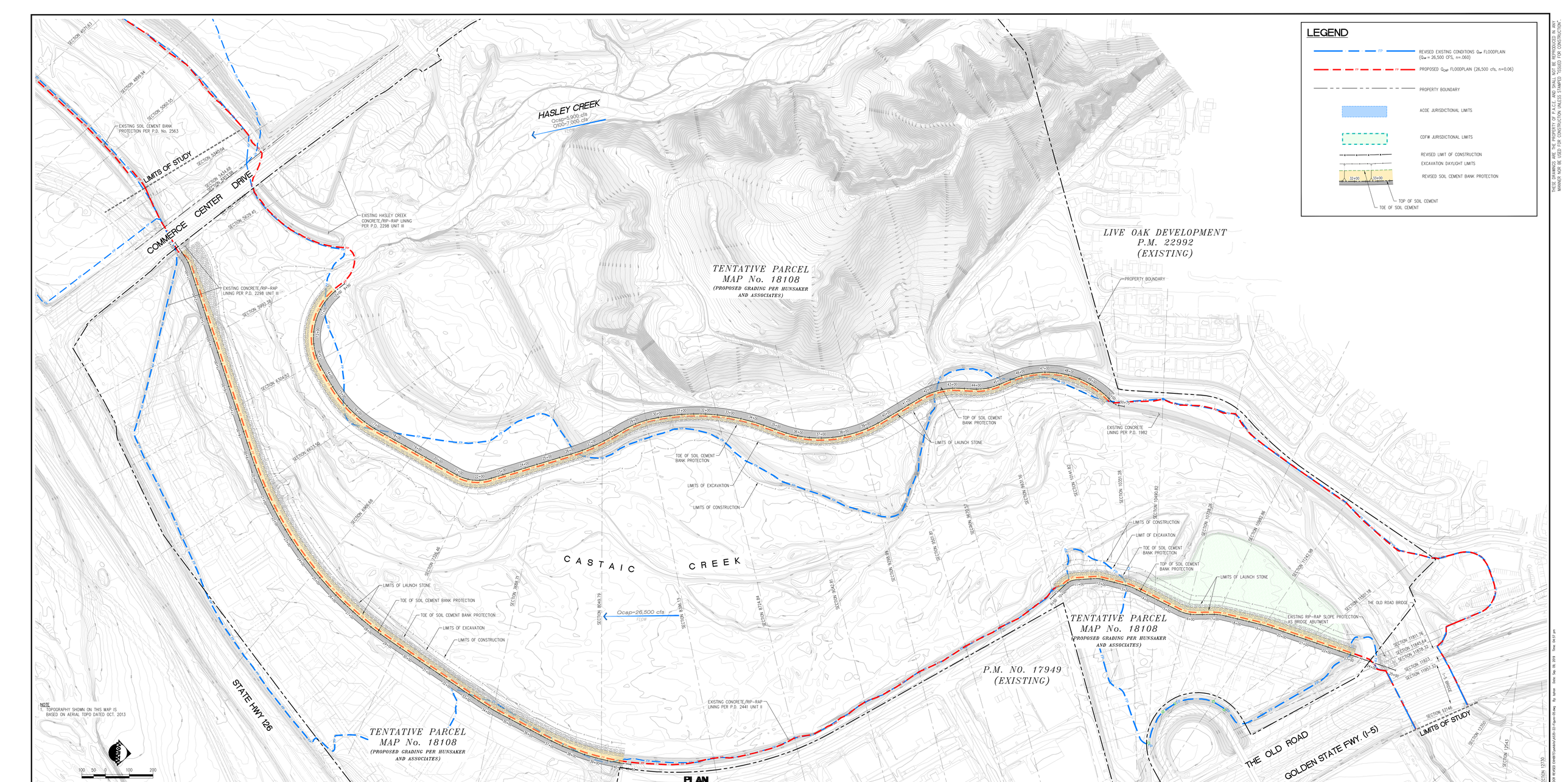

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DRAINAGE CONCEPT REPORT
VOLUME II OF V
CASTAIC CREEK SOIL CEMENT BANK PROTECTION
FOR T.P.M. NO. 18108

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TITLE

PROPOSED CONDITION FLOODPLAIN MAPPING

FIGURE 05

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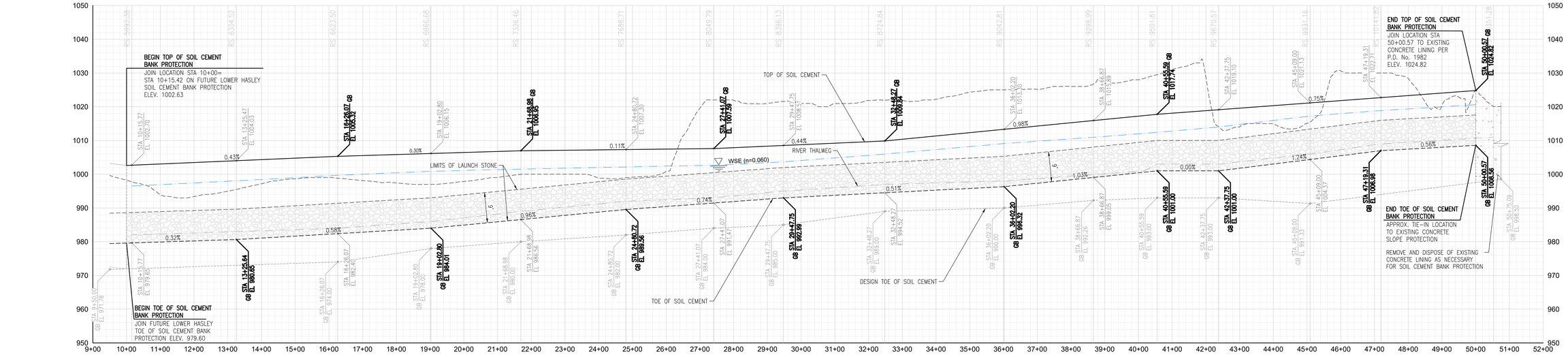
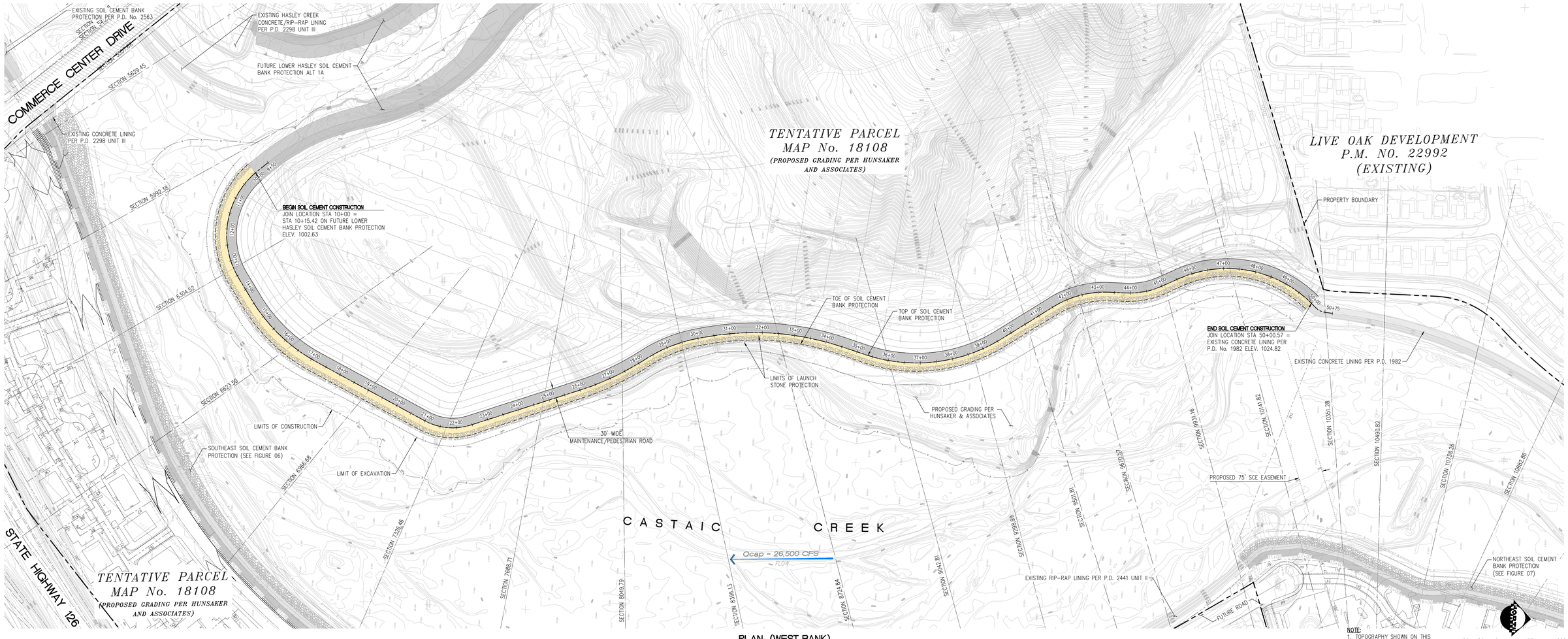


PROFILE (WEST BANK)



PLAN (WEST BANK)

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DRAINAGE CONCEPT REPORT
VOLUME II OF V
CASTAIC CREEK SOIL CEMENT BANK PROTECTION
FOR T.P.M. NO. 18108
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DESIGNED	JC
CHECKED	JC
DATE	Sep, 2019

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TITLE

WEST BANK SOIL CEMENT BANK PROTECTION PLAN & PROFILE

FIGURE 06

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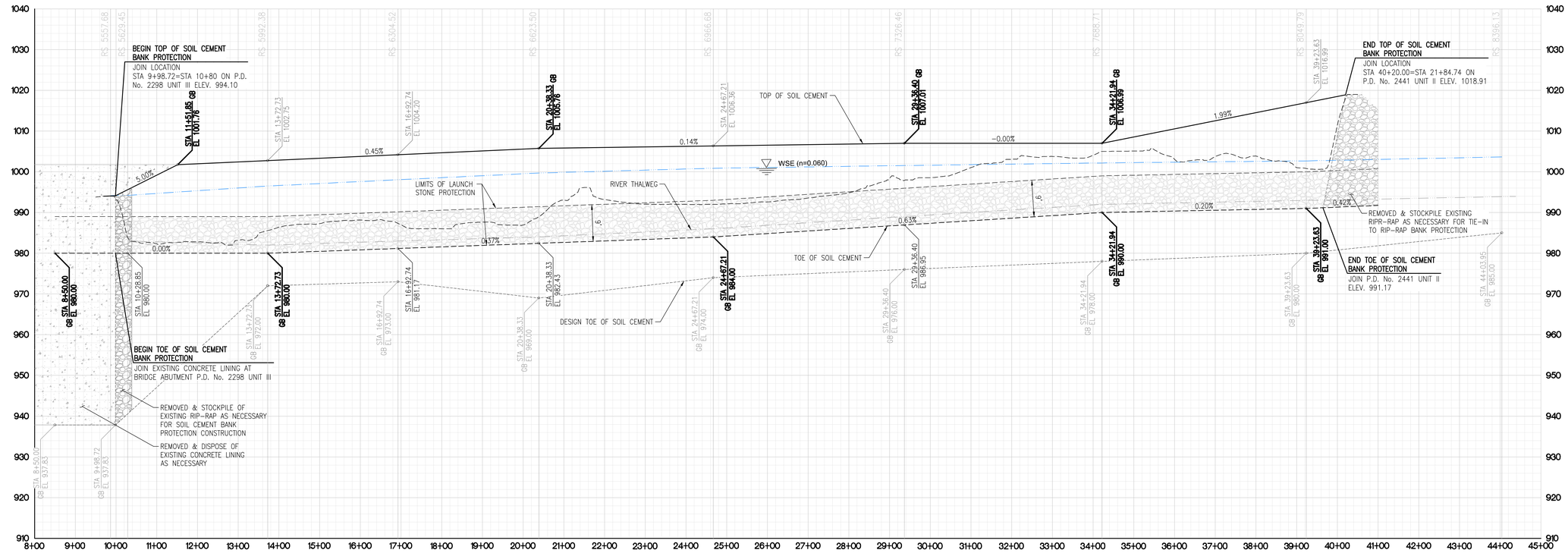
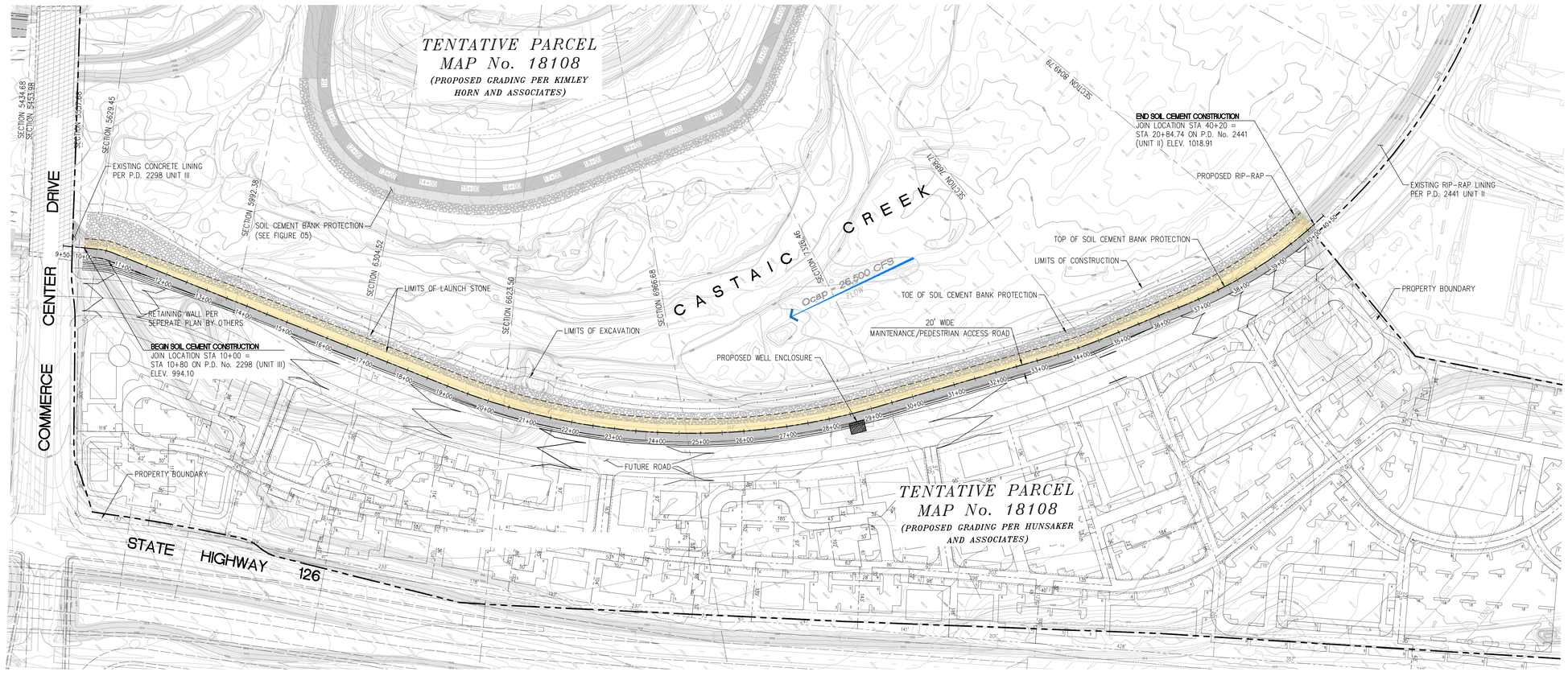


PROFILE (SOUTHEAST BANK)



PLAN (SOUTHEAST BANK)

NOTE
1. TOPOGRAPHY SHOWN ON THIS MAP IS BASED ON AERIAL TOPO DATED OCT. 2013

100 50 0 100 200

THESE DRAWINGS ARE THE PROPERTY OF P.A.C.E. AND SHALL NOT BE REPRODUCED IN ANY MANNER NOR BE USED FOR CONSTRUCTION UNLESS STAMPED "ISSUED FOR CONSTRUCTION".

NO	BY	DATE		DATE	APP.
1					
2					
3					
4					
5					

JOB
DRAINAGE CONCEPT REPORT VOLUME II OF V
CASTAIC CREEK SOIL CEMENT BANK PROTECTION
FOR T.P.M. NO. 18108
LOS ANGELES COUNTY CA.

PACE
Advanced Water Engineering
17520 Newhope Street, Suite 200 | Fountain Valley, CA 92708
P: (714) 481-7300 | www.pacewater.com

PREPARED BY	JOSE CRUZ
	PROJECT ENGINEER
R.C.E. NO.	72249
EXP.	06/30/20
DRAWN	BDP
SCALE	
DESIGNED	JC
CHECKED	JC
DATE	Sep, 2019

REGISTERED PROFESSIONAL ENGINEER JOSE CRUZ No. 72249 Exp. 06-30-20 CIVIL STATE OF CALIFORNIA

TITLE
SOUTHEAST BANK SOIL CEMENT BANK PROTECTION PLAN & PROFILE

FIGURE 07

JOB NO. A535

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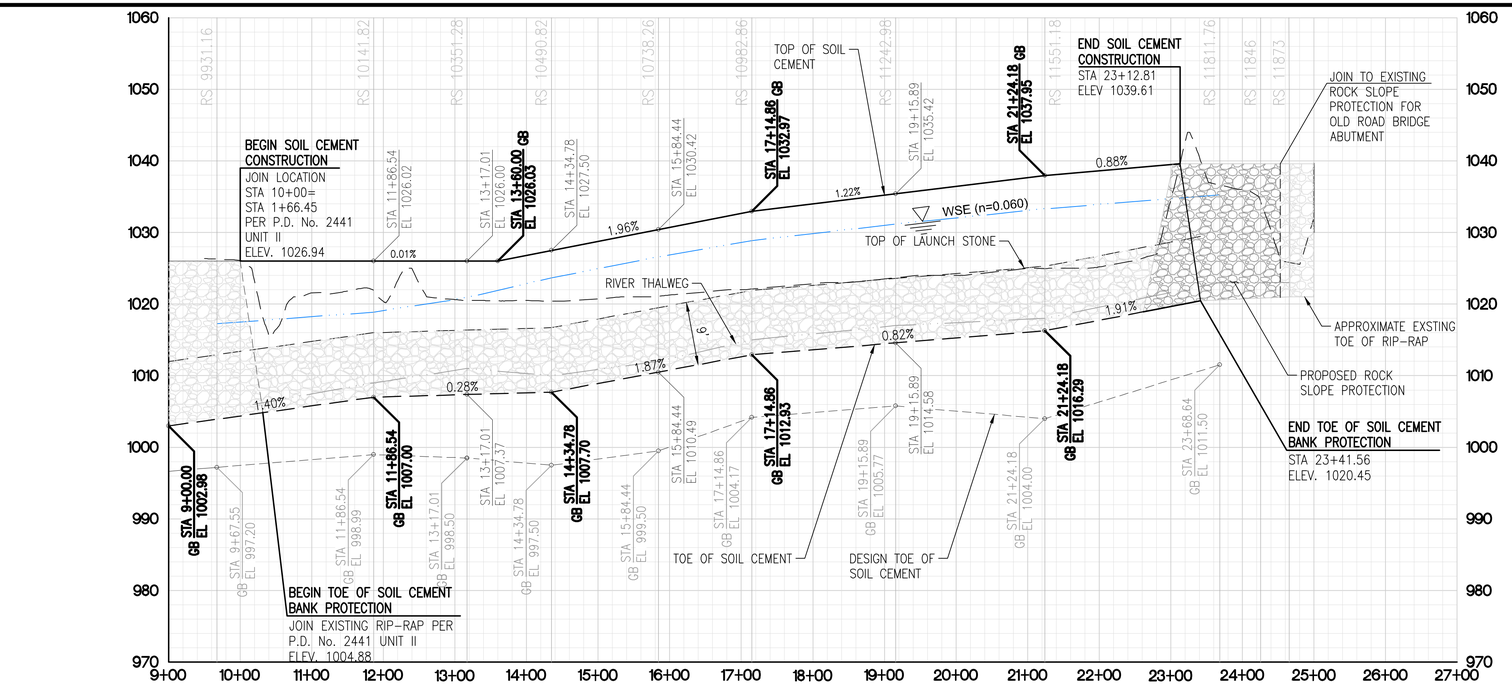
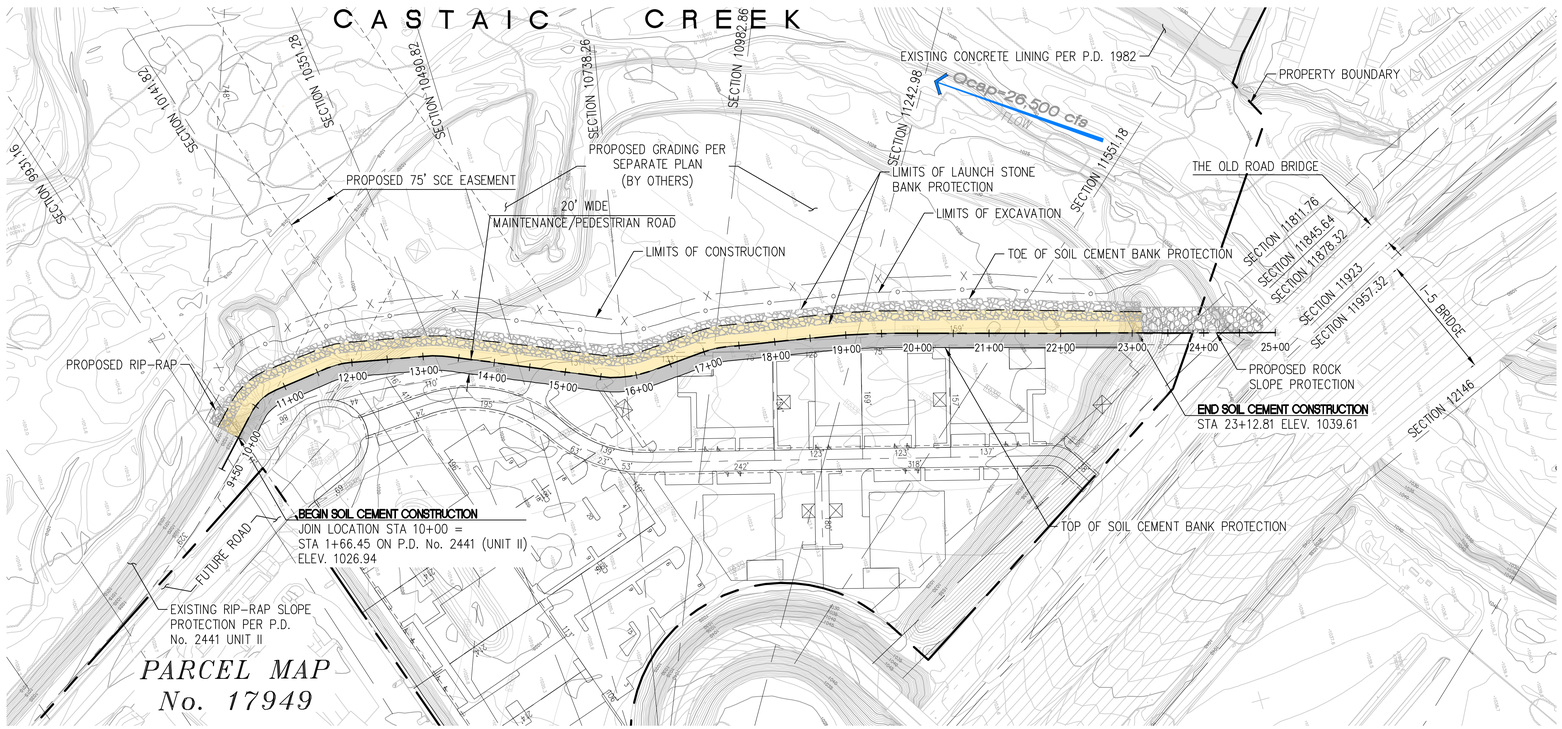


PROFILE (NORTHEAST BANK)



PLAN (NORTHEAST BANK)

NOTE:
1. TOPOGRAPHY SHOWN ON THIS MAP IS BASED ON AERIAL TOPO DATED OCT. 2013

NORTH

100 50 0 100 200

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NO	BY	DATE		DATE	APP.
1					
2					
3					
4					
5					

JOB

DRAINAGE CONCEPT REPORT
VOLUME II OF V
CASTAIC CREEK SOIL CEMENT BANK PROTECTION
FOR T.P.M. NO. 18108

LOS ANGELES CO CA

PACE
Advanced Water Engineering
17520 Newhope Street, Suite 200 | Fountain Valley, CA 92708
P: (714) 481-7300 | www.pacewater.com

PREPARED BY	JEP
PROJECT ENGINEER R.C.E. NO.	-- 72249
EXP.	06/30/20
DRAWN	BDP
SCALE	AS SHOWN
DESIGNED	JC
CHECKED	JC
DATE	Sep, 2019

REGISTERED PROFESSIONAL ENGINEER JOSE CRUZ No. 72249 Exp. 06-30-20 CIVIL STATE OF CALIFORNIA

TITLE

NORTHEAST BANK SOIL CEMENT BANK PROTECTION PLAN & PROFILE

SHEET 08

JOB NO. A535

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Appendix A – Existing Condition HEC-RAS Model Results (n=0.060)

HEC-RAS Plan: Existing .06 WS River: Castaic Reach: 1 Profile: Qcap

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)	
1	13371	Qcap	26500.00	1032.00	1043.14	1040.16	1043.50	0.003404	4.98	5651.27	1157.84	0.35
1	13141	Qcap	26500.00	1031.00	1042.63	1038.45	1042.89	0.001937	4.14	6692.75	1142.67	0.27
1	12937	Qcap	26500.00	1029.00	1042.33	1036.97	1042.55	0.001327	3.86	7471.13	1154.99	0.23
1	12730	Qcap	26500.00	1028.00	1042.01	1035.81	1042.27	0.001293	4.16	6761.88	828.29	0.23
1	12543	Qcap	26500.00	1026.00	1041.66	1034.90	1042.00	0.001550	4.64	5716.83	972.84	0.25
1	12350	Qcap	26500.00	1026.00	1040.74	1034.86	1041.53	0.003261	7.19	3962.79	1125.92	0.37
1	12146	Qcap	26500.00	1026.00	1039.37	1034.52	1040.64	0.005320	9.03	2933.93	807.73	0.47
1	12059.65		Bridge									
1	11934	Qcap	26500.00	1024.00	1037.09	1033.58	1038.76	0.008007	10.35	2560.13	237.96	0.56
1	11923	Qcap	26500.00	1024.00	1036.95	1033.59	1038.66	0.008351	10.49	2526.94	237.88	0.57
1	11917.82		Bridge									
1	11873	Qcap	26500.00	1023.00	1036.03	1032.64	1037.58	0.007916	9.98	2654.60	272.64	0.56
1	11846	Qcap	26500.00	1023.00	1035.94	1032.49	1037.31	0.008186	9.38	2823.68	325.16	0.56
1	11811.76	Qcap	26500.00	1023.00	1035.26	1032.65	1036.97	0.010151	10.47	2534.23	356.44	0.62
1	11551.18	Qcap	26500.00	1018.00	1033.31	1030.29	1034.43	0.008278	8.51	3114.91	1046.56	0.55
1	11242.98	Qcap	26500.00	1017.00	1031.18	1027.80	1032.20	0.006205	8.11	3266.16	1006.62	0.49
1	10982.86	Qcap	26500.00	1015.00	1028.86	1026.42	1030.20	0.009278	9.30	2850.31	1121.03	0.59
1	10738.26	Qcap	26500.00	1012.00	1026.57	1024.67	1027.71	0.011324	8.56	3096.26	1292.89	0.62
1	10490.82	Qcap	26500.00	1010.00	1023.79	1021.97	1024.67	0.011913	7.54	3514.39	1436.67	0.61
1	10351.28	Qcap	26500.00	1011.00	1021.33	1019.85	1022.48	0.015830	8.63	3071.43	1283.89	0.71
1	10141.82	Qcap	26500.00	1009.00	1019.04	1017.34	1019.79	0.010161	6.94	3816.56	1045.37	0.57
1	9931.16	Qcap	26500.00	1006.00	1017.13		1017.76	0.008720	6.35	4175.01	914.73	0.52
1	9670.57	Qcap	26500.00	1003.00	1015.10		1015.61	0.007417	5.73	4622.83	1042.85	0.48
1	9501.81	Qcap	26500.00	1003.00	1013.83		1014.43	0.006355	6.23	4255.49	754.68	0.46
1	9298.99	Qcap	26500.00	1001.00	1011.81		1012.72	0.010864	7.65	3463.67	674.29	0.60
1	9042.81	Qcap	26500.00	998.00	1009.43		1010.08	0.009231	6.47	4095.59	909.60	0.54
1	8724.84	Qcap	26500.00	997.00	1006.14		1006.71	0.011848	6.07	4367.99	1289.25	0.58
1	8396.13	Qcap	26500.00	995.00	1003.46		1003.84	0.006345	4.90	5405.08	1377.07	0.44
1	8049.79	Qcap	26500.00	993.00	1002.07		1002.31	0.002759	3.93	6739.33	1280.65	0.30
1	7688.71	Qcap	26500.00	992.00	1001.23		1001.43	0.001860	3.56	7434.61	1214.75	0.25
1	7326.46	Qcap	26500.00	989.00	1000.09	995.67	1000.41	0.003873	4.75	6003.23	1340.86	0.36
1	6966.68	Qcap	26500.00	986.00	998.98	994.66	999.22	0.002486	3.92	6761.96	1193.72	0.29
1	6623.5	Qcap	26500.00	984.00	997.66	993.58	998.09	0.004039	5.24	5052.85	822.49	0.37
1	6304.52	Qcap	26500.00	983.00	996.54	991.13	996.89	0.003353	4.76	5568.73	916.27	0.34
1	5992.38	Qcap	26500.00	982.00	995.72	989.70	996.04	0.002133	4.51	5967.40	877.03	0.28
1	5629.45	Qcap	26500.00	982.00	994.42	989.66	995.07	0.003498	6.46	4099.03	523.32	0.37
1	5557.68	Qcap	26500.00	982.00	994.15	989.28	994.86	0.003603	6.78	3909.78	505.46	0.38
1	5500		Bridge									
1	5453.98	Qcap	26500.00	980.01	993.78		994.26	0.002637	5.57	4760.93	514.37	0.32
1	5434.68	Qcap	26500.00	980.00	993.75		994.20	0.002334	5.37	4930.63	513.00	0.31
1	5340.04	Qcap	31400.00	979.00	992.52		993.69	0.008231	8.67	3621.76	473.85	0.55
1	5060.55	Qcap	31400.00	977.00	990.36		991.50	0.008113	8.57	3664.71	481.55	0.55
1	4899.94	Qcap	31400.00	975.00	989.40		990.37	0.005974	7.91	3967.35	466.52	0.48
1	4571.63	Qcap	31400.00	973.00	987.95		988.81	0.004304	7.44	4219.29	425.08	0.42
1	4220.87	Qcap	31400.00	971.00	985.47		986.75	0.008052	9.08	3458.47	413.41	0.55
1	3865.39	Qcap	31400.00	970.00	982.54		983.82	0.008516	9.06	3464.55	433.91	0.57
1	3579.81	Qcap	31400.00	967.00	980.35		981.51	0.007489	8.63	3638.83	444.02	0.53
1	3289.64	Qcap	31400.00	965.00	978.28		979.38	0.007109	8.42	3730.90	455.00	0.52
1	2974.88	Qcap	31400.00	963.00	973.65	973.14	975.60	0.022964	11.18	2807.58	540.79	0.87
1	2627.11	Qcap	31400.00	961.00	972.57		972.94	0.002888	4.89	6425.14	918.08	0.32
1	2307.93	Qcap	31400.00	959.00	970.93		971.59	0.006238	6.55	4796.13	778.97	0.47
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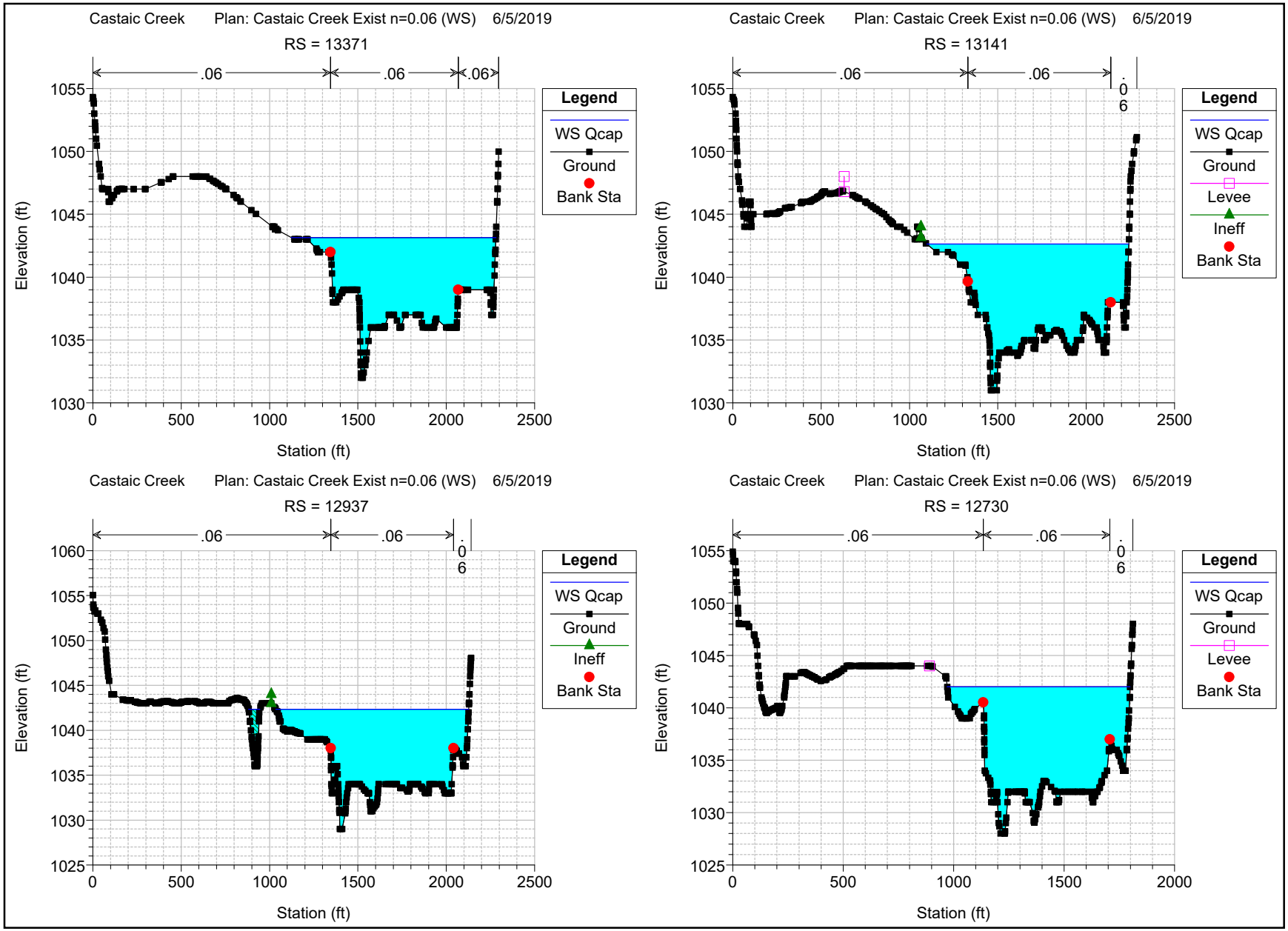
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Station (ft)
Legend
WS Qcap
Ground
Bank Sta
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RS = 13141
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.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Levee
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 12937
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Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
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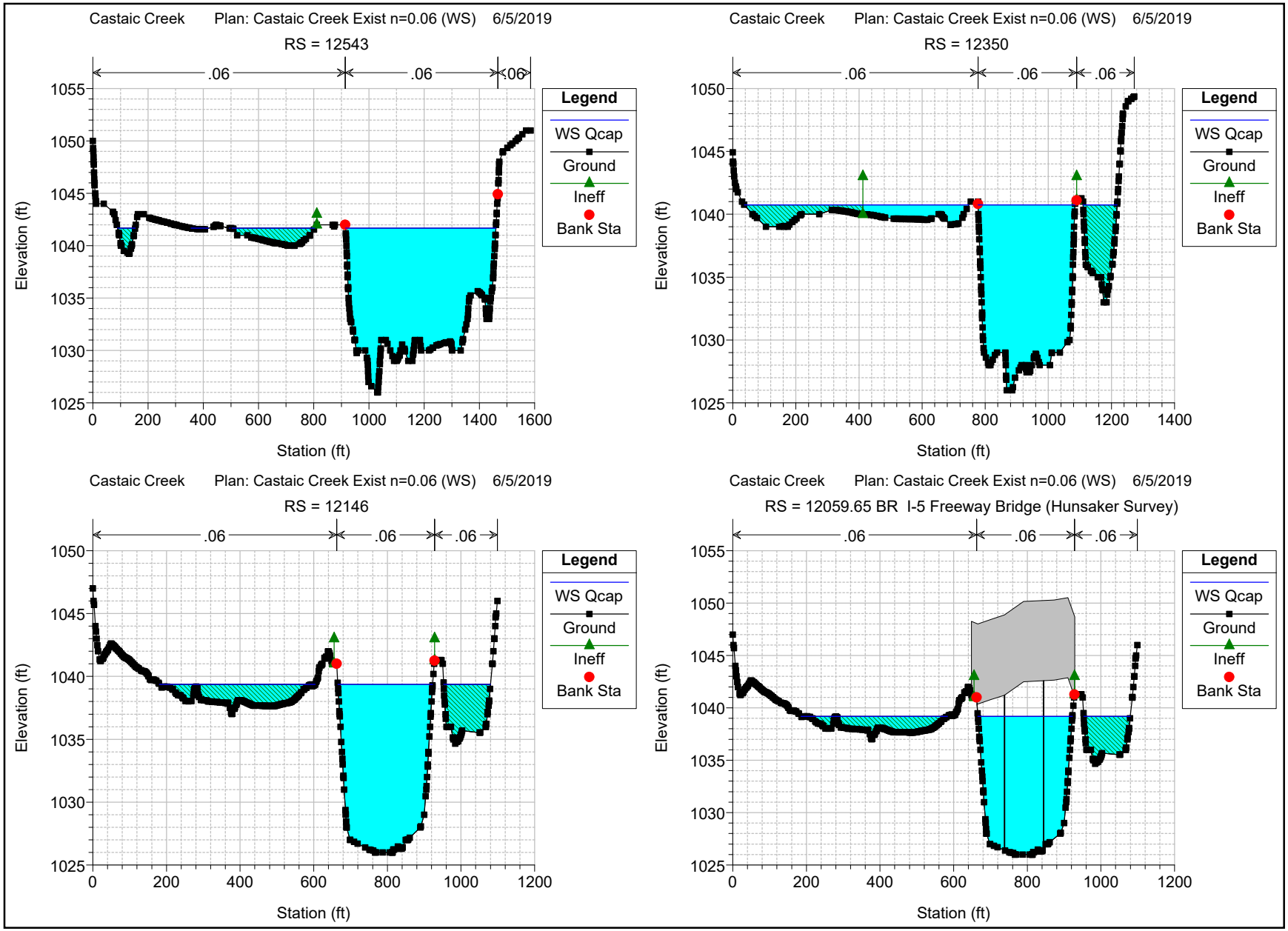
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Levee
Bank Sta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Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
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.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 12146
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.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
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RS = 12059.65 BR I-5 Freeway Bridge (Hunsaker Survey)
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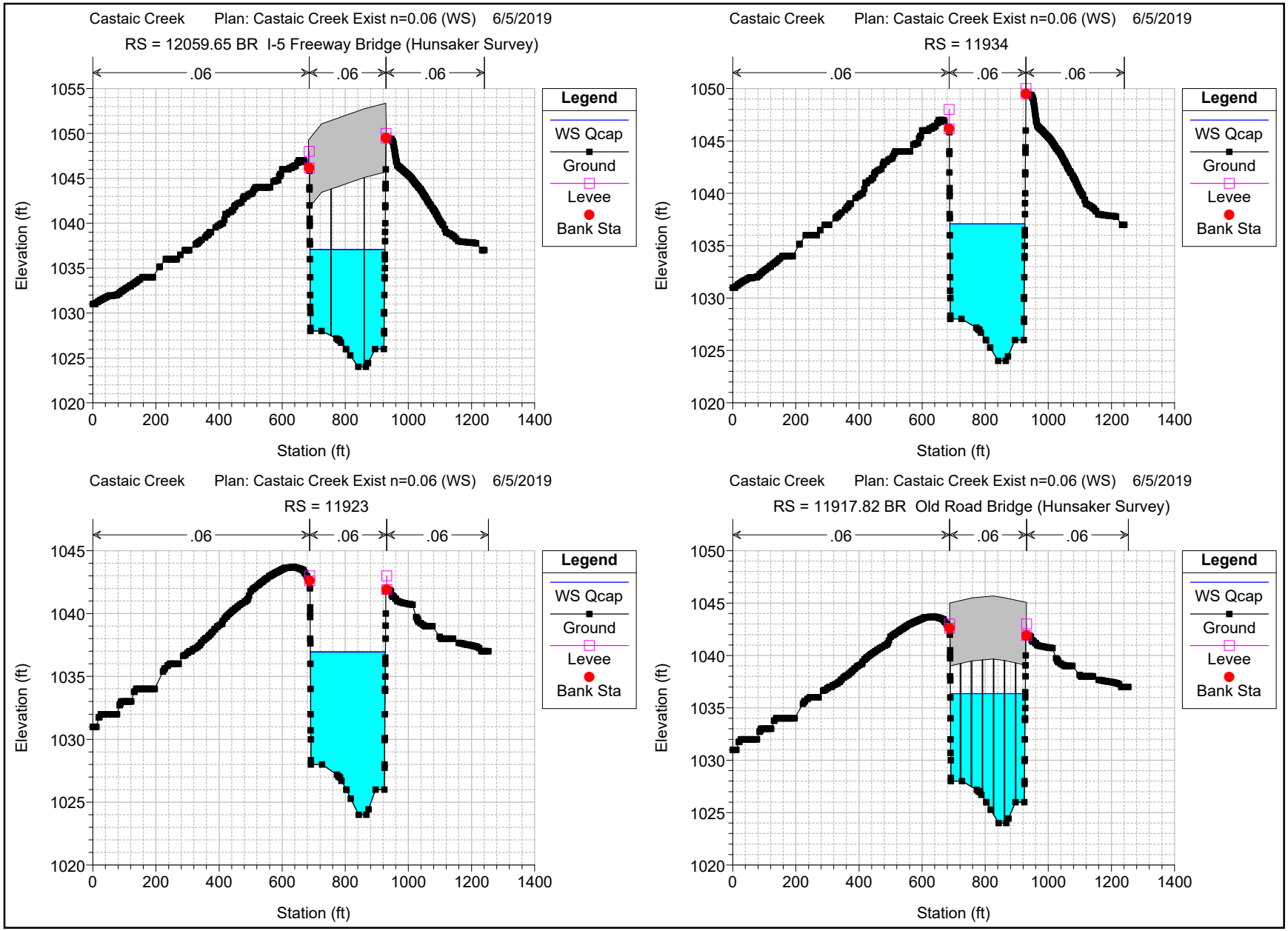
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta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Legend
WS Qcap
Ground
Levee
Bank Sta
Elevation (ft)
Station (ft)
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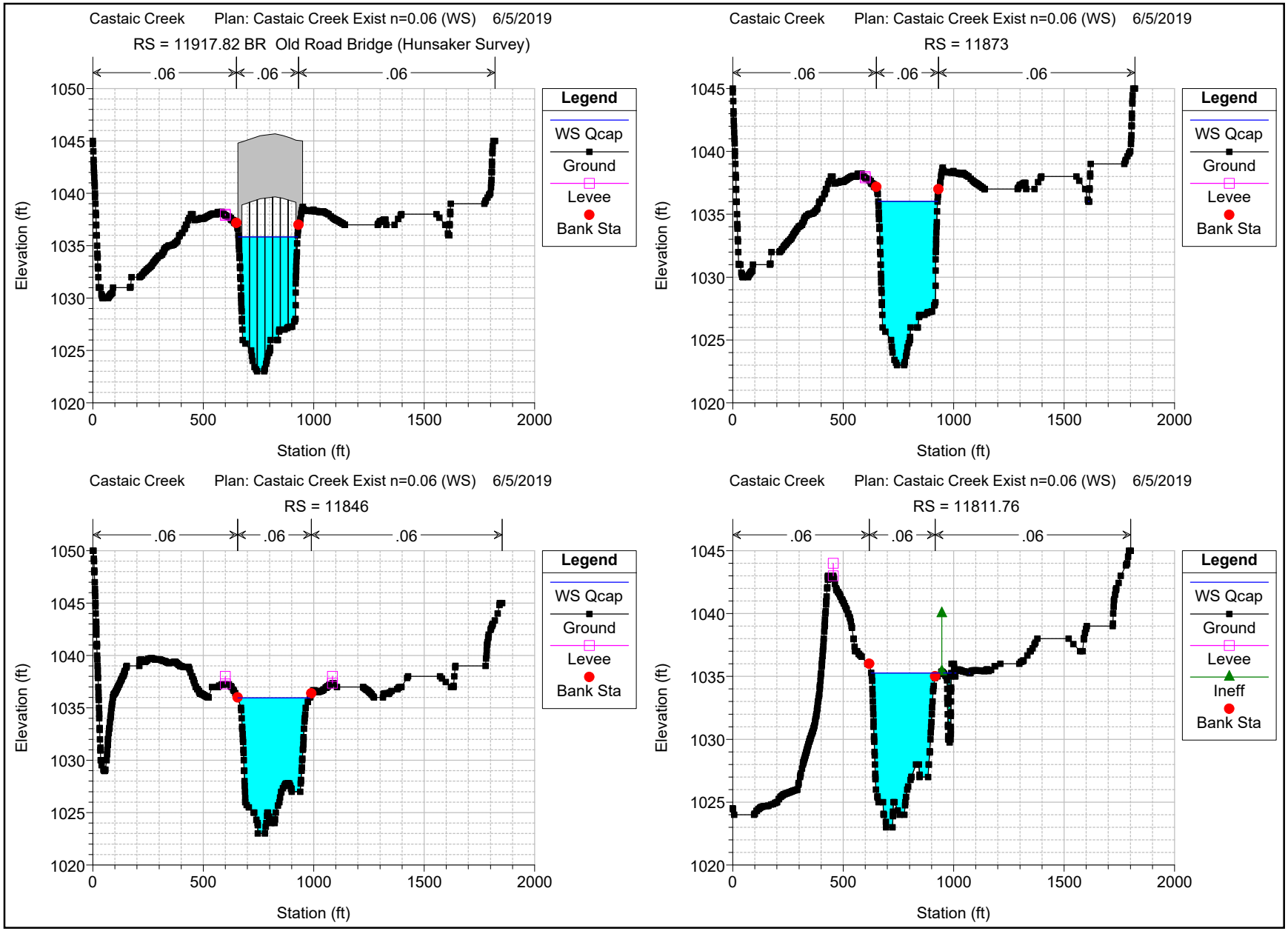
RS = 11934
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 11923
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 11917.82 BR Old Road Bridge (Hunsaker Survey)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Station (ft)
Legend
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Ground
Levee
Bank Sta
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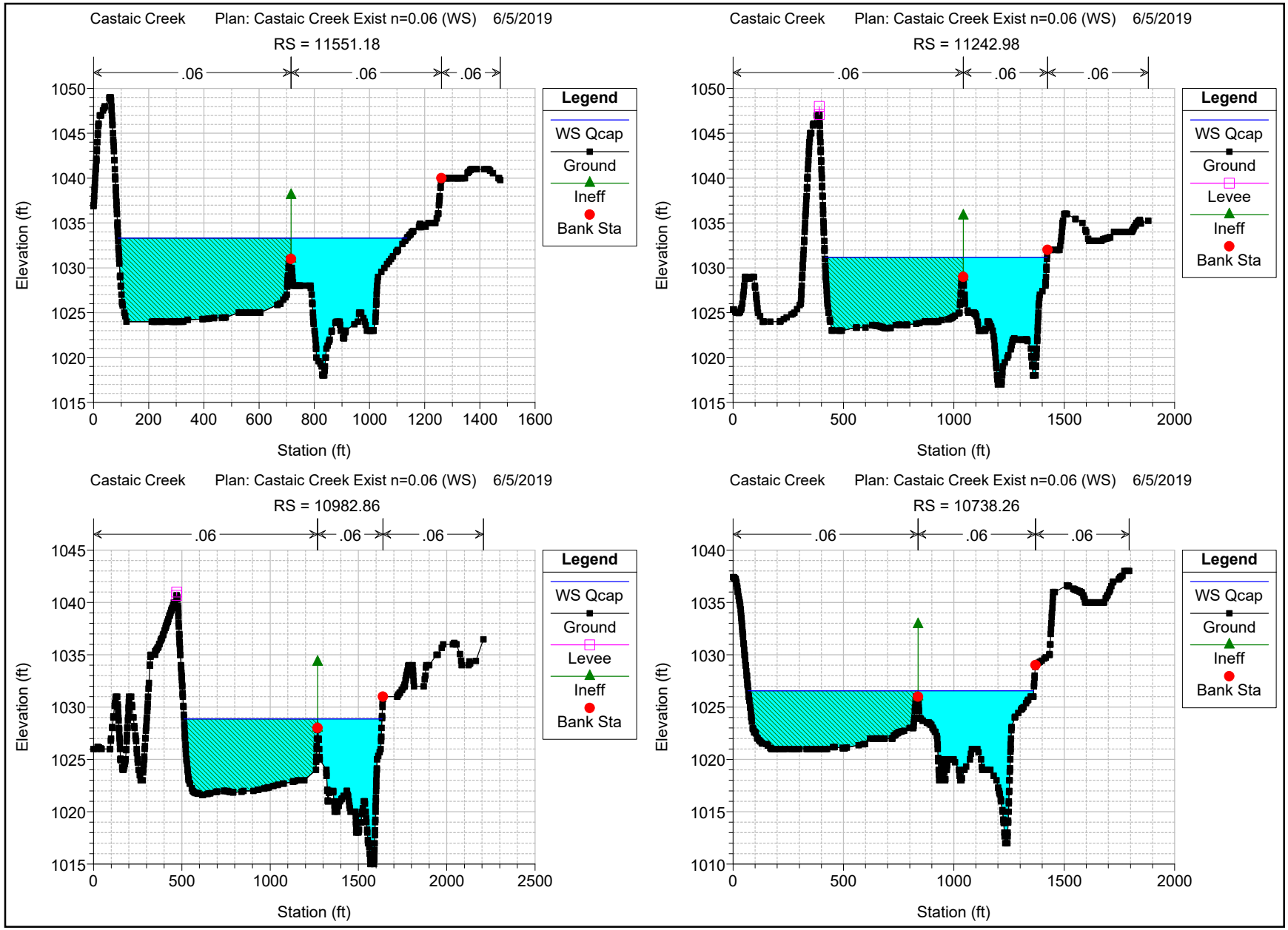
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Ineff
Bank Sta
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Station (ft)
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Ground
Levee
Ineff
Bank Sta
Elevation (ft)
Station (ft)
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Ineff
Bank Sta
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Station (ft)
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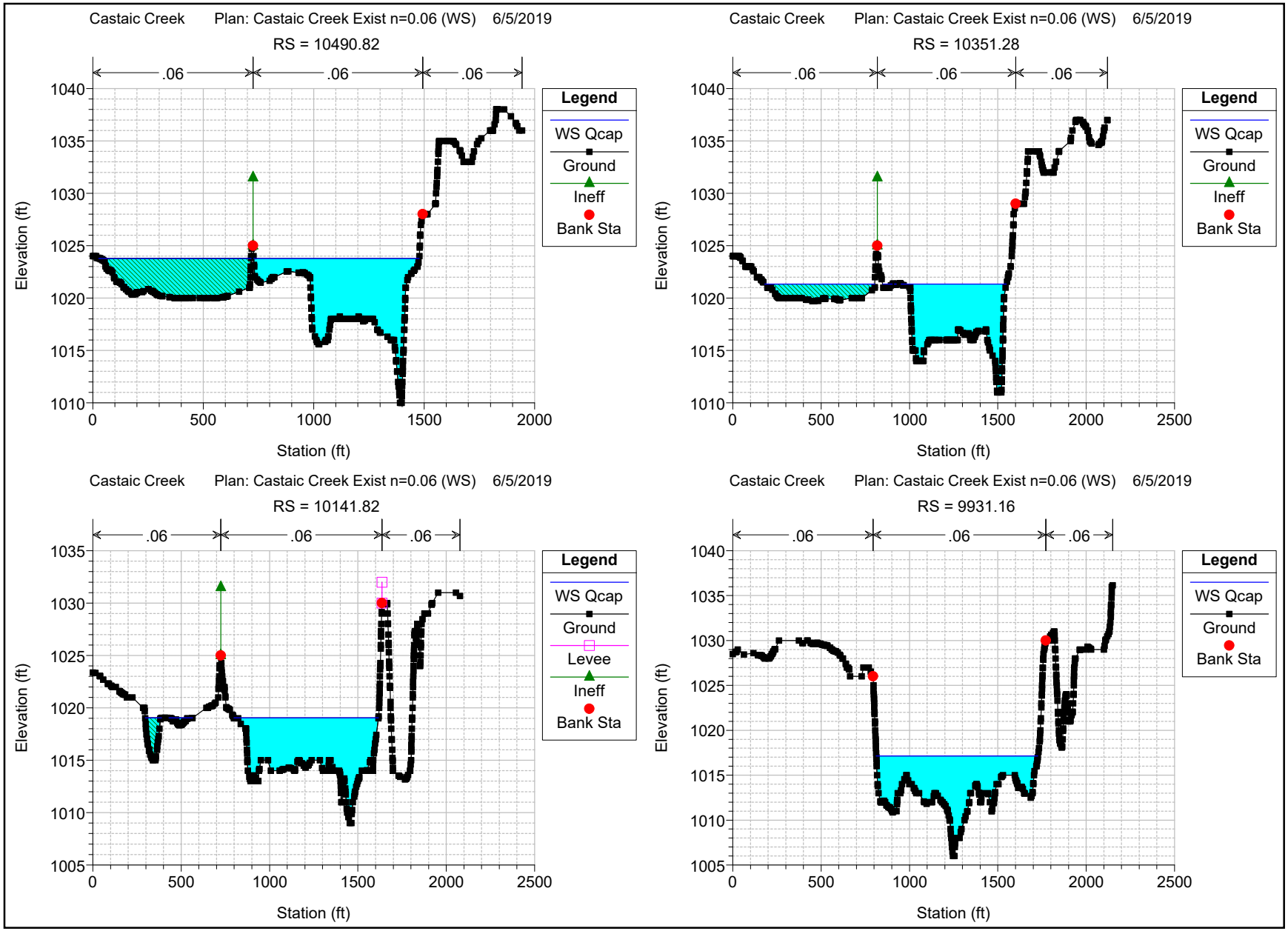
Station (ft)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Legend
WS Qcap
Ground
Ineff
Bank Sta
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Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
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RS = 10141.82
.06
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Levee
Ineff
Bank Sta
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Elevation (ft)
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Legend
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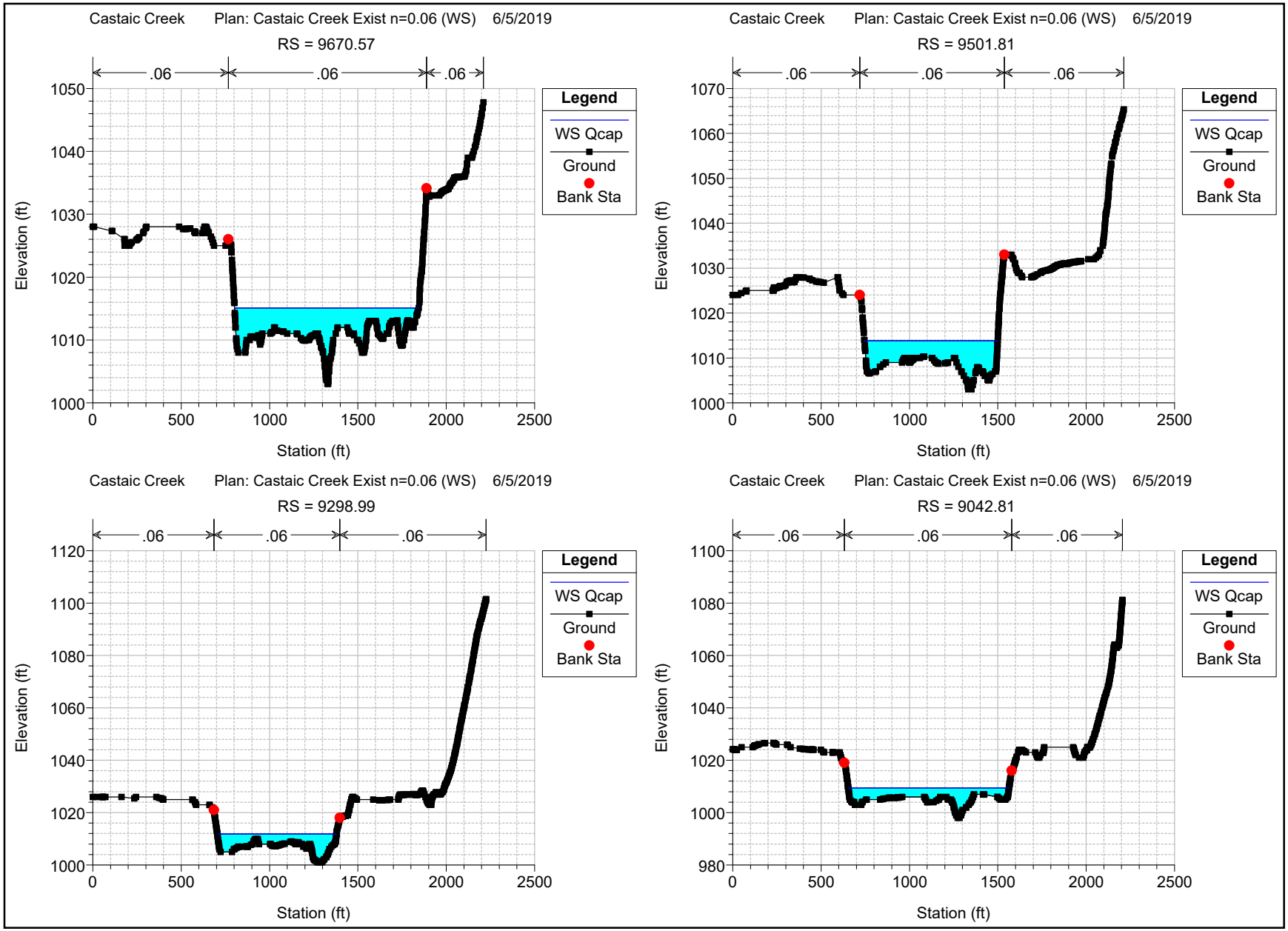
Ground
Bank Sta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Legend
WS Qcap
Ground
Bank Sta
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RS = 9501.81
.06
.06
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Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 9298.99
.06
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Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
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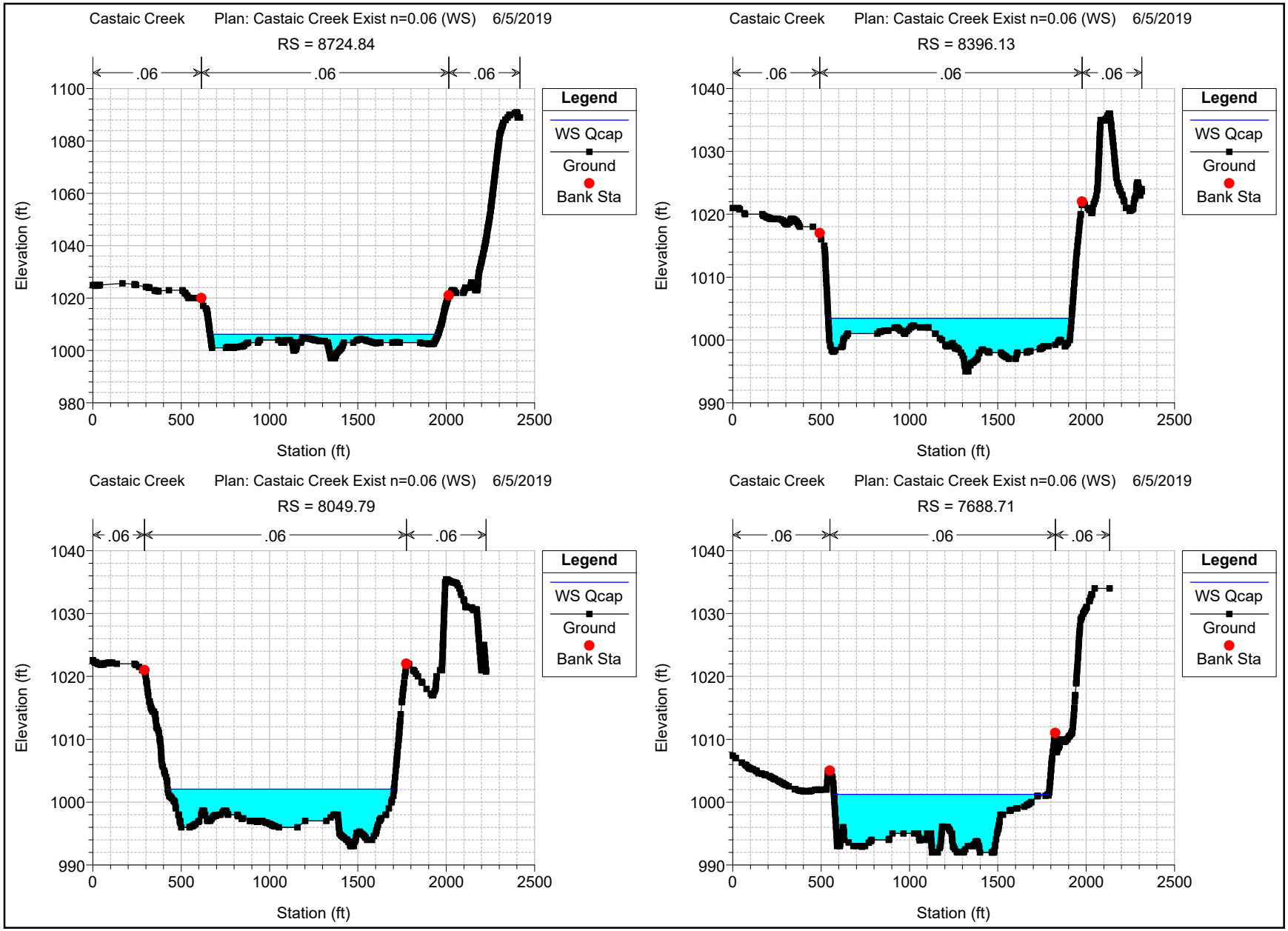
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta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Ground
Bank Sta
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Legend
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Ground
Bank Sta
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.06
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Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
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Station (ft)
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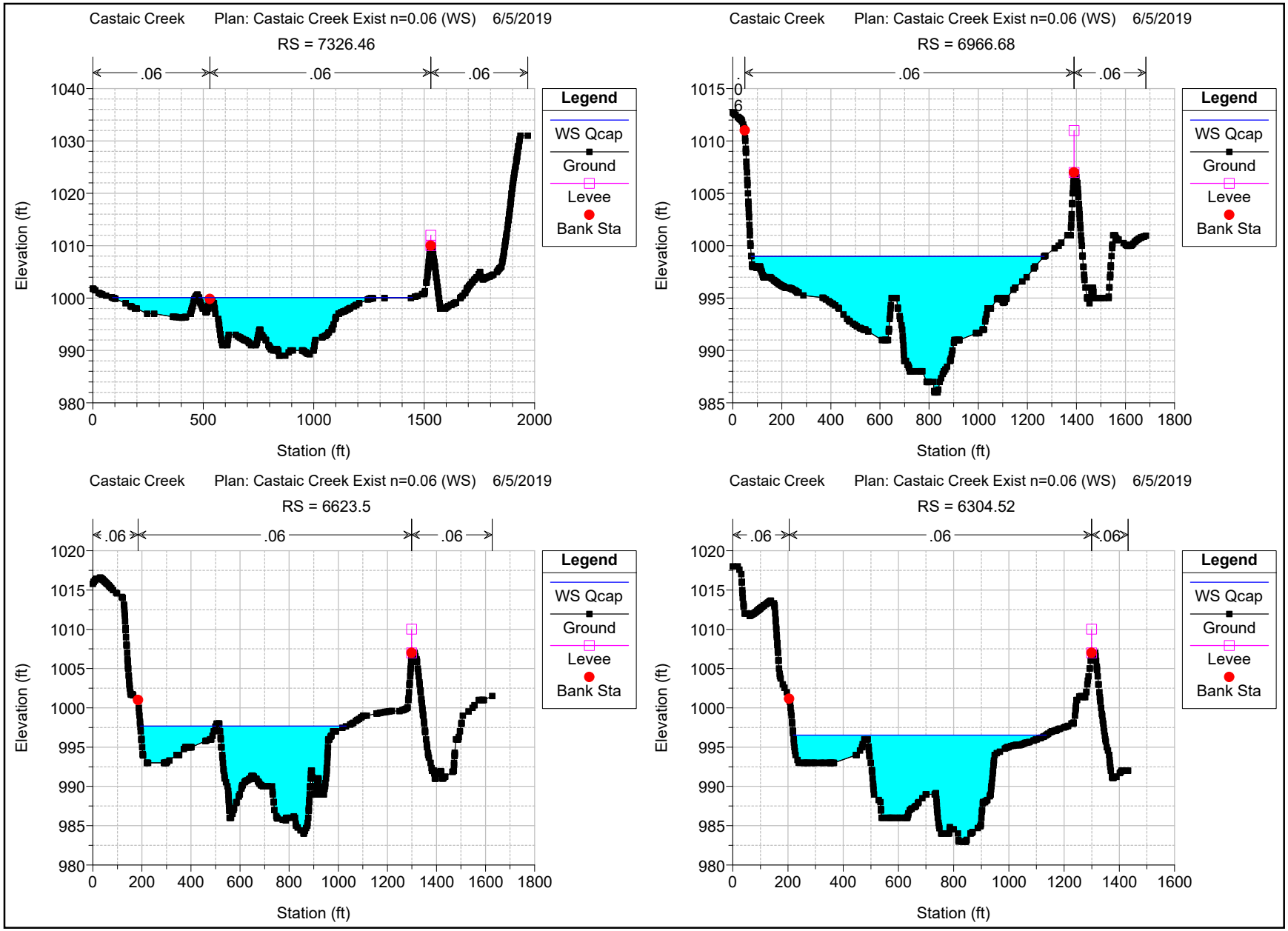
WS Qcap
Ground
Bank Sta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Station (ft)
Legend
WS Qcap
Ground
Levee
Bank Sta
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.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Levee
Bank Sta
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Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Levee
Bank Sta
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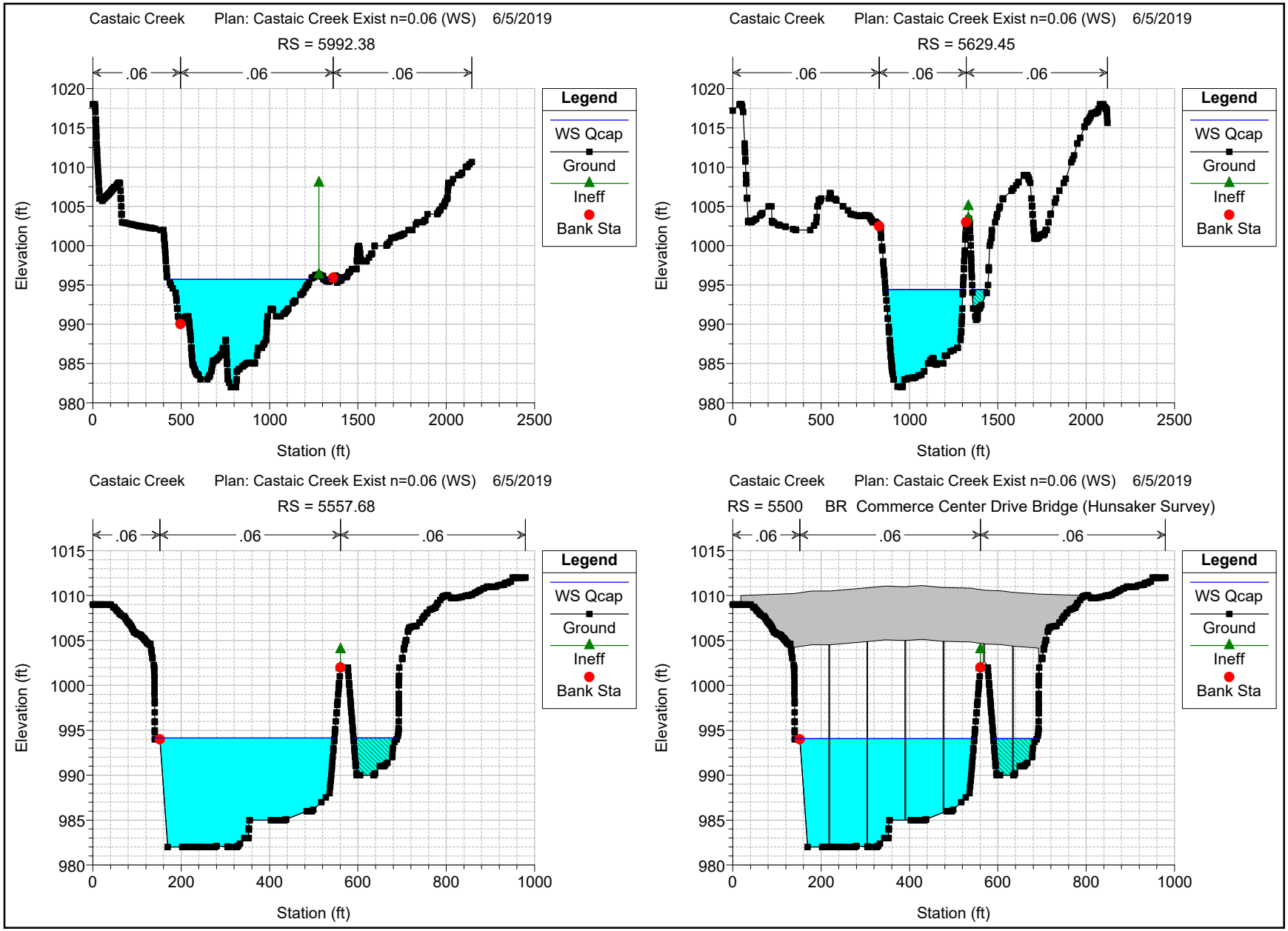
RS = 6304.52
.06
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Levee
Bank Sta


Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 5992.38
.06
.06
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Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 5629.45
.06
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 5557.68
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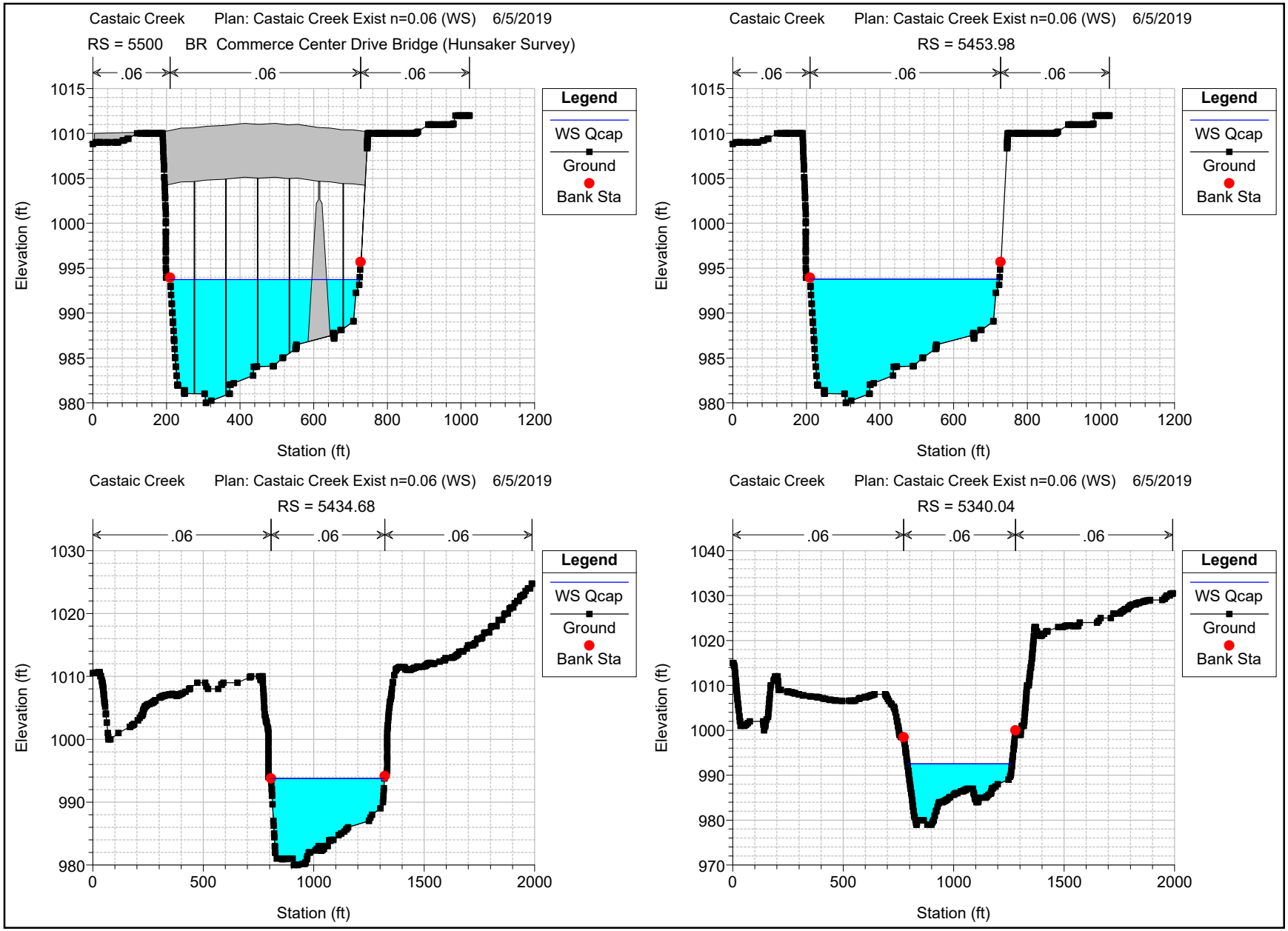
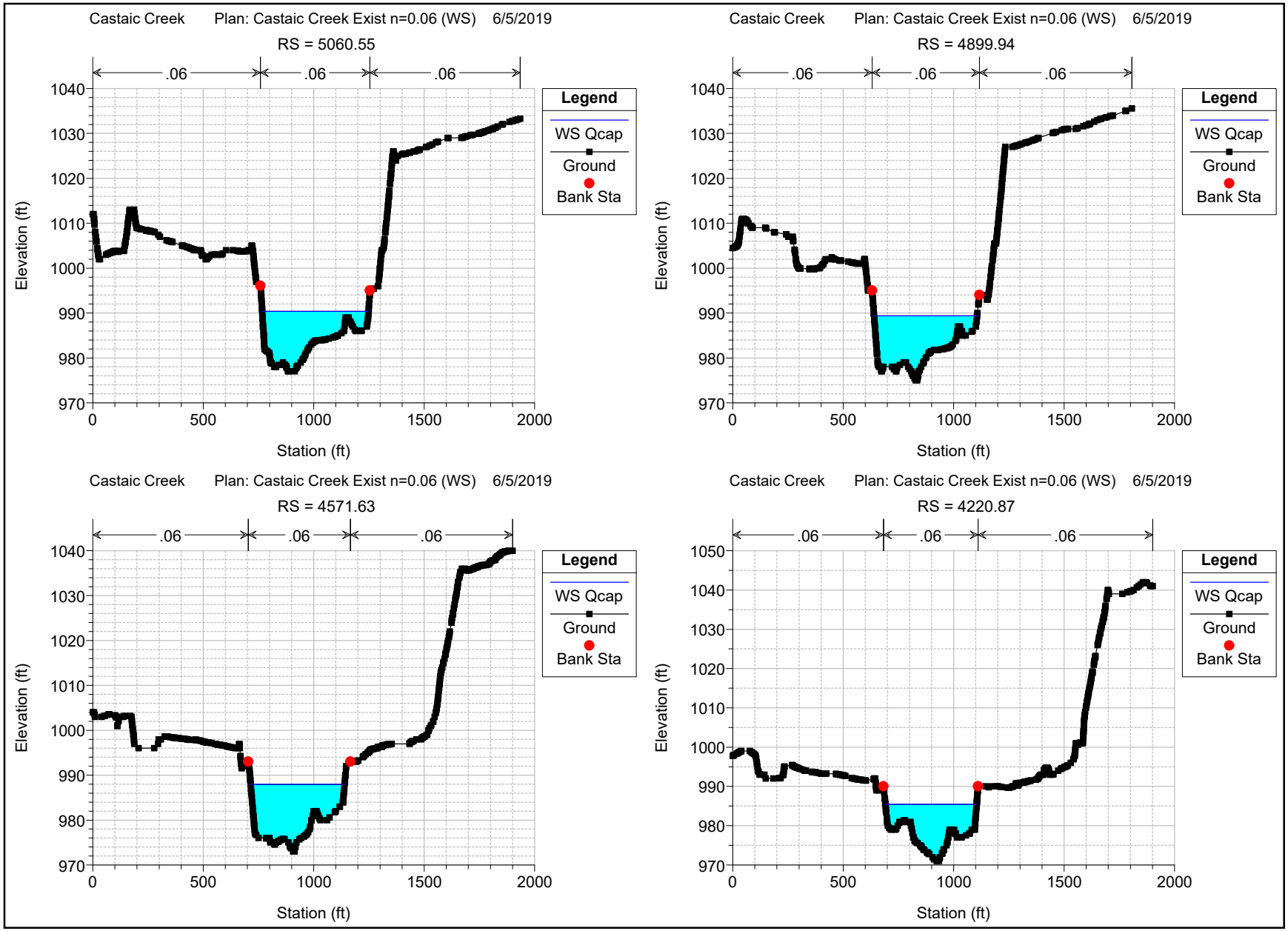
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 5500 BR Commerce Center Drive Bridge (Hunsaker Survey)
.06
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta


Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 5500 BR Commerce Center Drive Bridge (Hunsaker Survey)
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 5453.98
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 5434.68
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 5340.04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RS = 5060.55
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.06
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Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 4899.94
.06
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 4571.63
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.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
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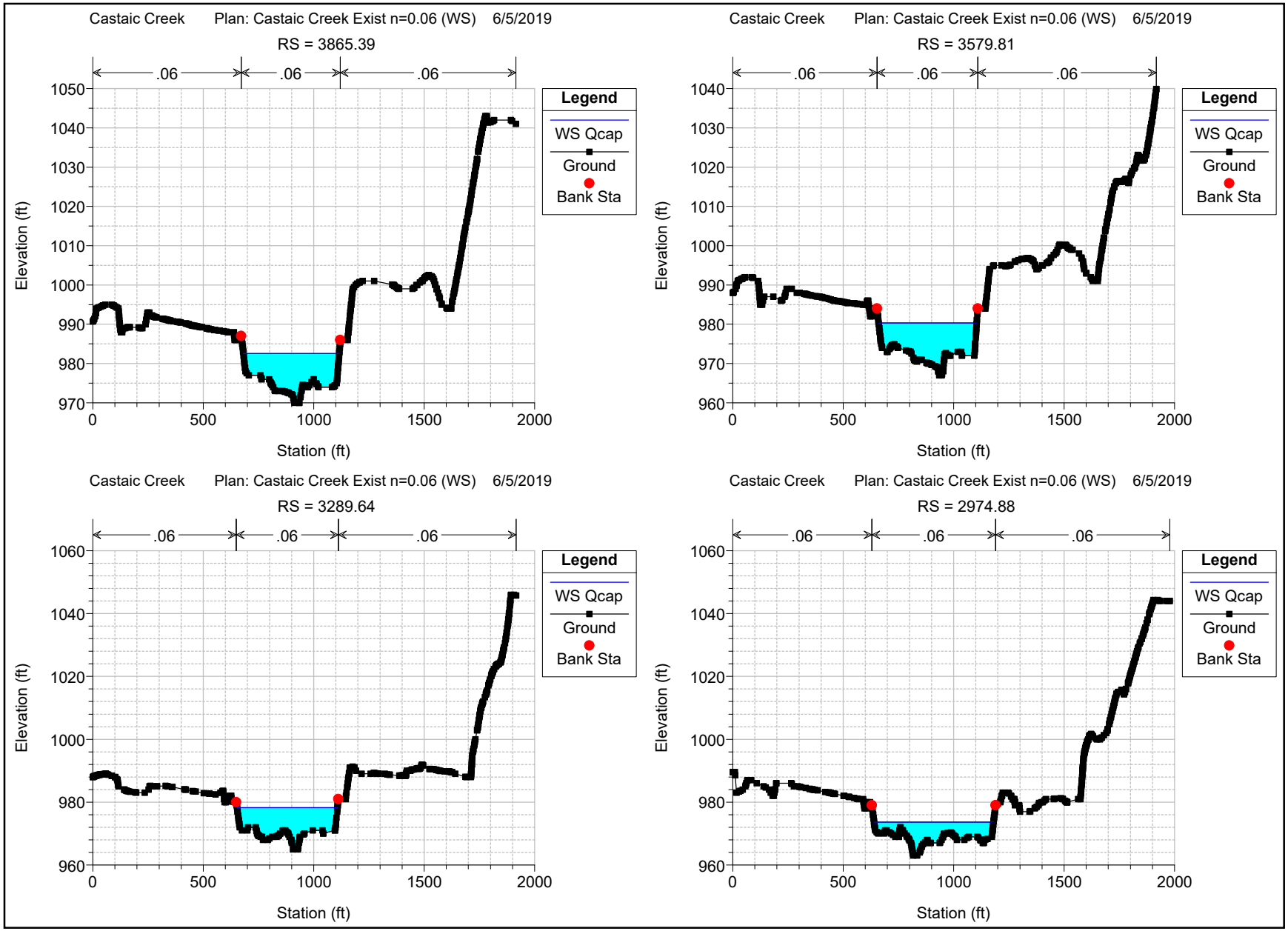
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta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RS = 3865.39
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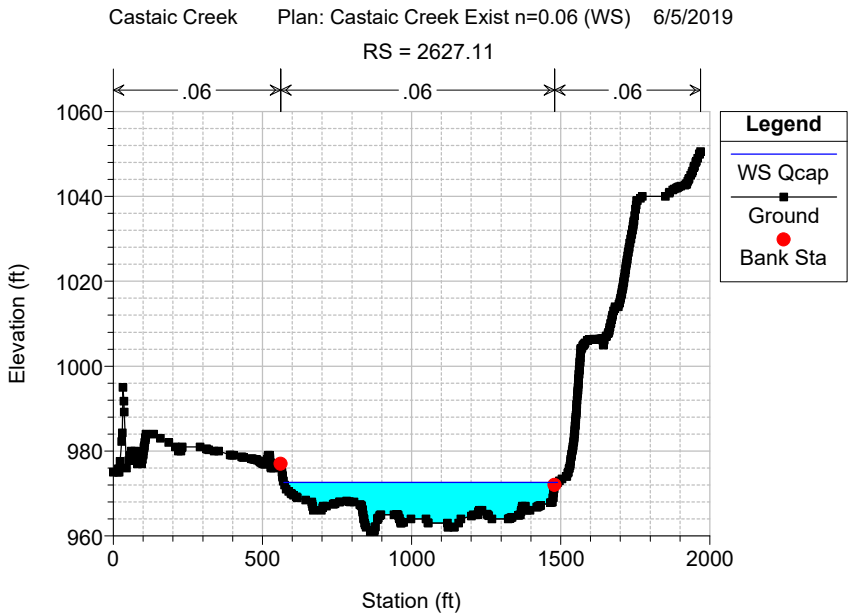
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 3579.81
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 3289.64
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 2974.88
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)


Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
RS = 2627.11
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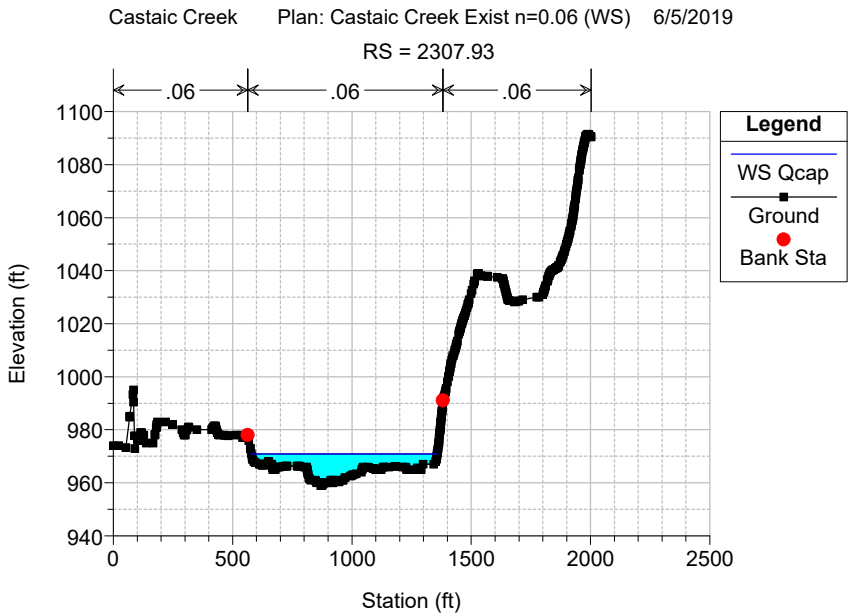
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta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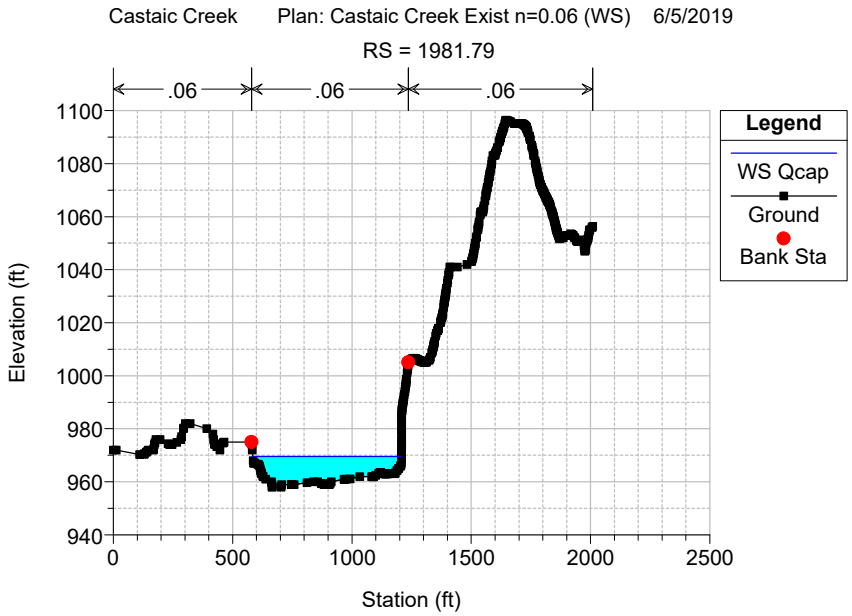
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
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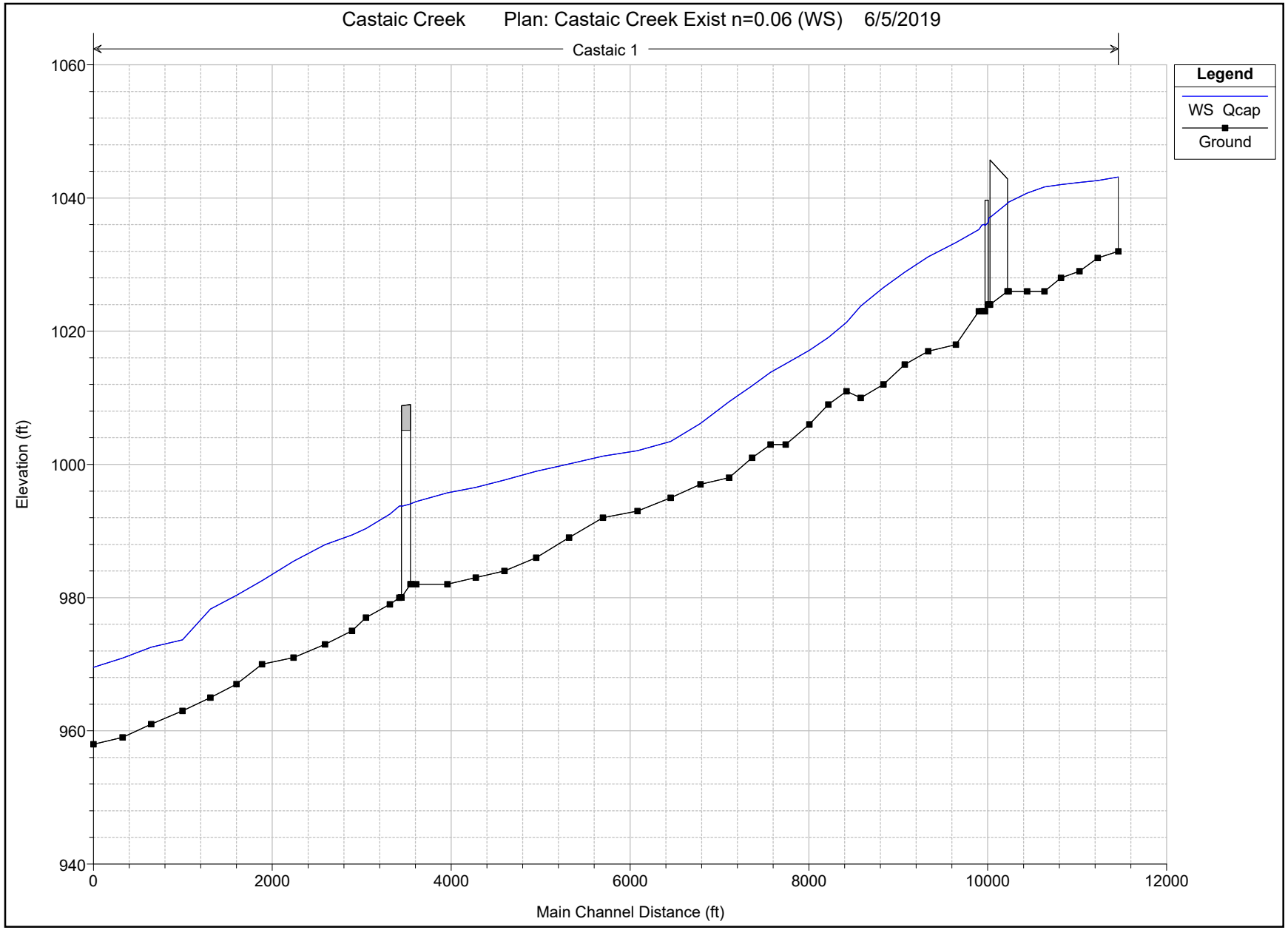
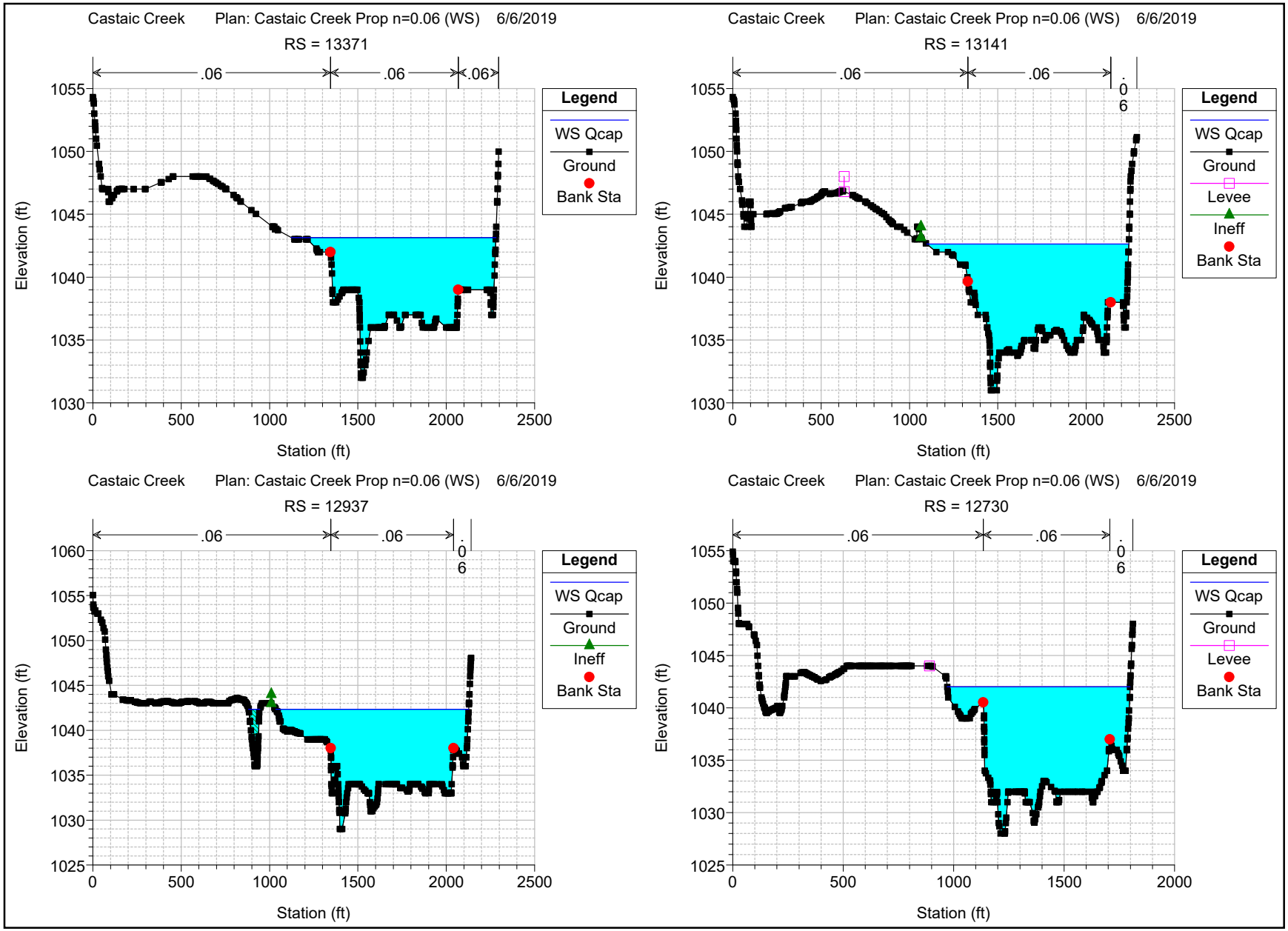
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Exist n=0.06 (WS) 6/5/2019
Castaic 1
Legend
WS Qcap
Ground
Elevation (ft)
1060
1040
1020
1000
980
960
940
0
2000
4000
6000
8000
10000
12000
Main Channel Distance (ft)

Appendix B – Proposed Condition
HEC-RAS Model Results (n=0.060)

HEC-RAS Plan: Prop n=0.06 River: Castaic Reach: 1 Profile: Qcap

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)	
1	13371	Qcap	26500.00	1032.00	1043.14	1040.16	1043.50	0.003404	4.98	5651.27	1157.84	0.35
1	13141	Qcap	26500.00	1031.00	1042.63	1038.45	1042.89	0.001937	4.14	6692.75	1142.67	0.27
1	12937	Qcap	26500.00	1029.00	1042.33	1036.97	1042.55	0.001327	3.86	7471.13	1154.99	0.23
1	12730	Qcap	26500.00	1028.00	1042.01	1035.81	1042.27	0.001293	4.16	6761.88	828.29	0.23
1	12543	Qcap	26500.00	1026.00	1041.66	1034.90	1042.00	0.001550	4.64	5716.77	972.72	0.25
1	12350	Qcap	26500.00	1026.00	1040.74	1034.86	1041.53	0.003261	7.19	3962.71	1125.91	0.37
1	12146	Qcap	26500.00	1026.00	1039.37	1034.52	1040.64	0.005321	9.03	2933.90	807.73	0.47
1	12059.65		Bridge									
1	11934	Qcap	26500.00	1024.00	1037.09	1033.58	1038.76	0.008008	10.35	2560.07	237.96	0.56
1	11923	Qcap	26500.00	1024.00	1036.95	1033.59	1038.66	0.008351	10.49	2526.91	237.88	0.57
1	11917.82		Bridge									
1	11873	Qcap	26500.00	1023.00	1036.03	1032.64	1037.58	0.007917	9.98	2654.47	272.63	0.56
1	11846	Qcap	26500.00	1023.00	1035.94	1032.52	1037.31	0.008187	9.39	2823.52	325.14	0.56
1	11811.76	Qcap	26500.00	1023.00	1035.26	1032.65	1036.97	0.010153	10.47	2534.04	356.18	0.63
1	11551.18	Qcap	26500.00	1018.00	1033.31	1030.29	1034.43	0.008278	8.51	3114.91	579.04	0.55
1	11242.98	Qcap	26500.00	1017.00	1031.18	1027.80	1032.20	0.006204	8.11	3266.30	614.24	0.49
1	10982.86	Qcap	26500.00	1015.00	1028.86	1026.41	1030.20	0.009271	9.30	2850.94	696.04	0.59
1	10738.26	Qcap	26500.00	1012.00	1026.59	1024.64	1027.72	0.011178	8.52	3108.51	858.84	0.62
1	10490.82	Qcap	26500.00	1010.00	1023.66	1021.97	1024.59	0.013009	7.75	3420.52	816.83	0.64
1	10351.28	Qcap	26500.00	1011.00	1020.91		1022.28	0.015946	9.38	2824.11	538.44	0.72
1	10141.82	Qcap	26500.00	1009.00	1018.88		1019.59	0.009552	6.80	3895.67	823.16	0.55
1	9931.16	Qcap	26500.00	1006.00	1017.25		1017.79	0.007170	5.90	4487.97	945.97	0.48
1	9670.57	Qcap	26500.00	1003.00	1014.01		1014.91	0.018367	7.58	3496.96	1024.58	0.72
1	9501.81	Qcap	26500.00	1003.00	1012.41		1012.92	0.007257	5.76	4599.58	1015.87	0.48
1	9298.99	Qcap	26500.00	1001.00	1010.95		1011.45	0.006955	5.66	4683.96	1029.02	0.47
1	9042.81	Qcap	26500.00	998.00	1009.09		1009.58	0.007629	5.58	4749.53	1143.00	0.48
1	8724.84	Qcap	26500.00	997.00	1005.98		1006.52	0.012096	5.93	4469.90	1387.29	0.58
1	8396.13	Qcap	26500.00	995.00	1003.65		1003.97	0.005071	4.51	5879.38	1436.50	0.39
1	8049.79	Qcap	26500.00	993.00	1002.67		1002.86	0.001869	3.49	7596.15	1288.85	0.25
1	7688.71	Qcap	26500.00	992.00	1002.17		1002.32	0.001028	3.18	8330.08	1034.11	0.20
1	7326.46	Qcap	26500.00	989.00	1001.53		1001.81	0.001753	4.29	6175.94	730.53	0.26
1	6966.68	Qcap	26500.00	986.00	1000.87		1001.19	0.001636	4.47	5932.92	625.47	0.26
1	6623.5	Qcap	26500.00	984.00	999.69		1000.32	0.003650	6.40	4140.83	461.00	0.38
1	6304.52	Qcap	26500.00	983.00	998.04		998.95	0.004799	7.66	3459.41	361.85	0.44
1	5992.38	Qcap	26500.00	982.00	996.49	992.70	997.22	0.005871	6.85	3866.43	725.60	0.46
1	5629.45	Qcap	26500.00	983.00	994.21	990.54	995.19	0.005663	7.92	3344.26	460.50	0.47
1	5557.68	Qcap	26500.00	982.00	994.15	989.28	994.86	0.003603	6.78	3909.78	505.46	0.38
1	5500		Bridge									
1	5453.98	Qcap	26500.00	980.01	993.78		994.26	0.002637	5.57	4760.93	514.37	0.32
1	5434.68	Qcap	26500.00	980.00	993.75		994.20	0.002334	5.37	4930.63	513.00	0.31
1	5340.04	Qcap	31400.00	979.00	992.52		993.69	0.008231	8.67	3621.76	473.85	0.55
1	5060.55	Qcap	31400.00	977.00	990.36		991.50	0.008113	8.57	3664.71	481.55	0.55
1	4899.94	Qcap	31400.00	975.00	989.40		990.37	0.005974	7.91	3967.35	466.52	0.48
1	4571.63	Qcap	31400.00	973.00	987.95		988.81	0.004304	7.44	4219.29	425.08	0.42
1	4220.87	Qcap	31400.00	971.00	985.47		986.75	0.008052	9.08	3458.47	413.41	0.55
1	3865.39	Qcap	31400.00	970.00	982.54		983.82	0.008516	9.06	3464.55	433.91	0.57
1	3579.81	Qcap	31400.00	967.00	980.35		981.51	0.007489	8.63	3638.83	444.02	0.53
1	3289.64	Qcap	31400.00	965.00	978.28		979.38	0.007109	8.42	3730.90	455.00	0.52
1	2974.88	Qcap	31400.00	963.00	973.65	973.14	975.60	0.022964	11.18	2807.58	540.79	0.87
1	2627.11	Qcap	31400.00	961.00	972.57		972.94	0.002888	4.89	6425.14	918.08	0.32
1	2307.93	Qcap	31400.00	959.00	970.93		971.59	0.006238	6.55	4796.13	778.97	0.47
1	1981.79	Qcap	31400.00	958.00	969.53	965.20	970.09	0.003439	5.98	5248.28	622.45	0.36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RS = 13371
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 13141
.06
.06
.06
Legend
WS Qcap
Ground
Levee
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 12937
.06
.06
.06
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
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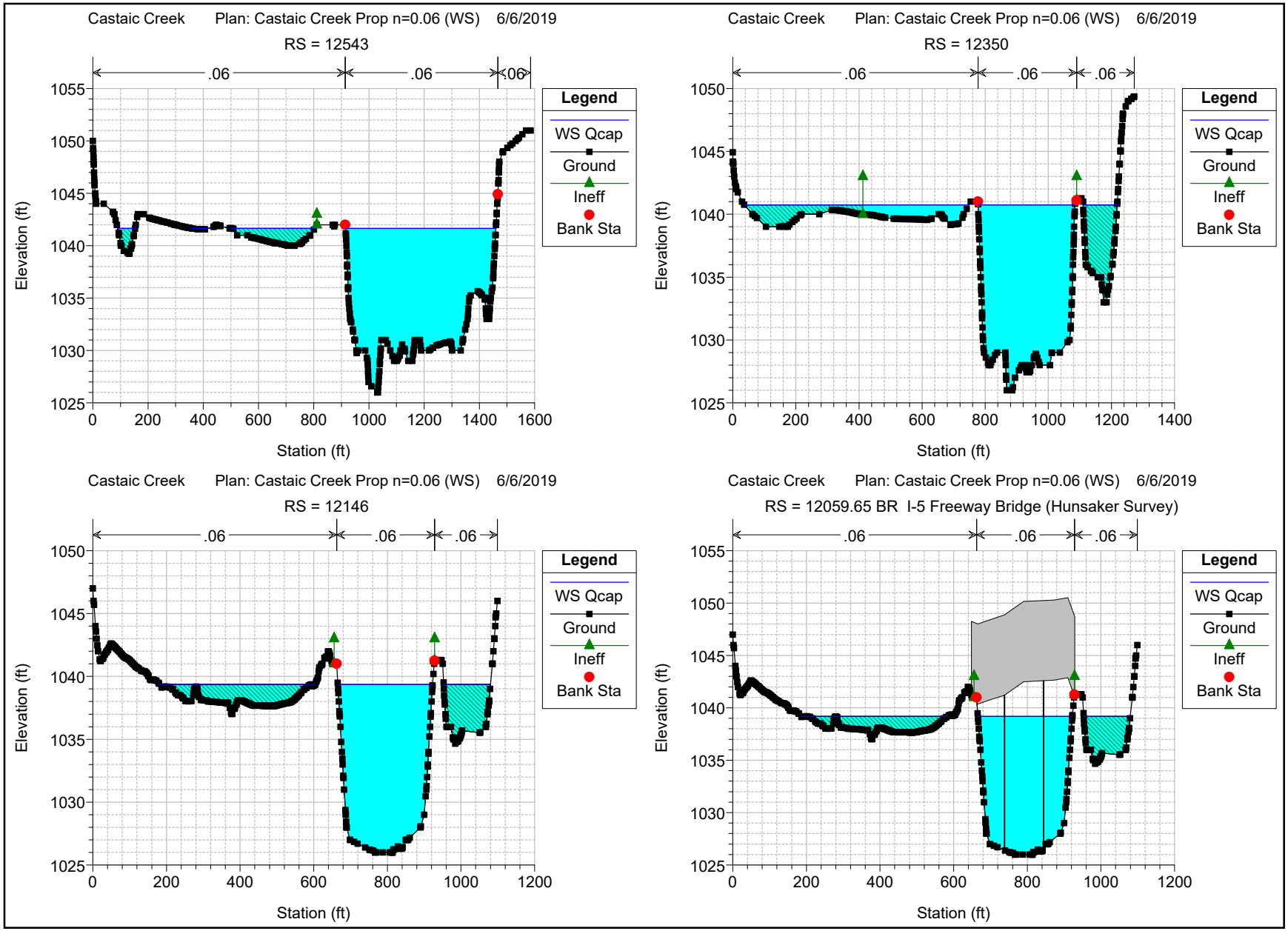
Legend
WS Qcap
Ground
Levee
Bank Sta
Elevation (ft)
Station (ft)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RS = 12543
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.06
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Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 12350
.06
.06
.06
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 12146
.06
.06
.06
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 12059.65 BR I-5 Freeway Bridge (Hunsaker Survey)
.06
.06
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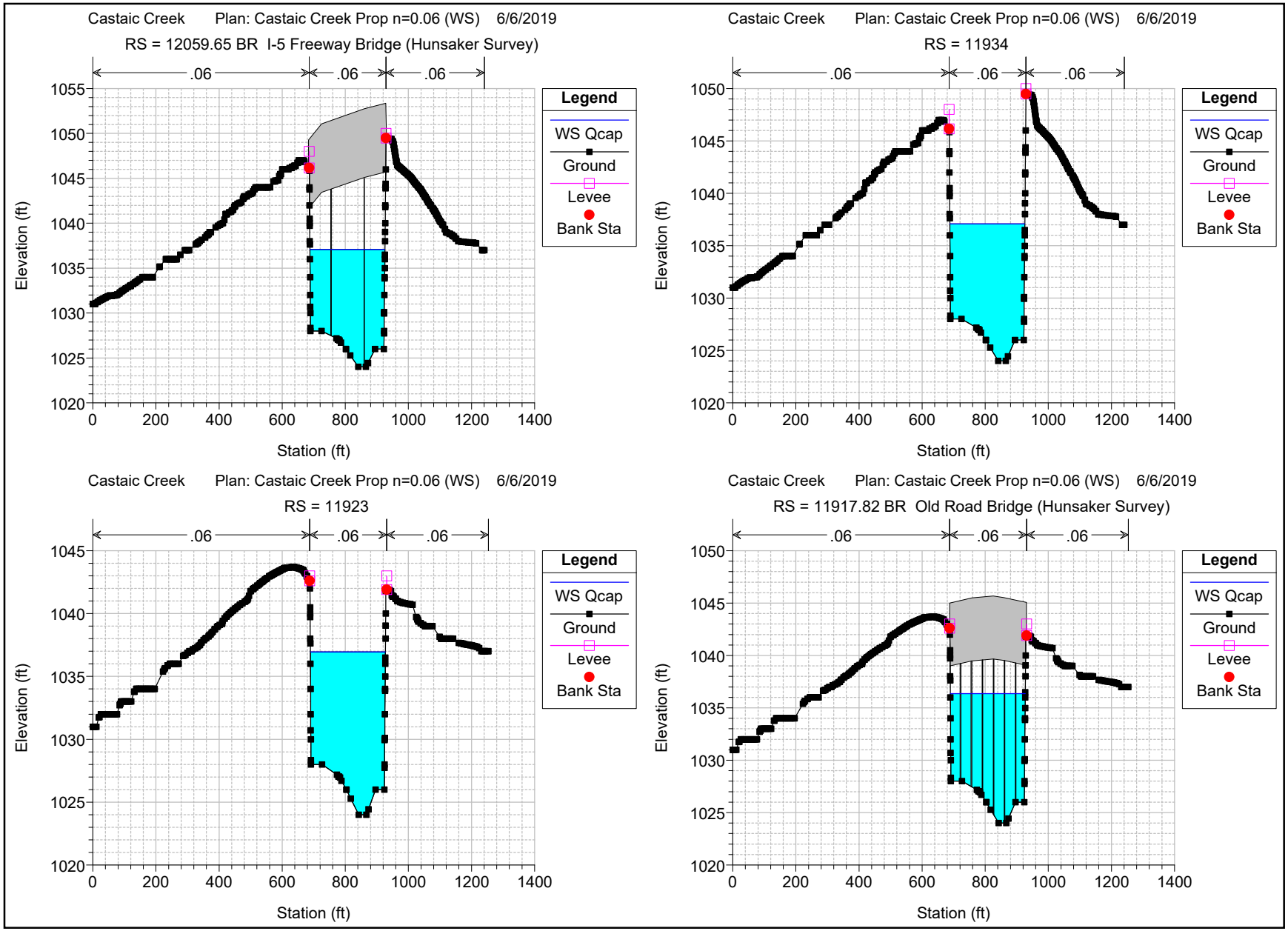
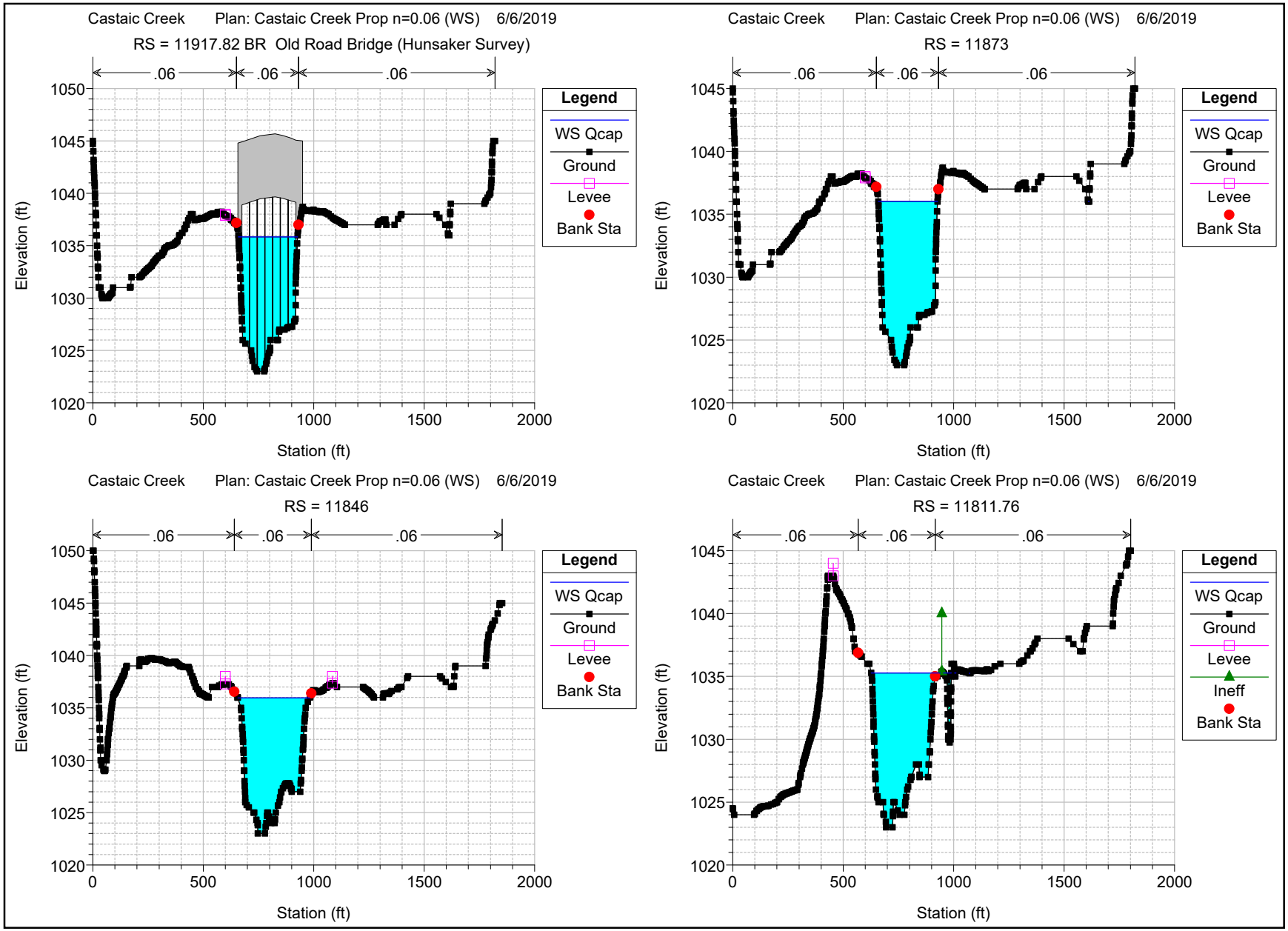
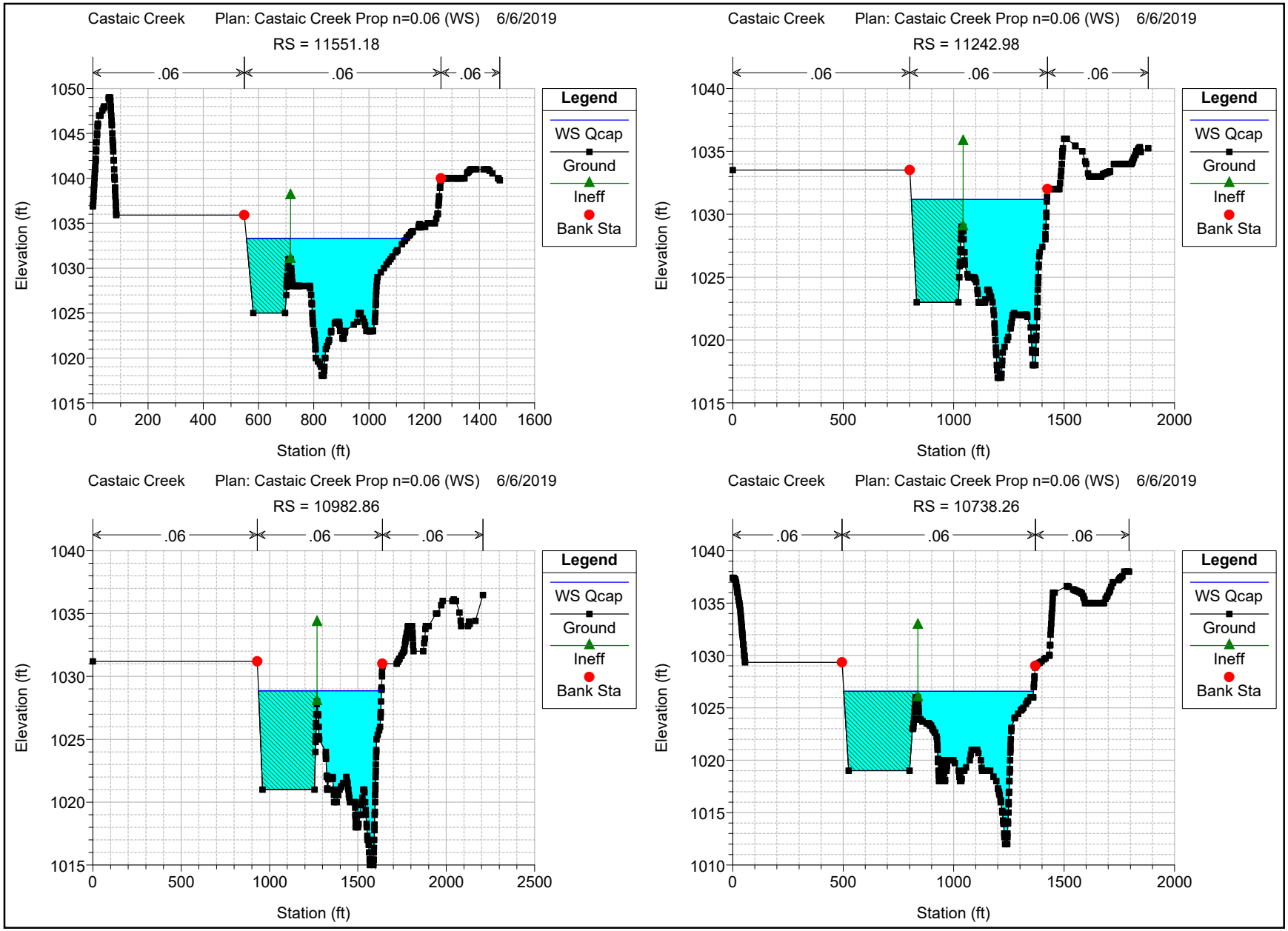
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 12059.65 BR I-5 Freeway Bridge (Hunsaker Survey)
.06
.06
.06
Legend
WS Qcap
Ground
Levee
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 11934
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 11923
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 11917.82 BR Old Road Bridge (Hunsaker Survey)


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 11917.82 BR Old Road Bridge (Hunsaker Survey)
.06
.06
.06
Legend
WS Qcap
Ground
Levee
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 11873
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 11846
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 11811.76
Ineff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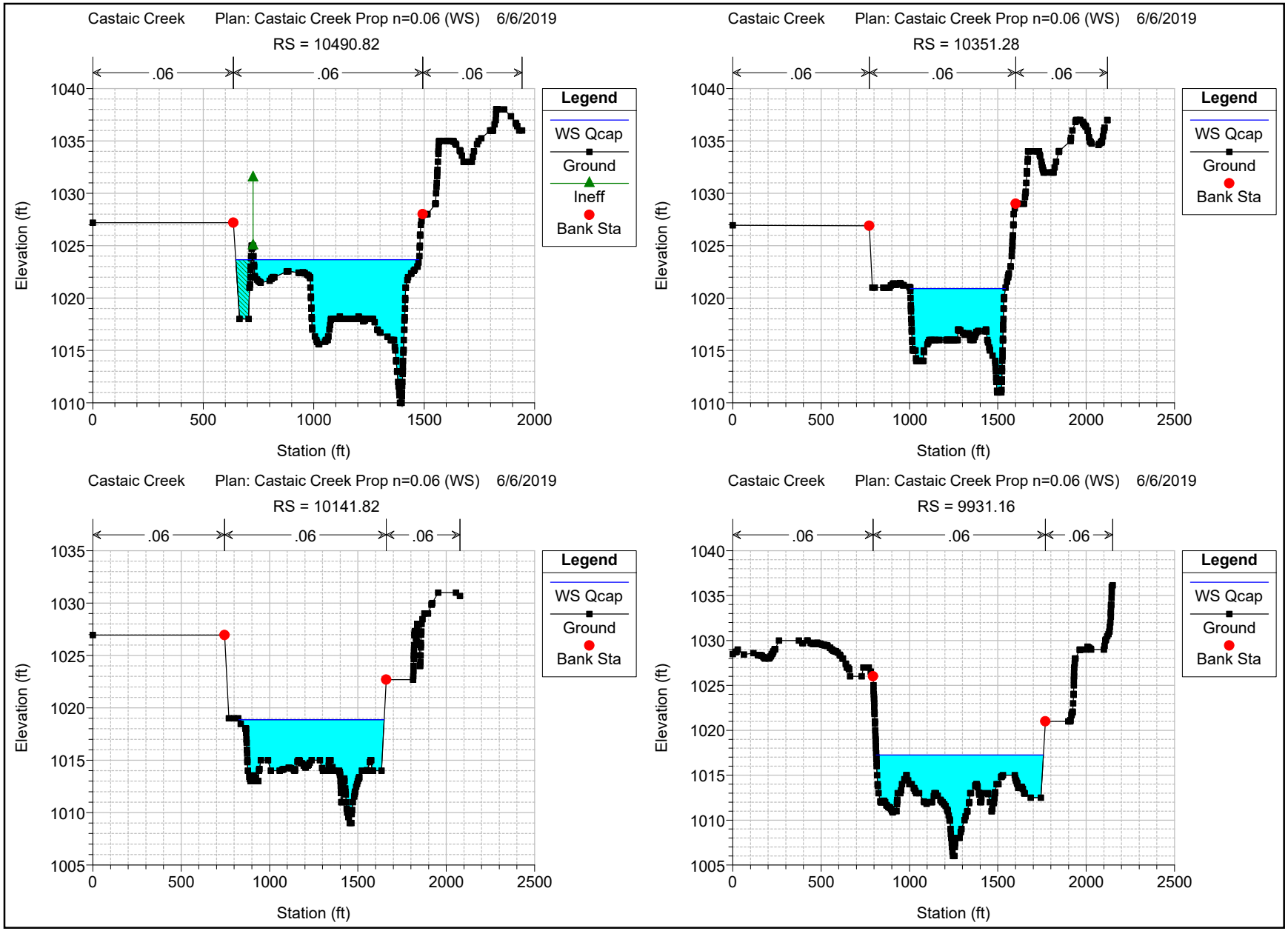
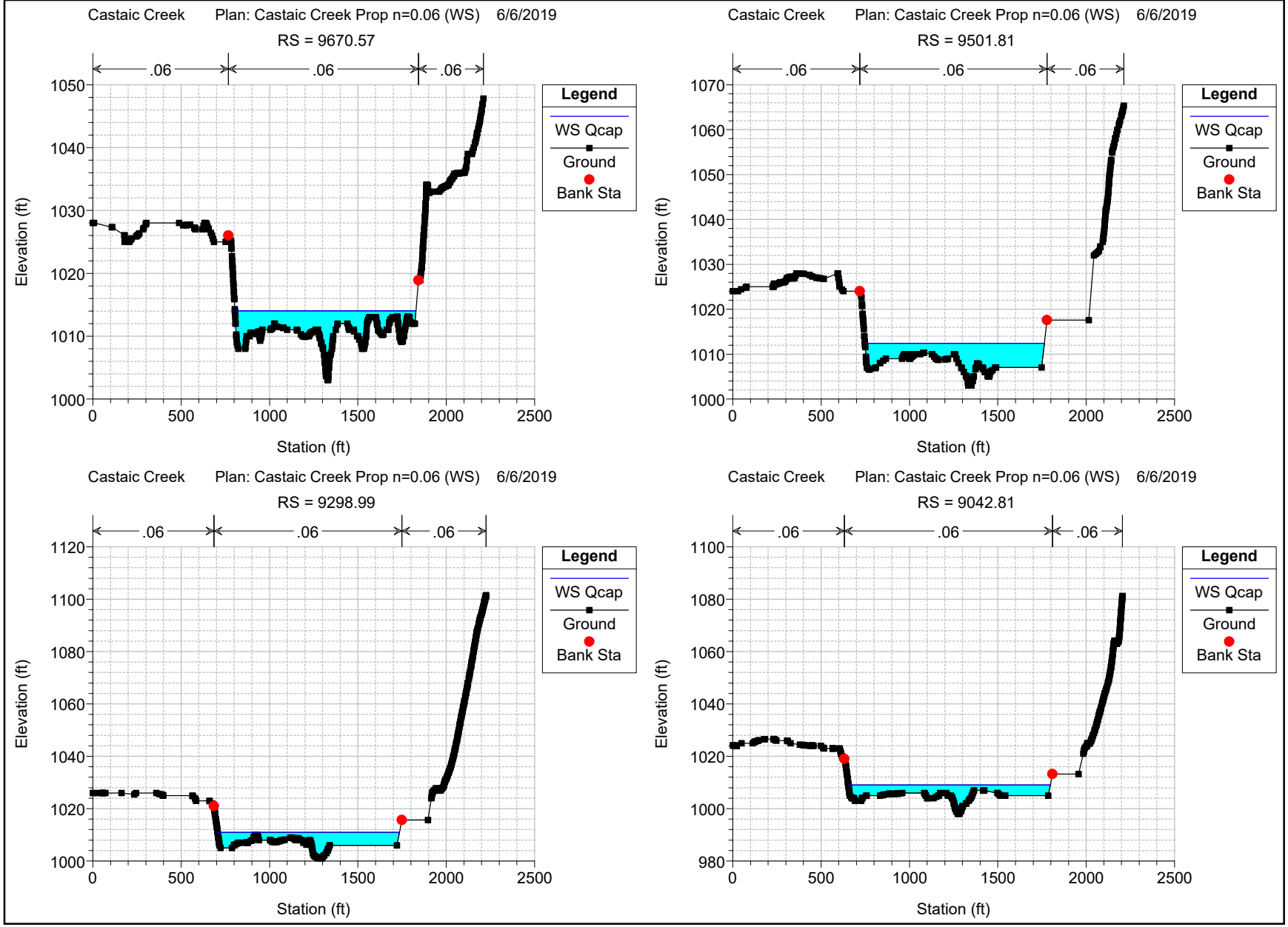
RS = 11551.18
.06
.06
.06
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 11242.98
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 10982.86
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 10738.26


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 10490.82
.06
.06
.06
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 10351.28
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 10141.82
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 9931.16
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)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RS = 9670.57
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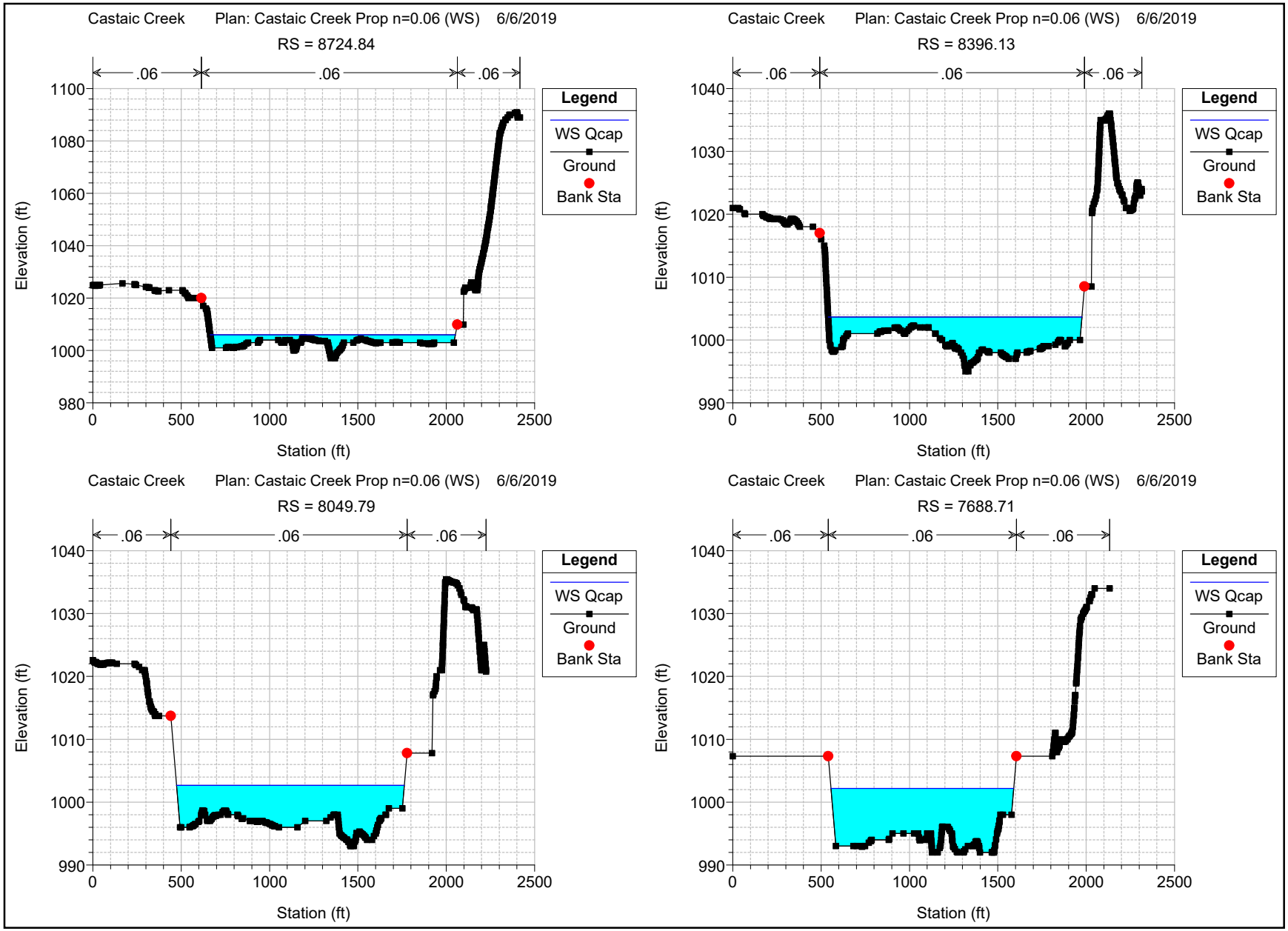
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 9501.81
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 9298.99
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 9042.81
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 8724.84
.06
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 8396.13
.06
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 8049.79
.06
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 7688.71
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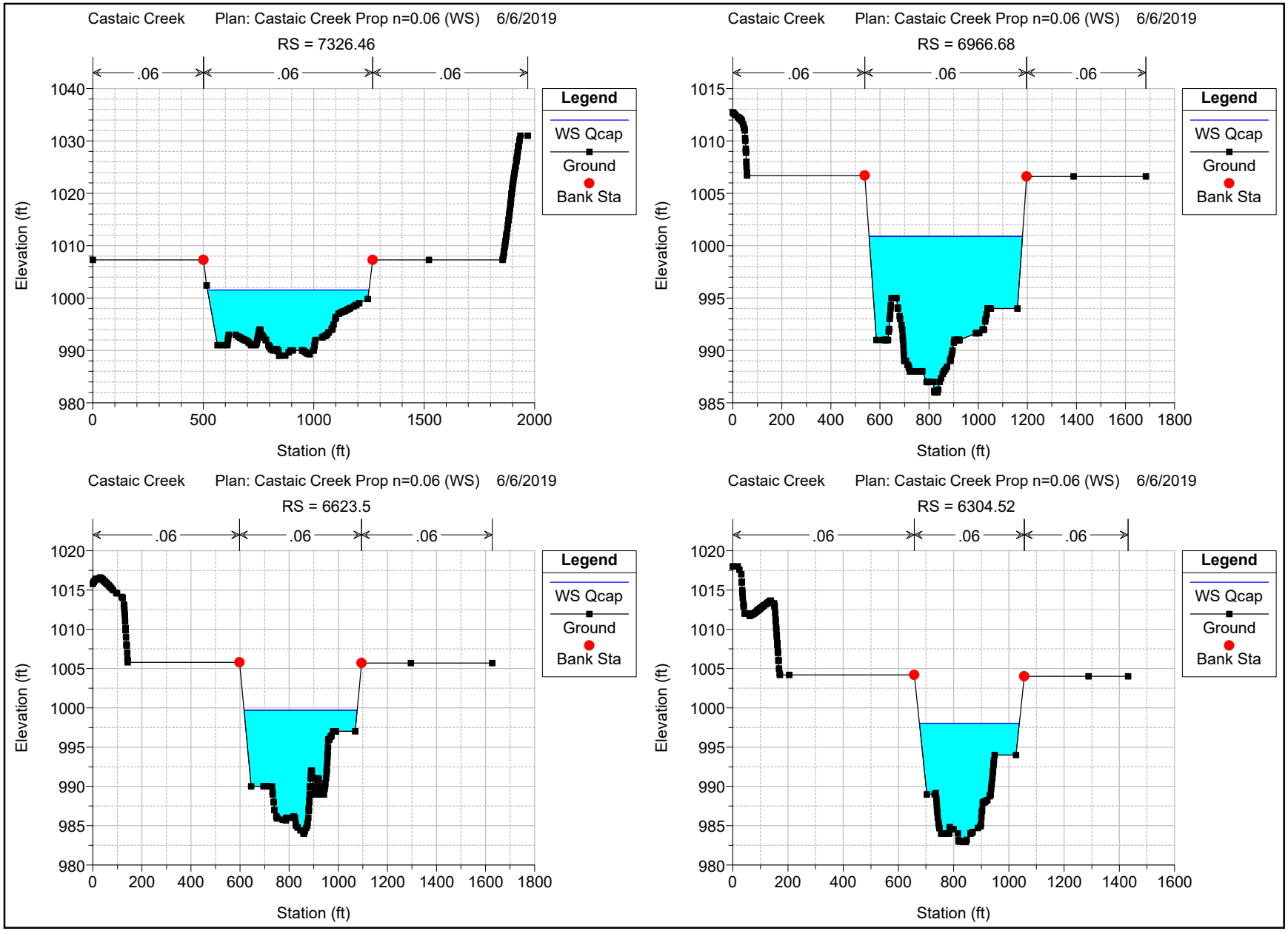
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta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RS = 7326.46
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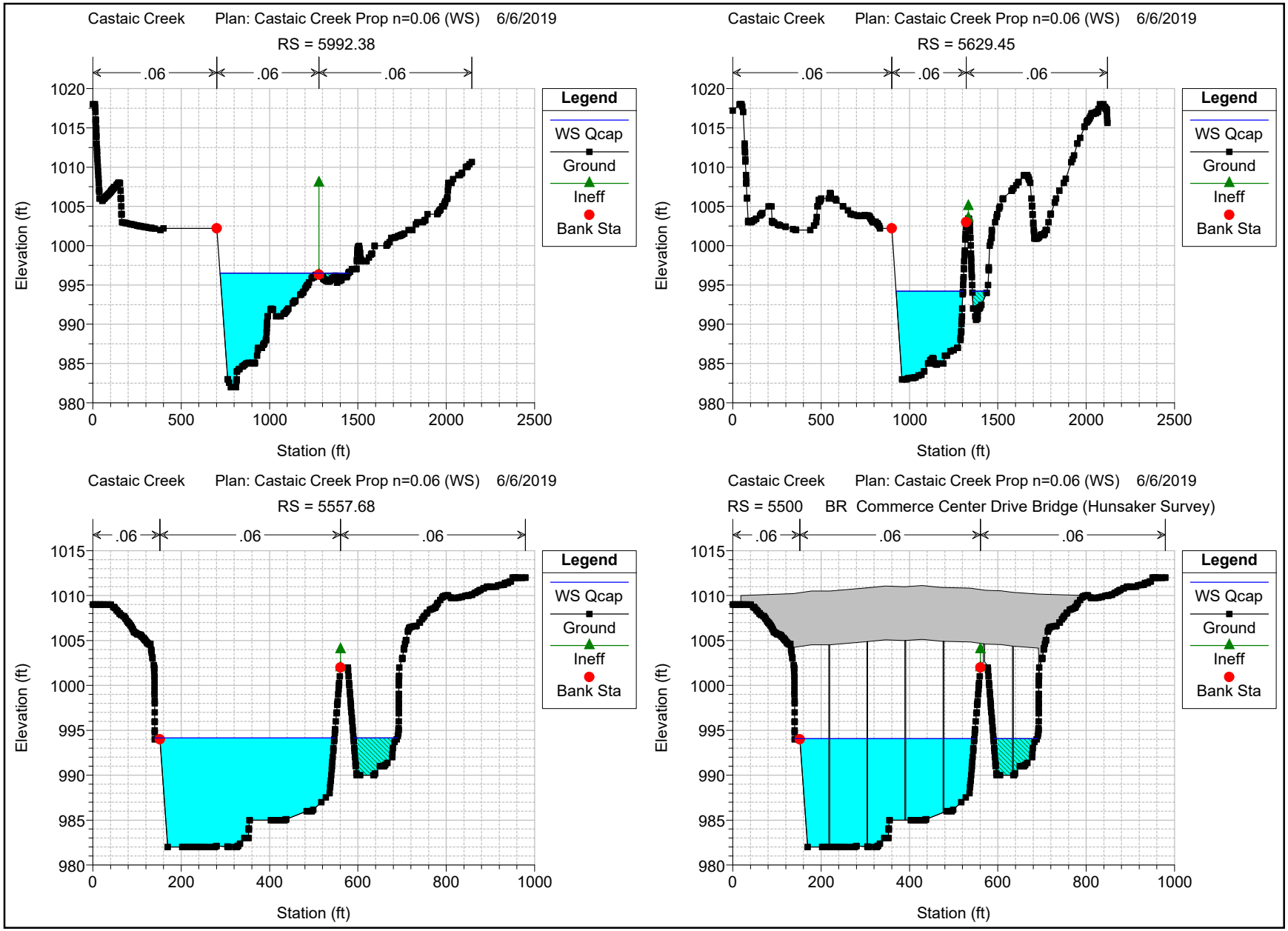
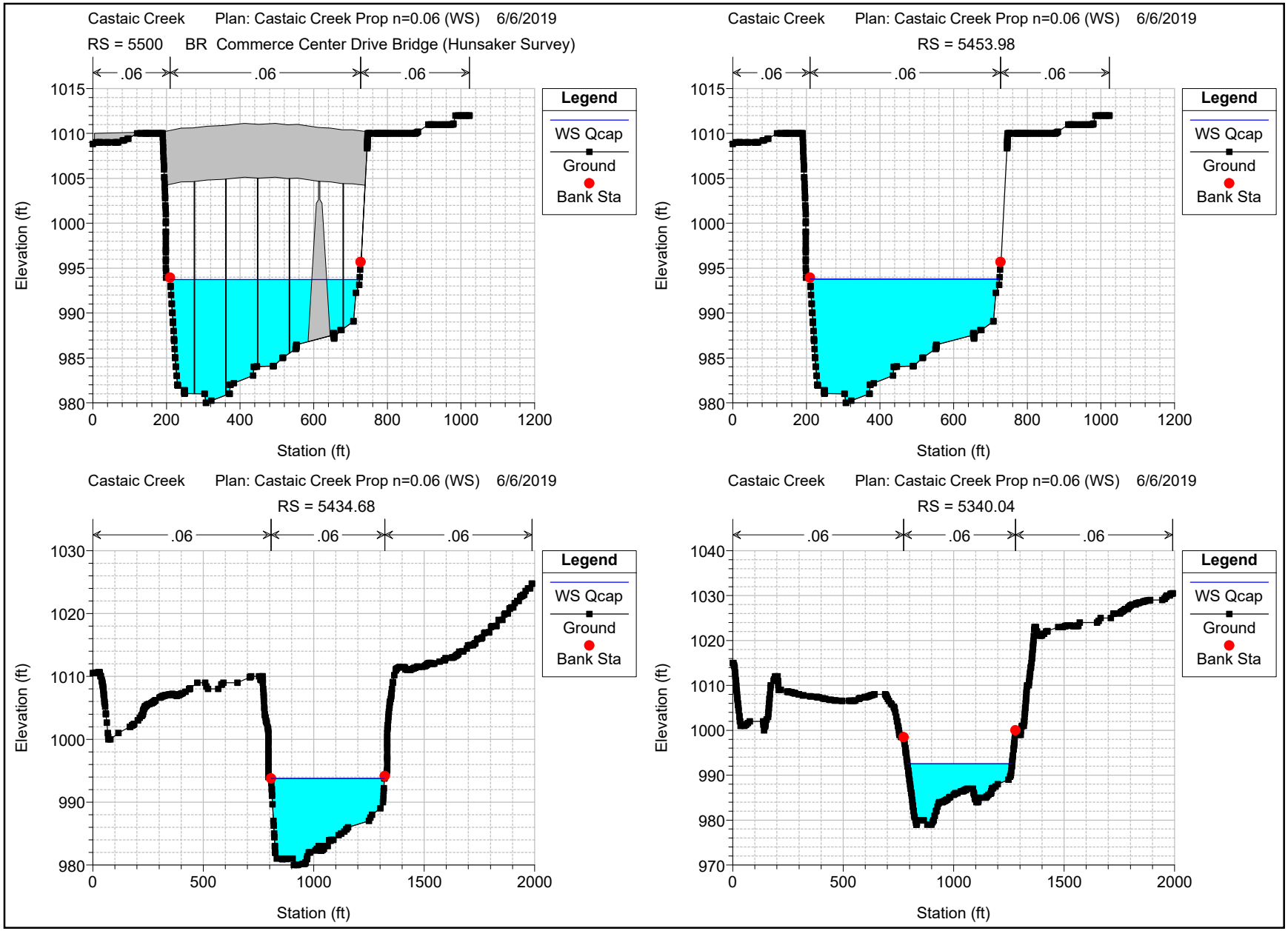
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 6966.68
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 6623.5
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 6304.52


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 5992.38
.06
.06
.06
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 5629.45
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 5557.68
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 5500 BR Commerce Center Drive Bridge (Hunsaker Survey)


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 5500 BR Commerce Center Drive Bridge (Hunsaker Survey)
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 5453.98
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 5434.68
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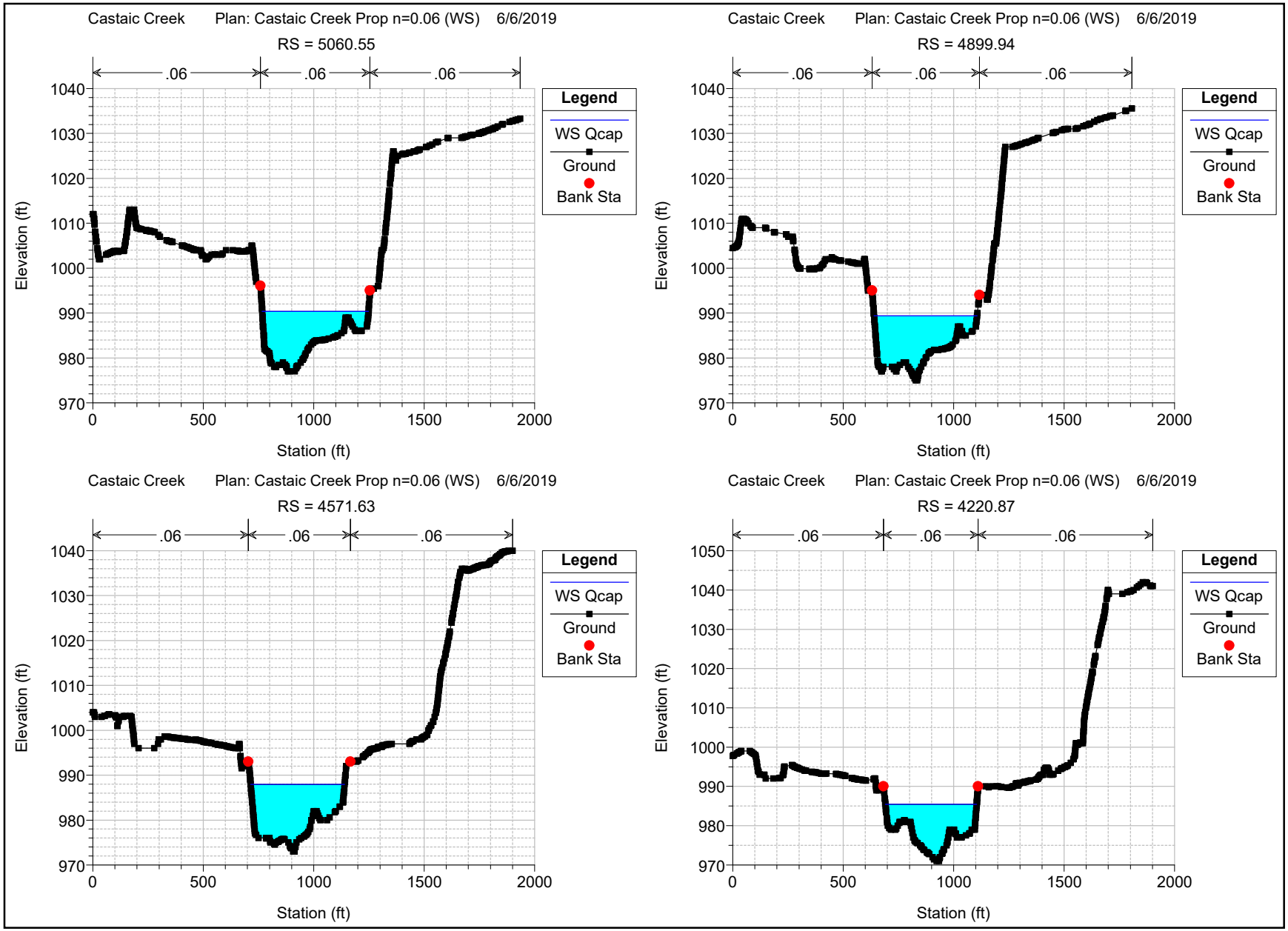
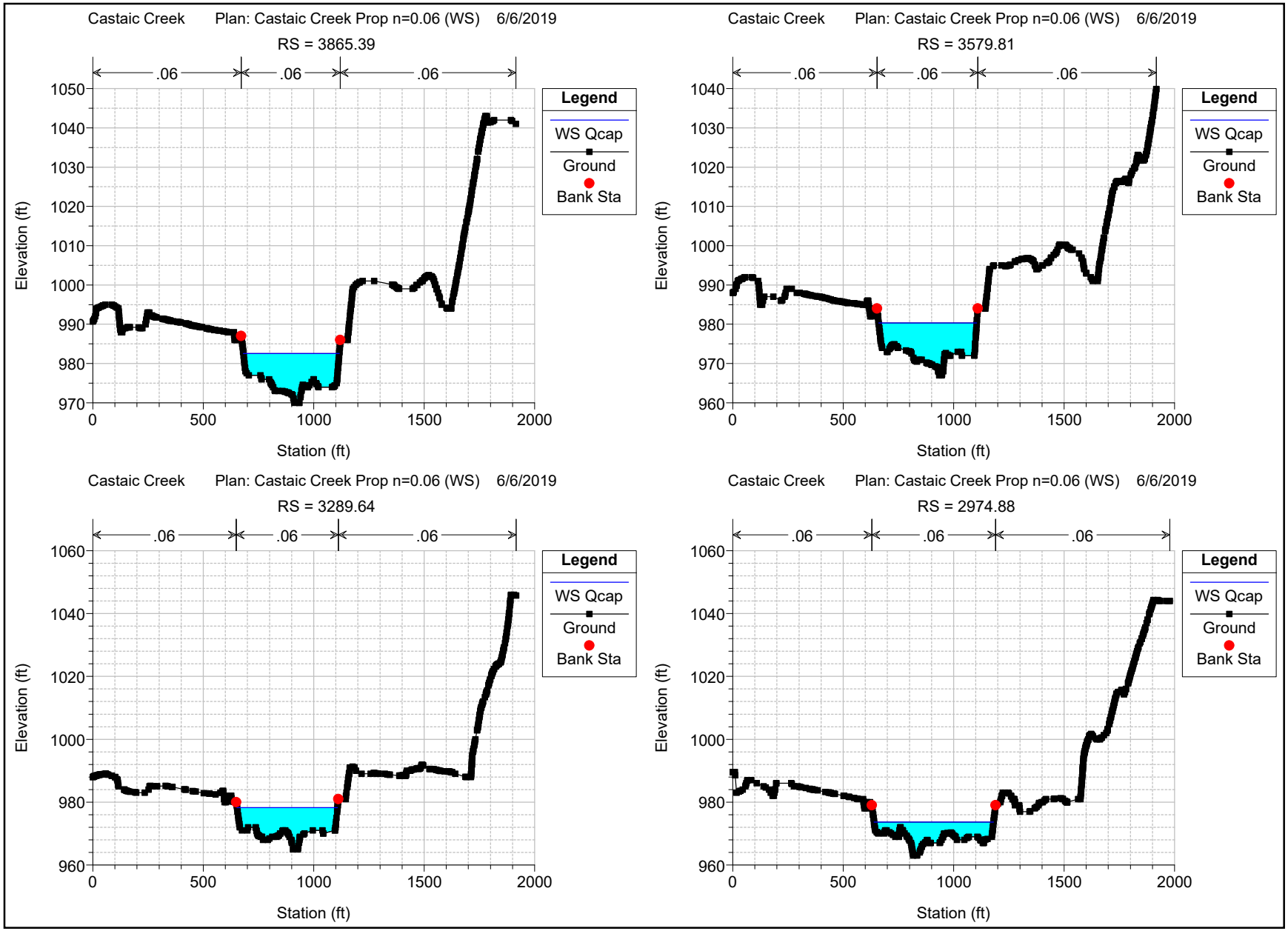
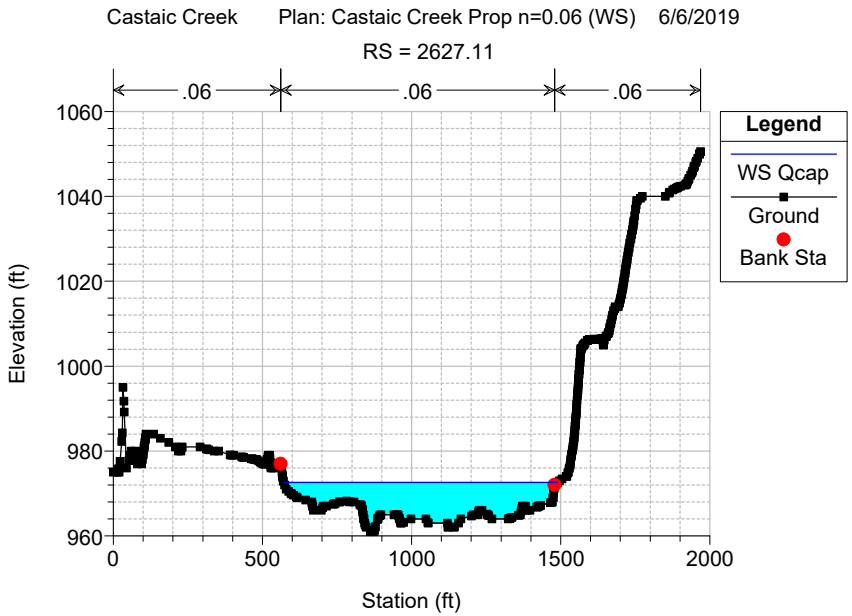
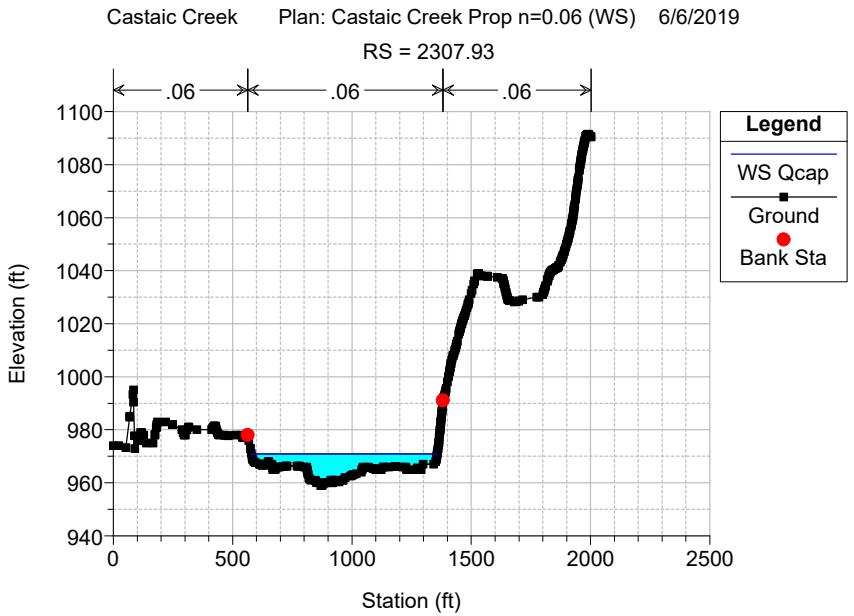
RS = 5340.04


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 5060.55
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 4899.94
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 4571.63
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 4220.87
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 3865.39
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 3579.81
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 3289.64
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 2974.88
.06
.06
.06
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 2627.11
.06
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 2307.93
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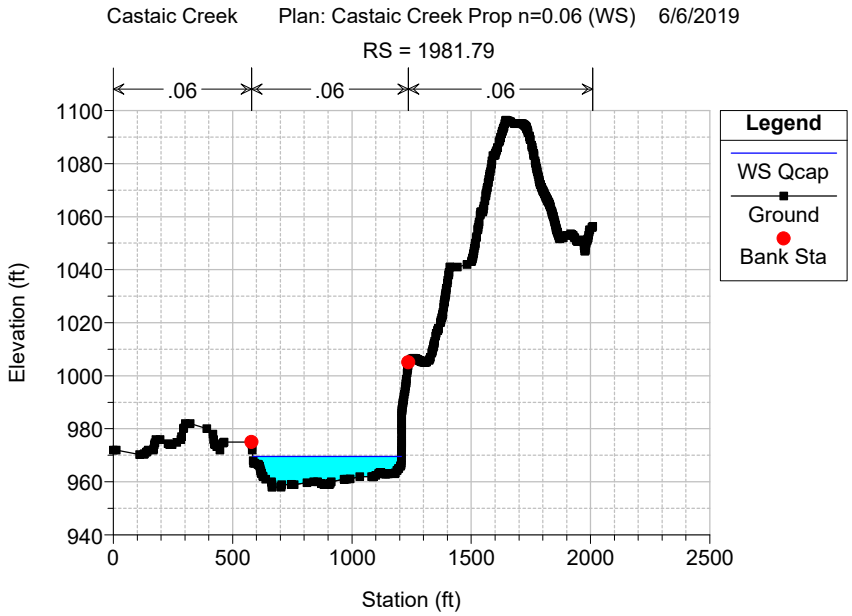
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
RS = 1981.79
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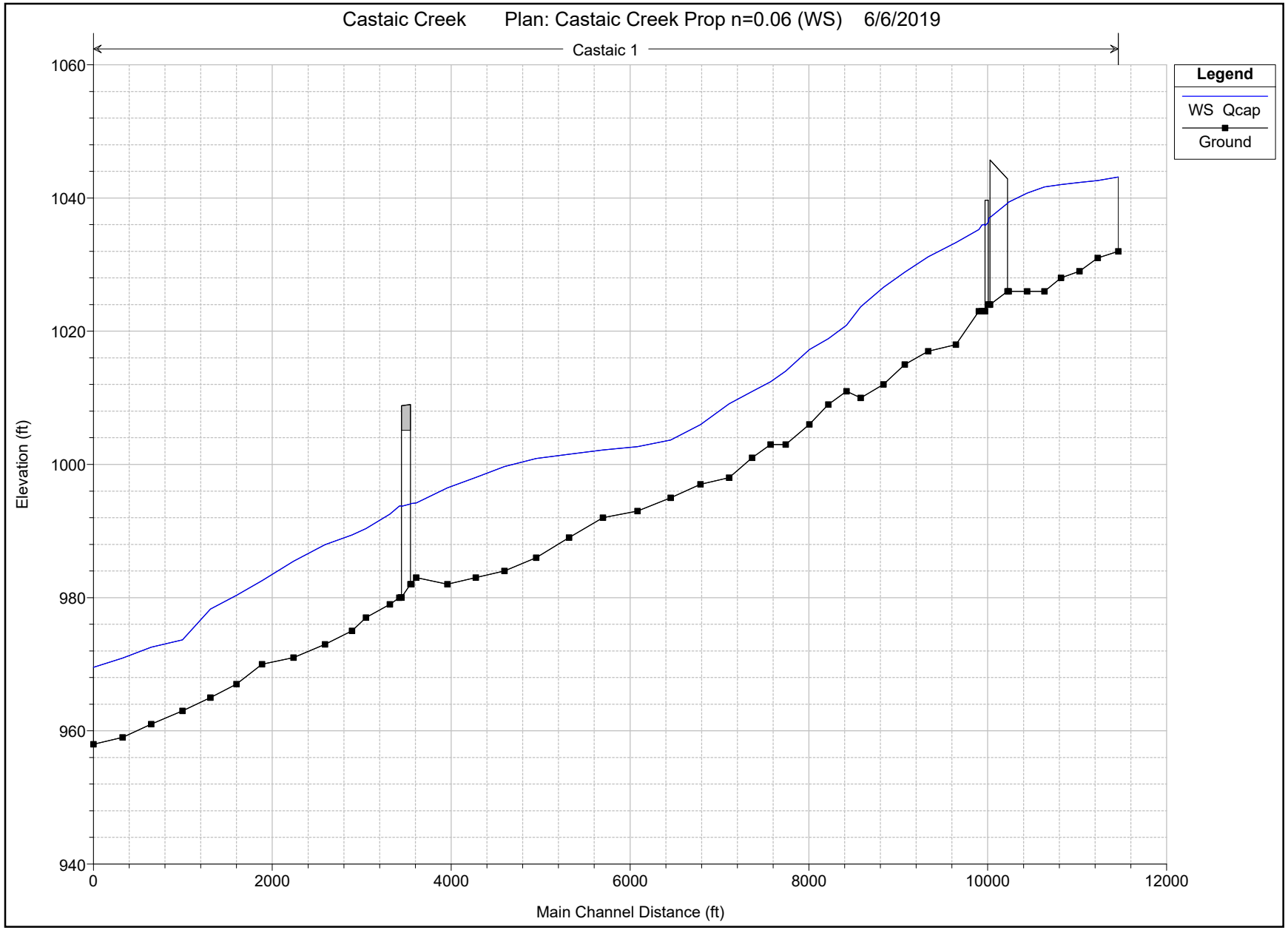
.06
.06
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta

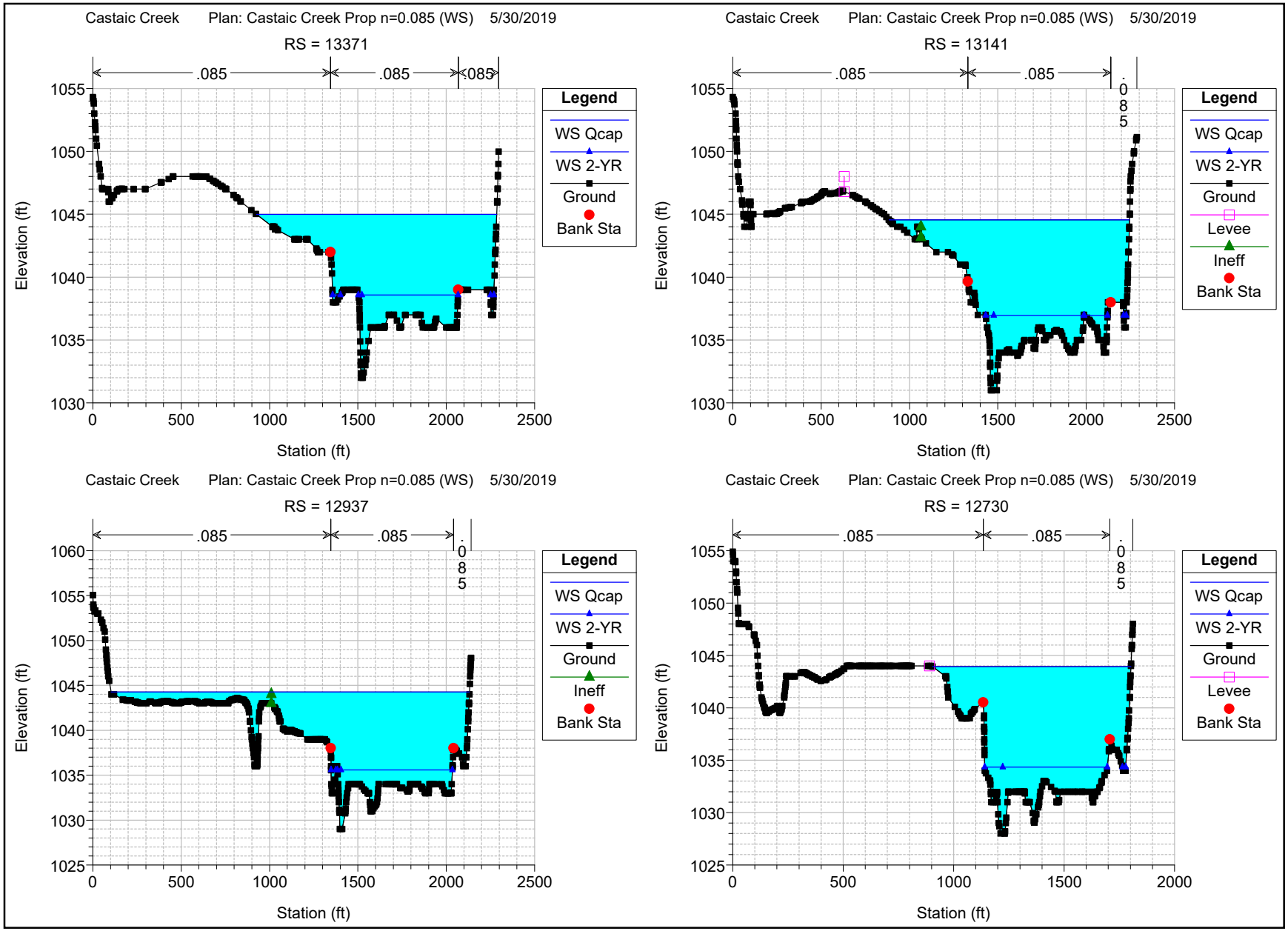

Castaic Creek Plan: Castaic Creek Prop n=0.06 (WS) 6/6/2019
Castaic 1
Legend
WS Qcap
Ground
Elevation (ft)
1060
1040
1020
1000
980
960
940
0
2000
4000
6000
8000
10000
12000
Main Channel Distance (ft)

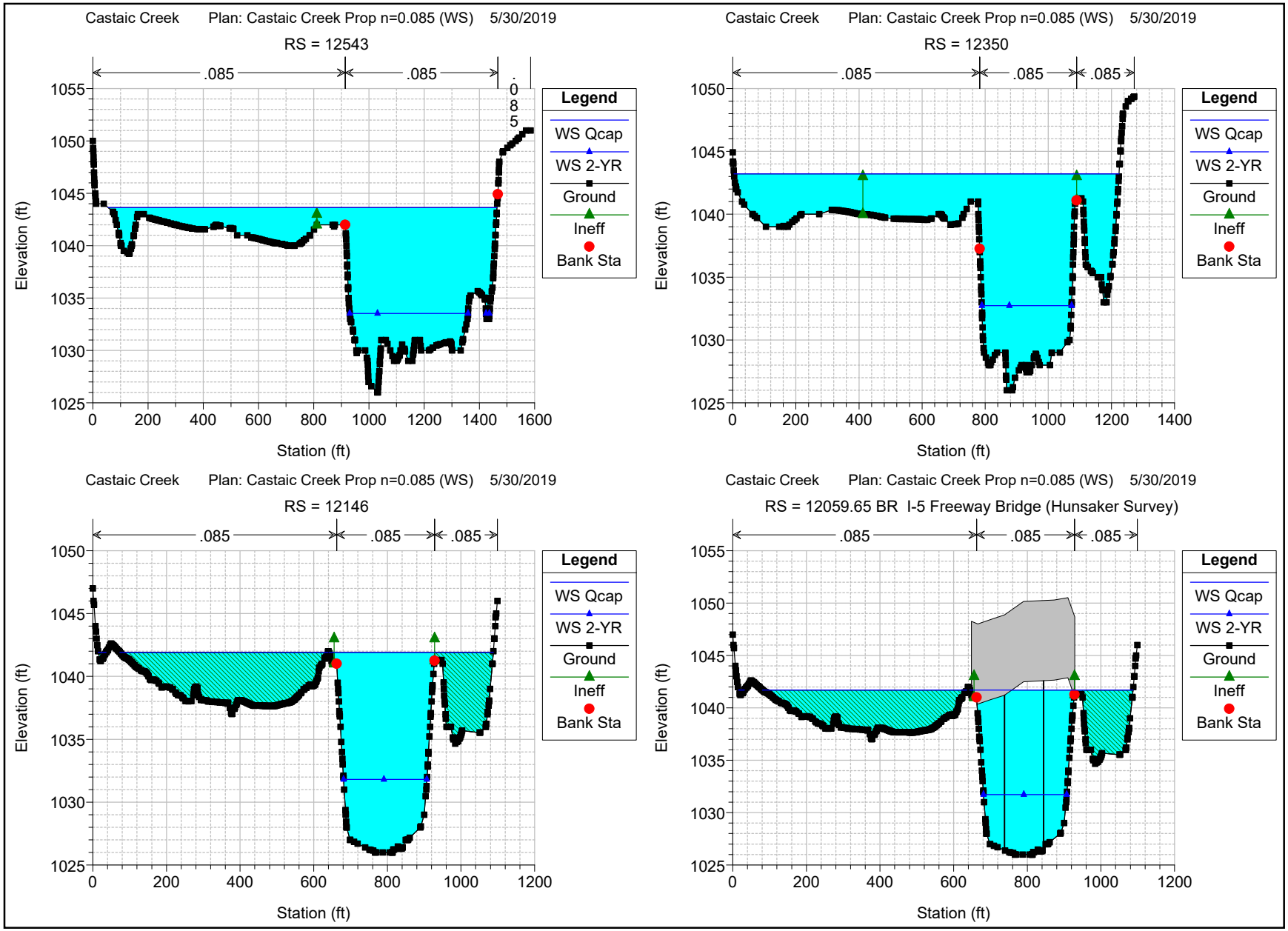
Appendix C
Proposed Condition
HEC-RAS Model Results
(n=0.085)

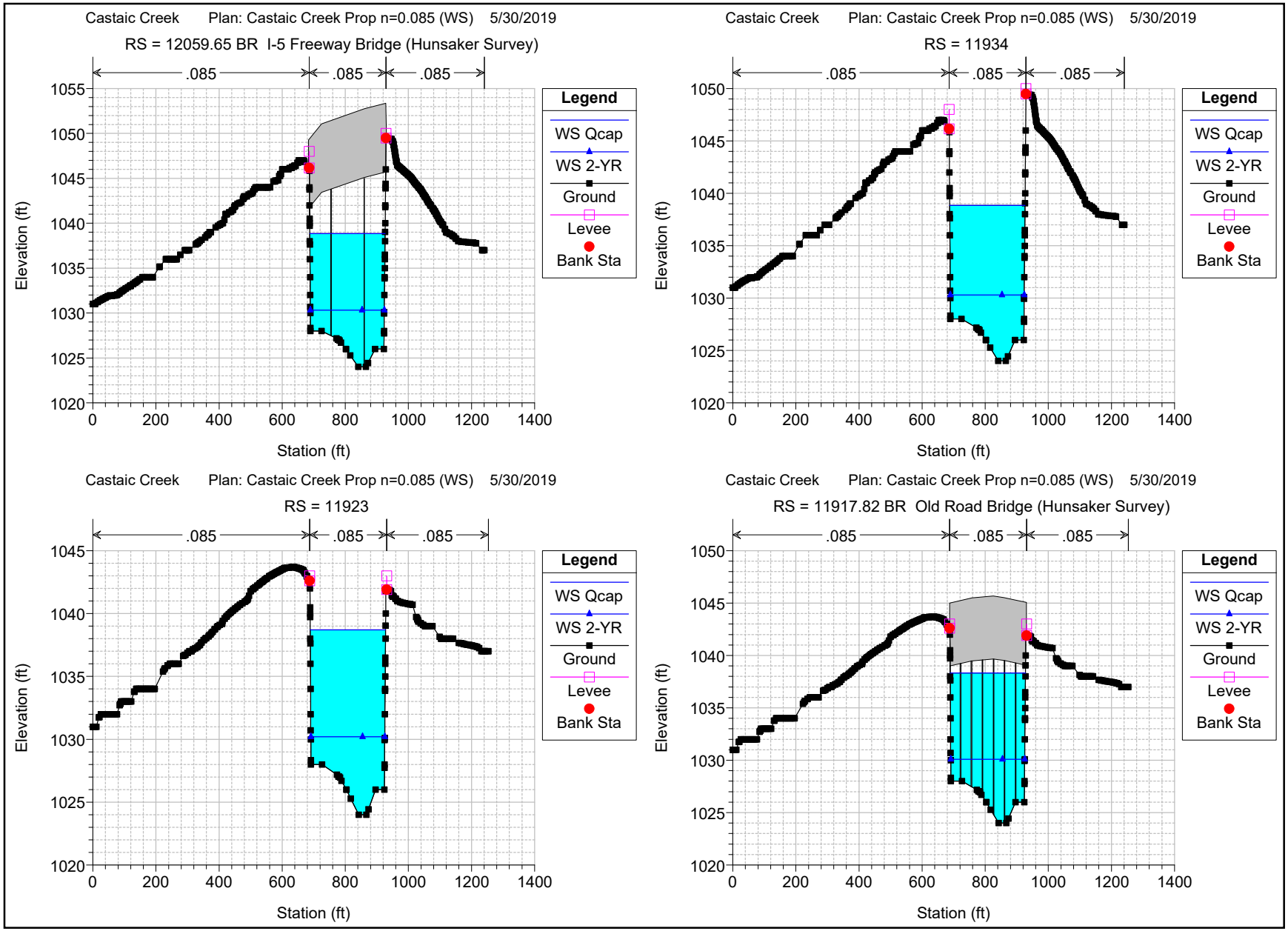
Appendix C – Proposed Condition
HEC-RAS Model Results (n=0.085)

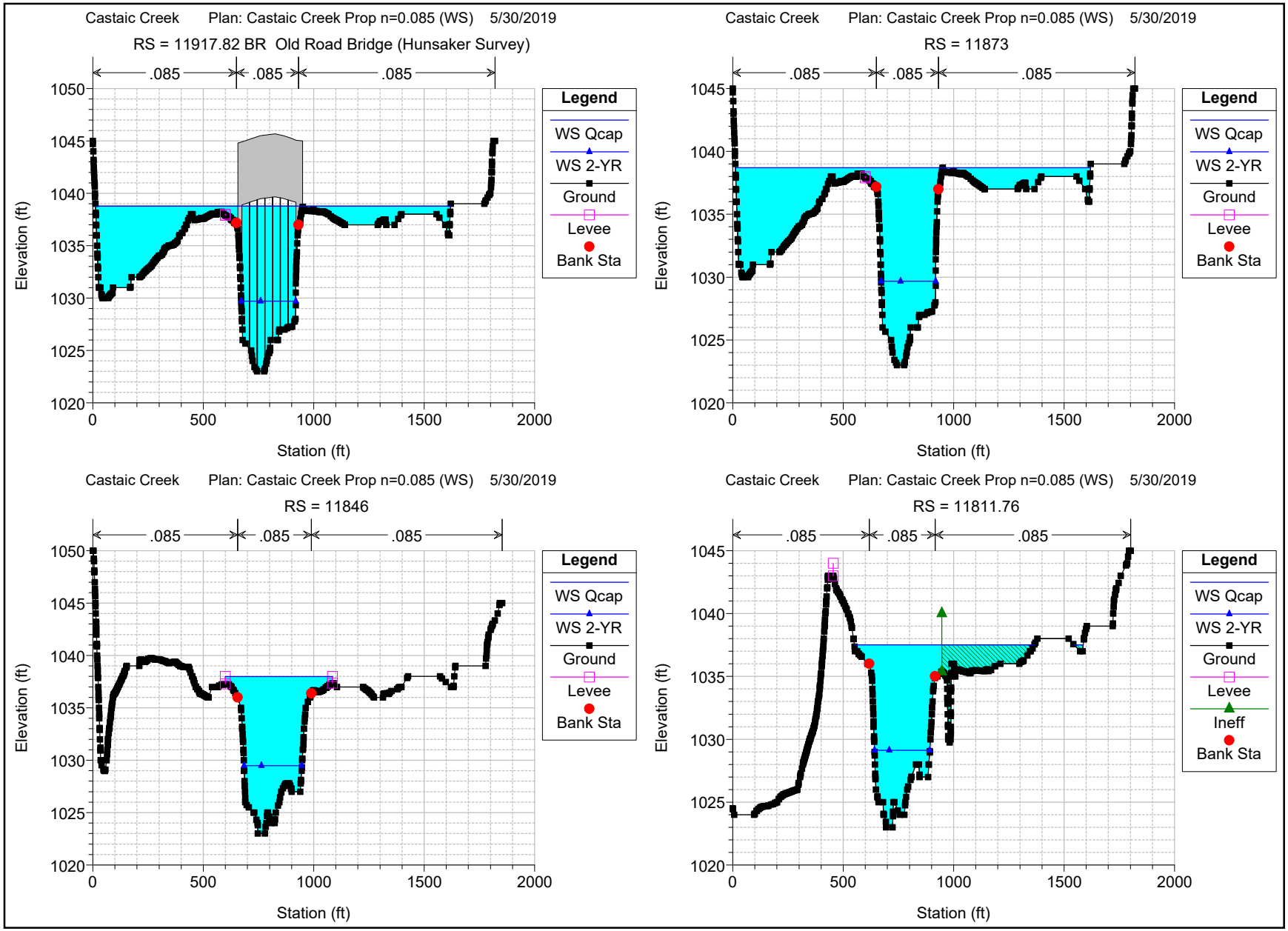
HEC-RAS Plan: Prop n=0.085 River: Castaic Reach: 1 Profile: Qcap

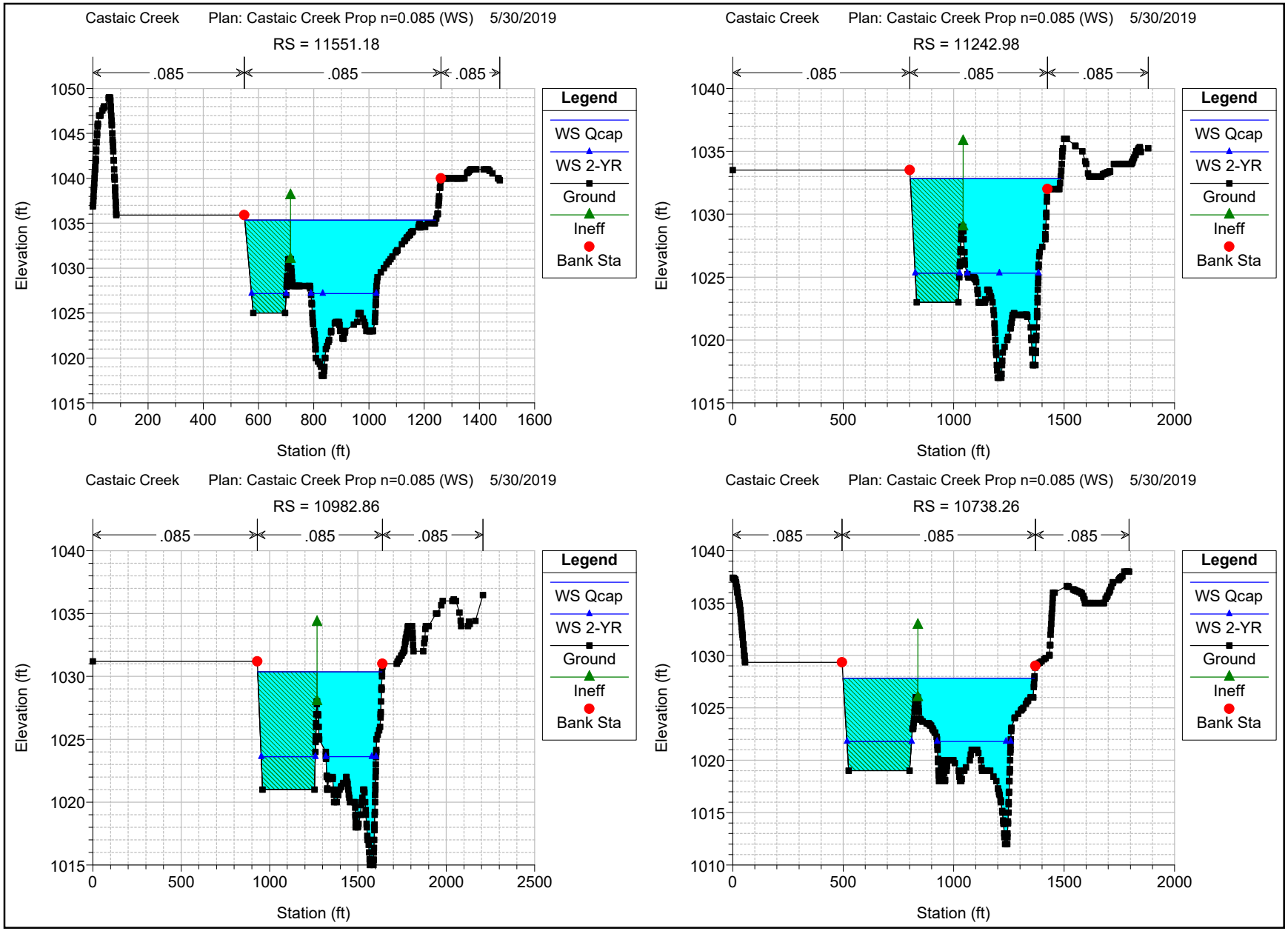
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	13371	Qcap	26500.00	1032.00	1044.99	1040.16	1045.18	0.002616	3.65	8001.71	1359.17	0.22
1	13141	Qcap	26500.00	1031.00	1044.55	1038.45	1044.69	0.001675	3.17	9088.48	1359.77	0.18
1	12937	Qcap	26500.00	1029.00	1044.27	1036.97	1044.39	0.001214	2.97	10901.34	2027.67	0.16
1	12730	Qcap	26500.00	1028.00	1043.94	1035.81	1044.11	0.001380	3.41	8398.79	897.76	0.17
1	12543	Qcap	26500.00	1026.00	1043.64	1034.90	1043.83	0.001489	3.57	8709.04	1408.99	0.18
1	12350	Qcap	26500.00	1026.00	1043.21	1034.86	1043.47	0.001987	4.50	7841.63	1220.05	0.21
1	12146	Qcap	26500.00	1026.00	1041.92	1034.52	1042.76	0.005625	7.35	3609.97	1032.92	0.35
1	12059.65		Bridge									
1	11934	Qcap	26500.00	1024.00	1038.85	1033.58	1040.08	0.009881	8.89	2979.32	238.92	0.44
1	11923	Qcap	26500.00	1024.00	1038.70	1033.59	1039.96	0.010263	9.00	2944.24	238.85	0.45
1	11917.82		Bridge									
1	11873	Qcap	26500.00	1023.00	1038.69	1032.64	1039.05	0.003642	5.46	6787.91	1607.15	0.28
1	11846	Qcap	26500.00	1023.00	1038.00	1032.49	1038.85	0.008055	7.47	3686.75	484.95	0.41
1	11811.76	Qcap	26500.00	1023.00	1037.51	1032.65	1038.54	0.009482	8.19	3333.26	855.98	0.44
1	11551.18	Qcap	26500.00	1018.00	1035.35	1030.29	1036.01	0.009252	6.53	4061.01	691.67	0.41
1	11242.98	Qcap	26500.00	1017.00	1032.83	1027.80	1033.54	0.006941	6.79	3942.53	680.69	0.37
1	10982.86	Qcap	26500.00	1015.00	1030.37	1026.41	1031.32	0.010434	7.78	3405.14	704.07	0.45
1	10738.26	Qcap	26500.00	1012.00	1027.82	1024.64	1028.59	0.012118	7.06	3752.18	866.91	0.47
1	10490.82	Qcap	26500.00	1010.00	1024.84	1021.97	1025.43	0.012281	6.16	4304.88	829.22	0.45
1	10351.28	Qcap	26500.00	1011.00	1022.45		1023.15	0.016966	6.70	3953.52	777.51	0.52
1	10141.82	Qcap	26500.00	1009.00	1019.99		1020.45	0.010013	5.44	4875.01	885.90	0.41
1	9931.16	Qcap	26500.00	1006.00	1018.10		1018.49	0.008347	5.01	5294.65	950.23	0.37
1	9670.57	Qcap	26500.00	1003.00	1015.06		1015.59	0.015137	5.79	4577.17	1029.76	0.48
1	9501.81	Qcap	26500.00	1003.00	1013.44		1013.79	0.007370	4.69	5654.55	1020.97	0.35
1	9298.99	Qcap	26500.00	1001.00	1011.93		1012.27	0.007354	4.66	5688.58	1033.99	0.35
1	9042.81	Qcap	26500.00	998.00	1009.90		1010.24	0.008494	4.67	5676.29	1147.02	0.37
1	8724.84	Qcap	26500.00	997.00	1006.97		1007.28	0.009979	4.53	5845.08	1392.25	0.39
1	8396.13	Qcap	26500.00	995.00	1005.31		1005.47	0.003302	3.21	8261.22	1444.39	0.24
1	8049.79	Qcap	26500.00	993.00	1004.55		1004.66	0.001504	2.64	10028.29	1300.12	0.17
1	7688.71	Qcap	26500.00	992.00	1004.08		1004.18	0.001028	2.57	10314.43	1045.56	0.14
1	7326.46	Qcap	26500.00	989.00	1003.48		1003.67	0.001793	3.48	7614.28	743.30	0.19
1	6966.68	Qcap	26500.00	986.00	1002.78		1003.00	0.001819	3.71	7136.35	636.91	0.20
1	6623.5	Qcap	26500.00	984.00	1001.64		1002.07	0.003908	5.25	5052.19	472.72	0.28
1	6304.52	Qcap	26500.00	983.00	999.94		1000.57	0.005453	6.38	4156.13	373.23	0.34
1	5992.38	Qcap	26500.00	982.00	998.34	992.70	998.79	0.005393	5.40	4909.35	822.85	0.32
1	5629.45	Qcap	26500.00	983.00	996.13	990.54	996.78	0.006084	6.50	4076.69	481.52	0.35
1	5557.68	Qcap	26500.00	982.00	995.96	989.28	996.46	0.004165	5.72	4647.14	515.27	0.30
1	5500		Bridge									
1	5453.98	Qcap	26500.00	980.01	995.44		995.78	0.003071	4.71	5635.09	528.64	0.25
1	5434.68	Qcap	26500.00	980.00	995.39		995.72	0.002773	4.58	5803.52	536.89	0.24
1	5340.04	Qcap	31400.00	979.00	994.46		995.20	0.007949	6.90	4549.46	483.47	0.40
1	5060.55	Qcap	31400.00	977.00	992.44		993.14	0.007373	6.72	4673.90	486.98	0.38
1	4899.94	Qcap	31400.00	975.00	991.47		992.10	0.005916	6.35	4941.68	474.66	0.35
1	4571.63	Qcap	31400.00	973.00	989.85		990.45	0.004906	6.24	5035.07	431.74	0.32
1	4220.87	Qcap	31400.00	971.00	987.39		988.24	0.008262	7.37	4258.93	419.64	0.41
1	3865.39	Qcap	31400.00	970.00	984.41		985.25	0.008634	7.33	4281.05	440.37	0.41
1	3579.81	Qcap	31400.00	967.00	982.05		982.84	0.008152	7.14	4397.18	455.75	0.40
1	3289.64	Qcap	31400.00	965.00	979.53		980.36	0.008993	7.30	4303.75	459.39	0.42
1	2974.88	Qcap	31400.00	963.00	975.66		976.67	0.015694	8.05	3902.26	548.09	0.53
1	2627.11	Qcap	31400.00	961.00	974.25		974.49	0.002848	3.94	8004.64	954.48	0.24
1	2307.93	Qcap	31400.00	959.00	972.91		973.29	0.004986	4.95	6349.38	786.55	0.31
1	1981.79	Qcap	31400.00	958.00	971.62	965.20	971.97	0.003316	4.78	6632.52	733.22	0.26

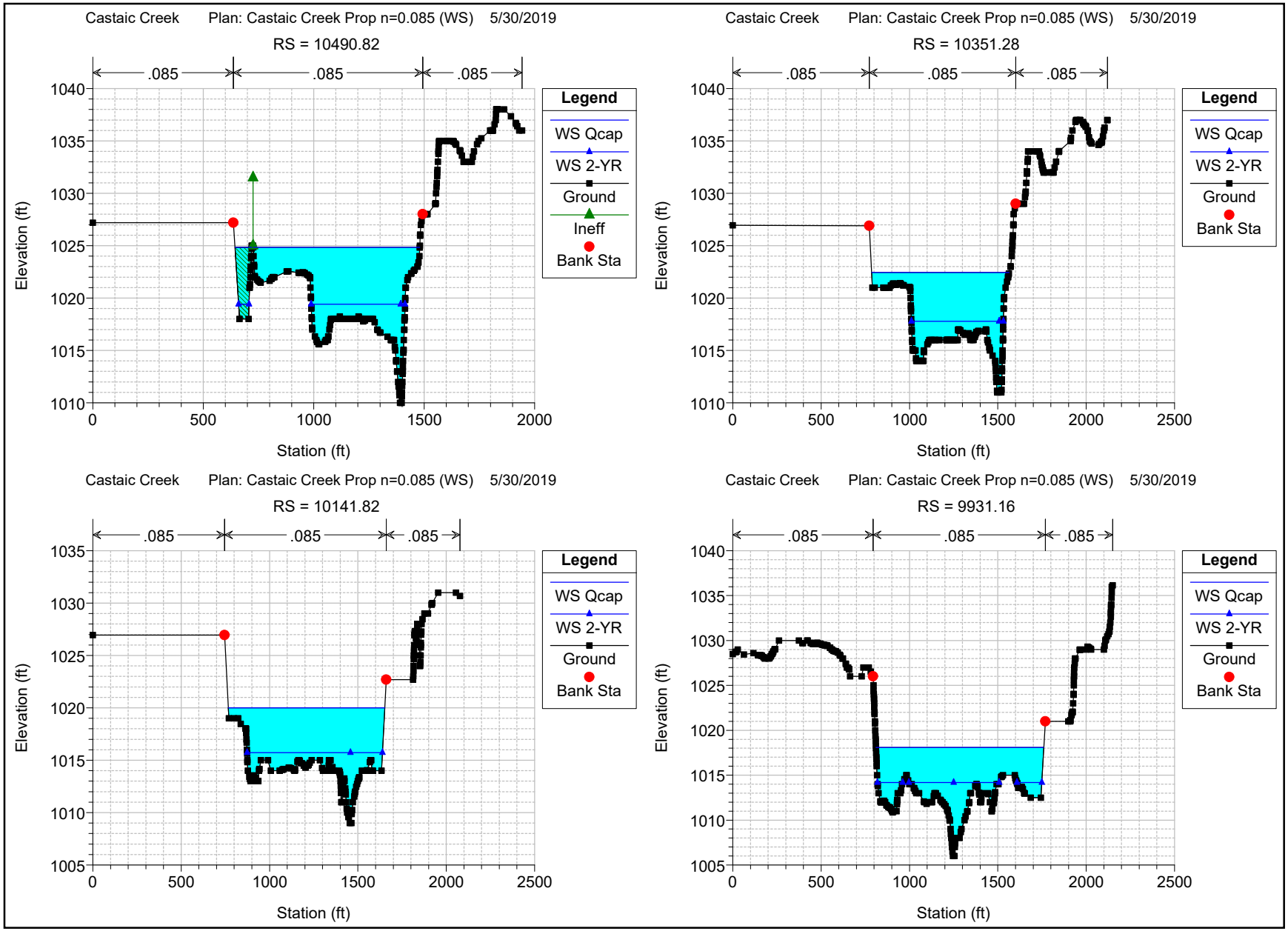

Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 13371
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 13141
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Levee
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 12937
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 12730
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Levee
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 12543
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 12350
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 12146
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 12059.65 BR I-5 Freeway Bridge (Hunsaker Survey)
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 12059.65 BR I-5 Freeway Bridge (Hunsaker Survey)
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Levee
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 11934
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 11923
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 11917.82 BR Old Road Bridge (Hunsaker Survey)


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 11917.82 BR Old Road Bridge (Hunsaker Survey)
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Levee
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 11873
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 11846
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 11811.76
Ineff


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 11551.18
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 11242.98
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 10982.86
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 10738.26


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 10490.82
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 10351.28
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 10141.82
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 9931.16
.085
.085
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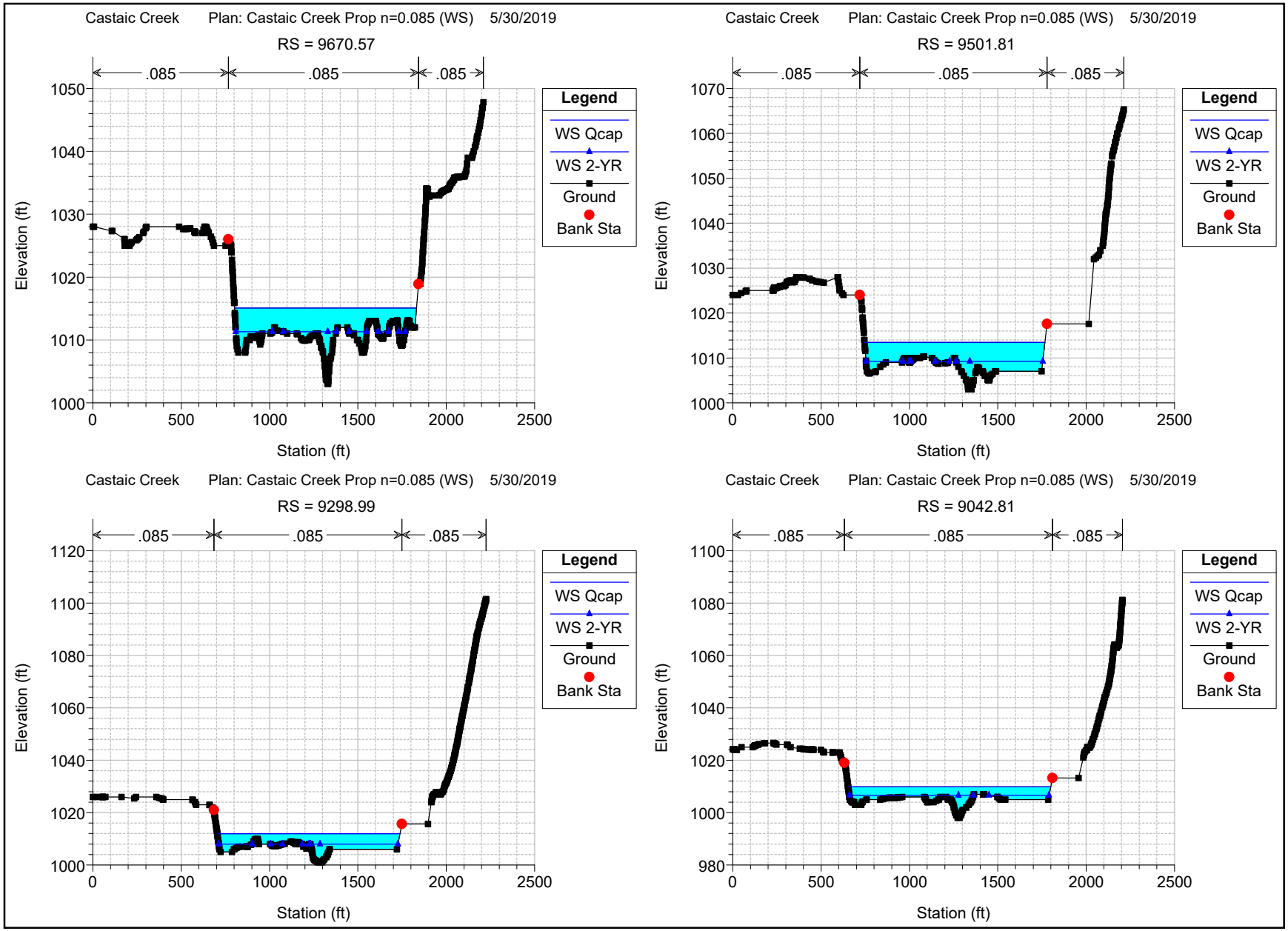
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 9670.57
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 9501.81
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 9298.99
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 9042.81
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.085
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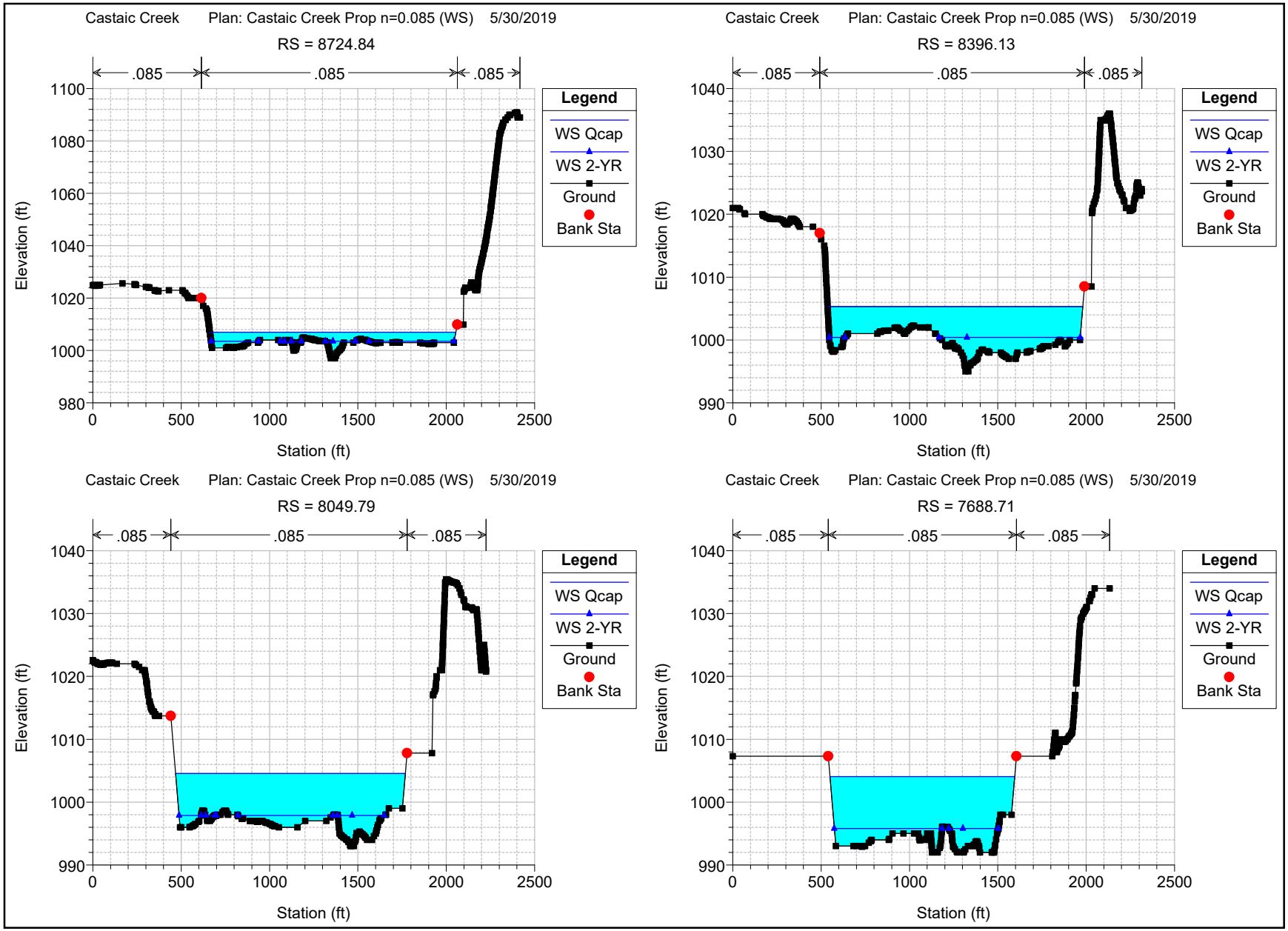
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta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RS = 8724.84
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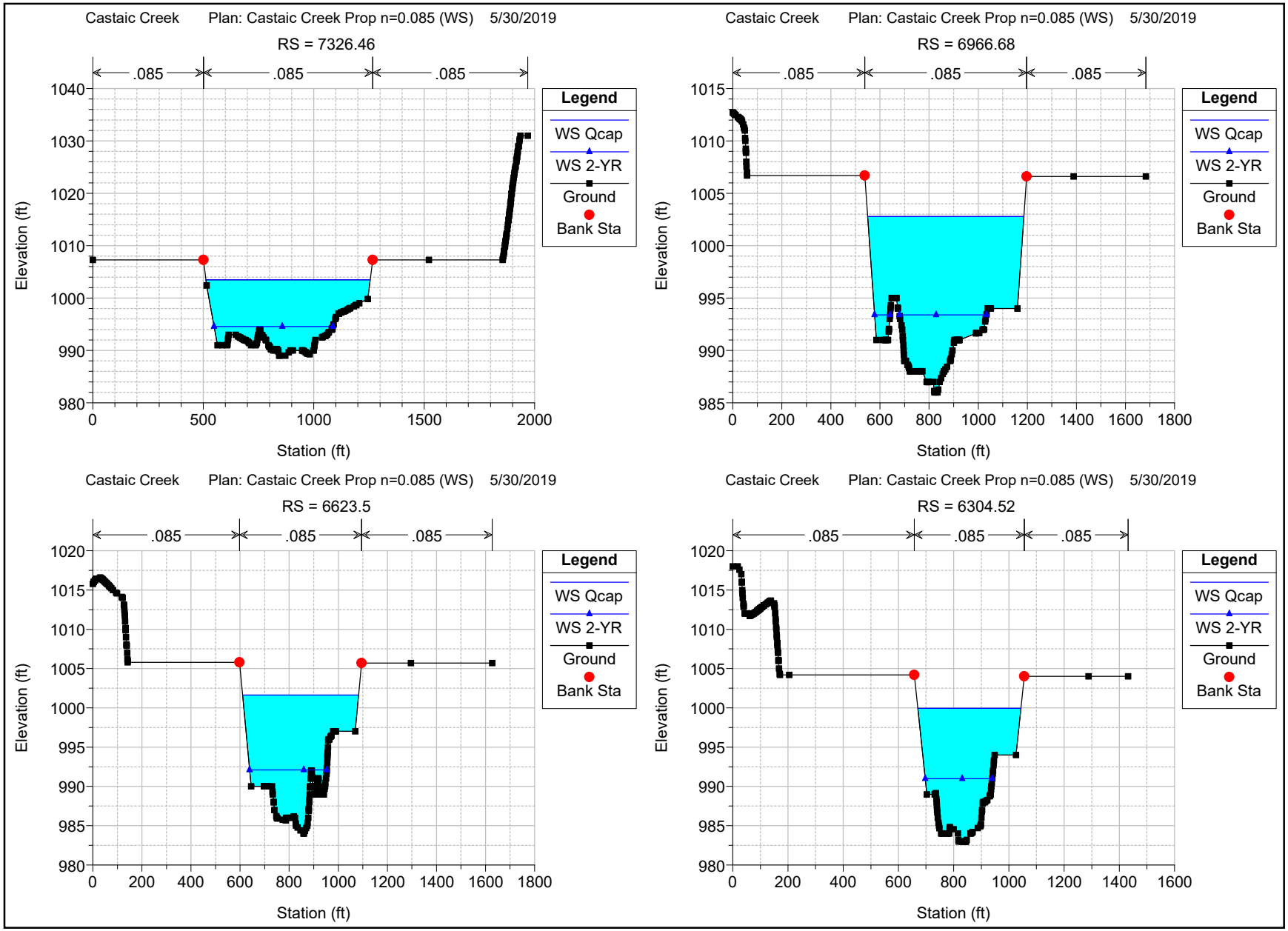
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 8396.13
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 8049.79
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 7688.71
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta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RS = 7326.46
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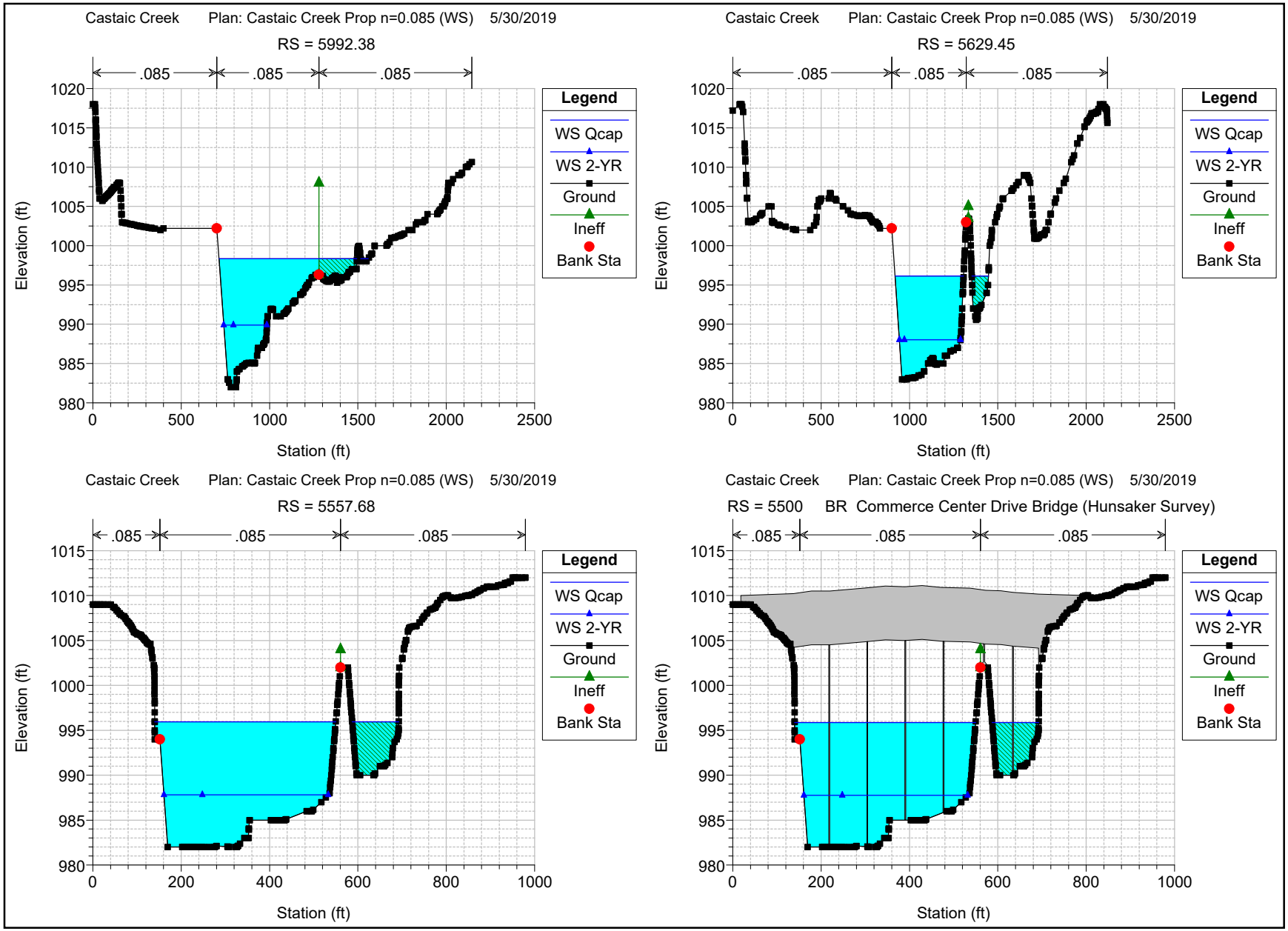
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 6966.68
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 6623.5
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 6304.52


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 5992.38
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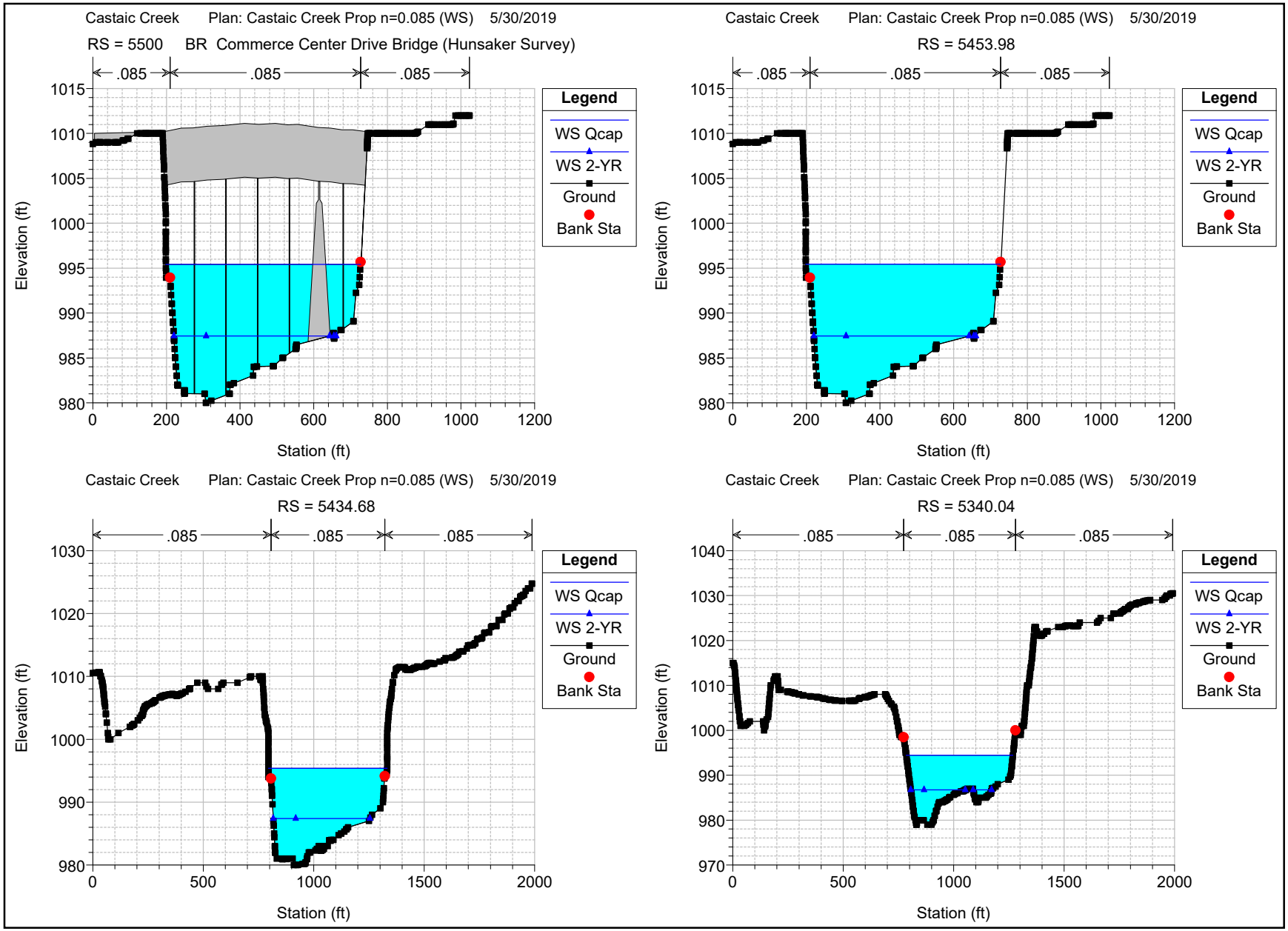
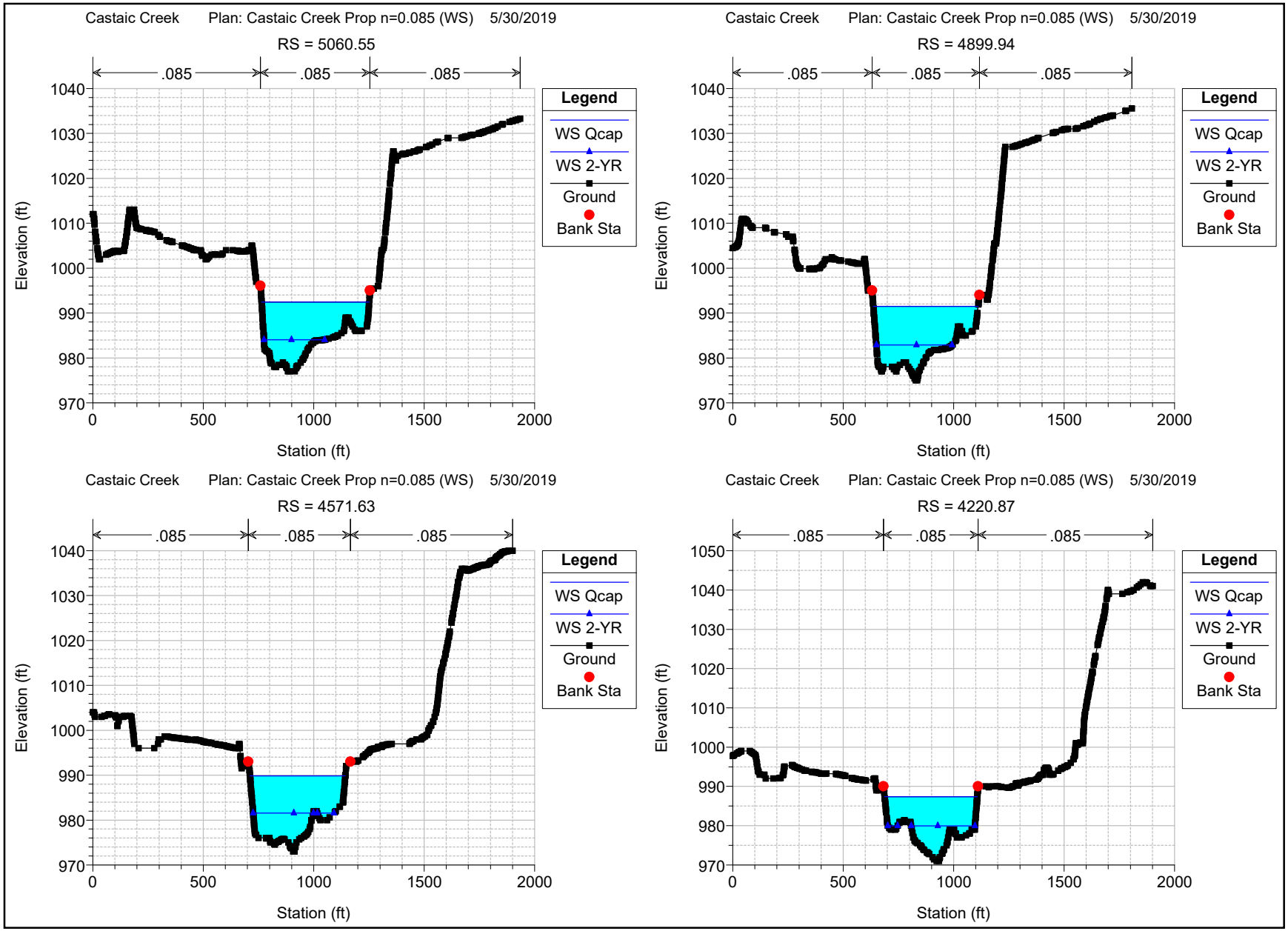
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 5629.45
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 5557.68
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 5500 BR Commerce Center Drive Bridge (Hunsaker Survey)
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Ineff
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 5500 BR Commerce Center Drive Bridge (Hunsaker Survey)
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 5453.98
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 5434.68
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 5340.04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RS = 5060.55
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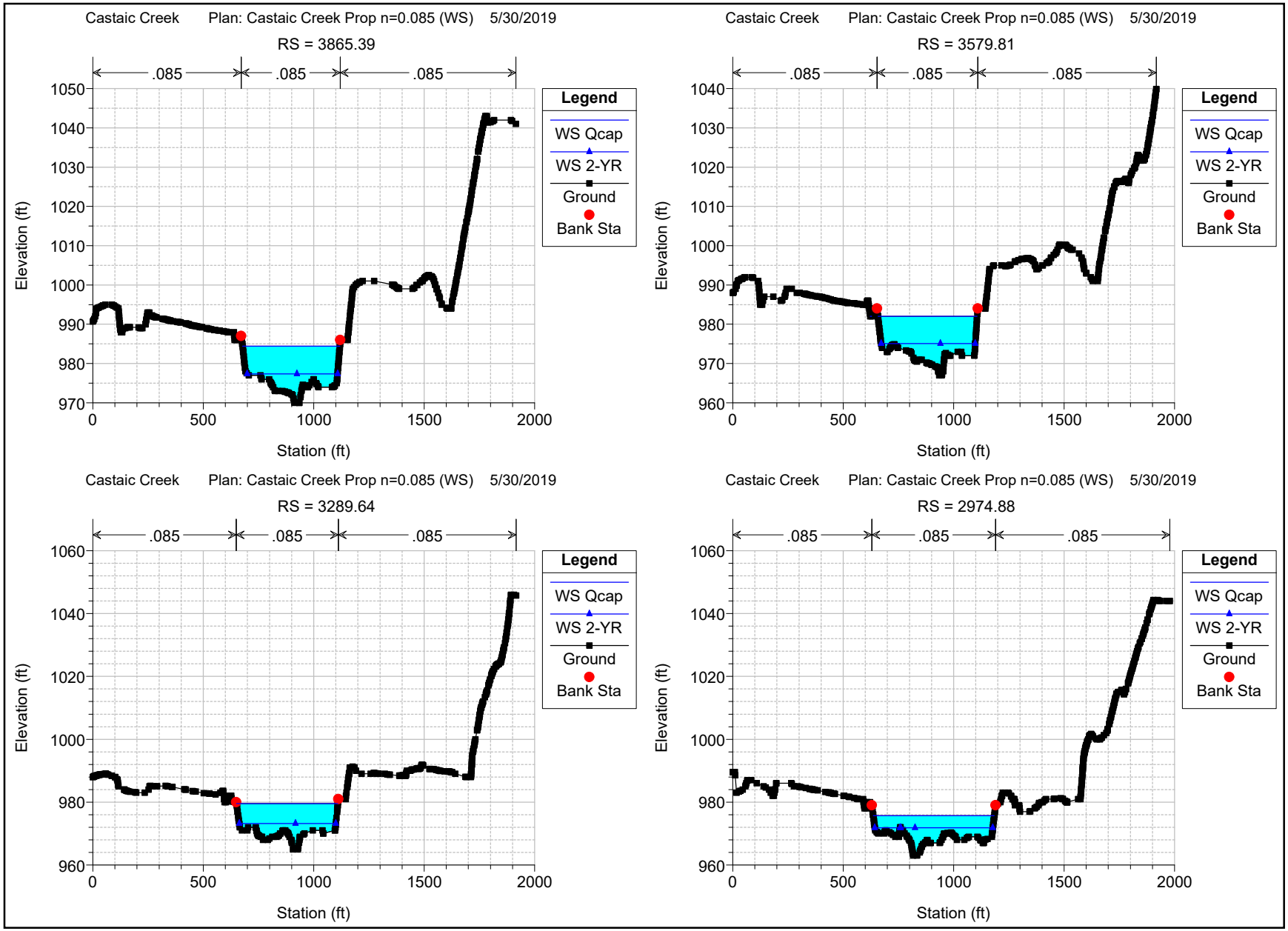
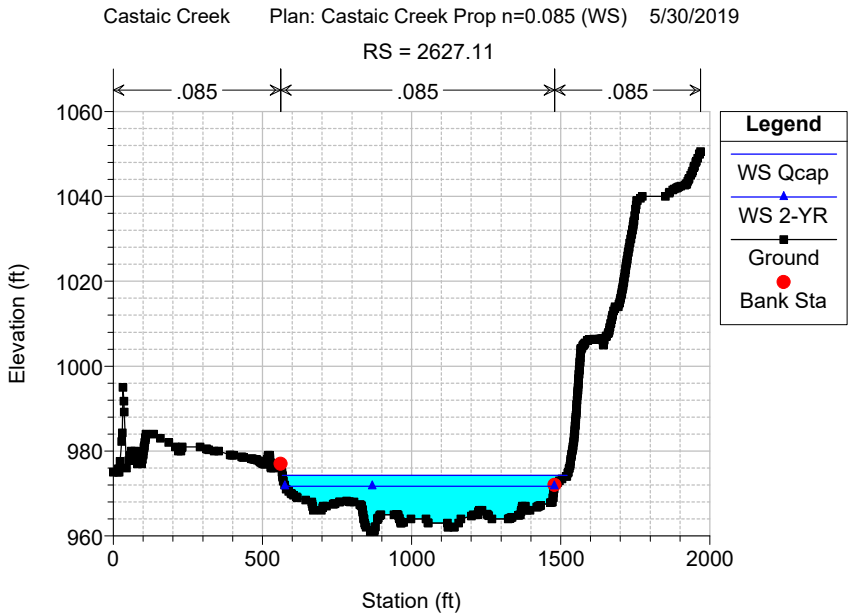
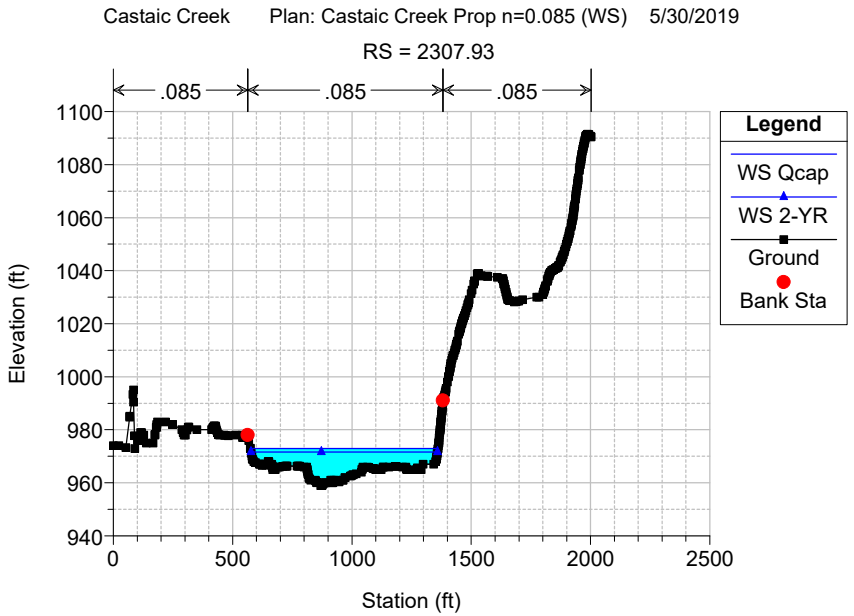
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 4899.94
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 4571.63
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 4220.87


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 3865.39
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 3579.81
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 3289.64
Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 2974.88


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 2627.11
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 2307.93
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.085
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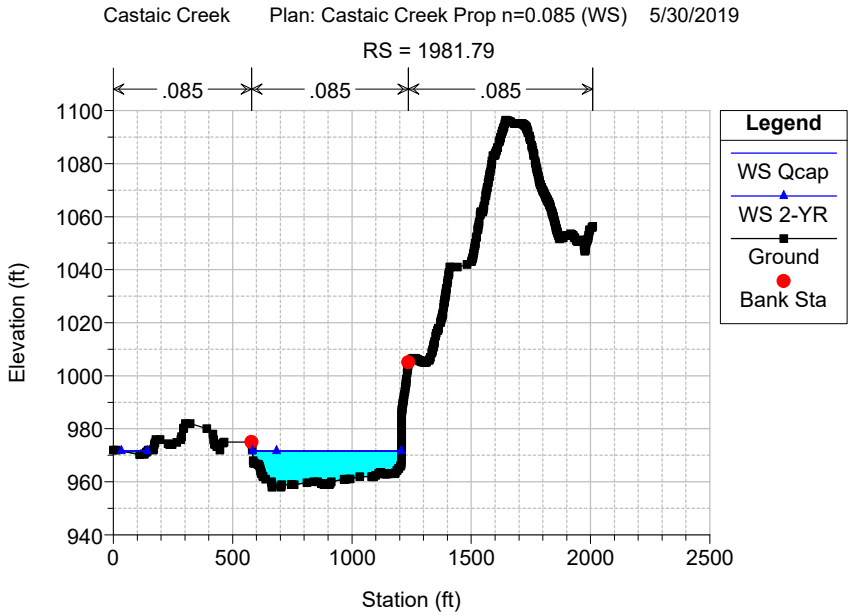
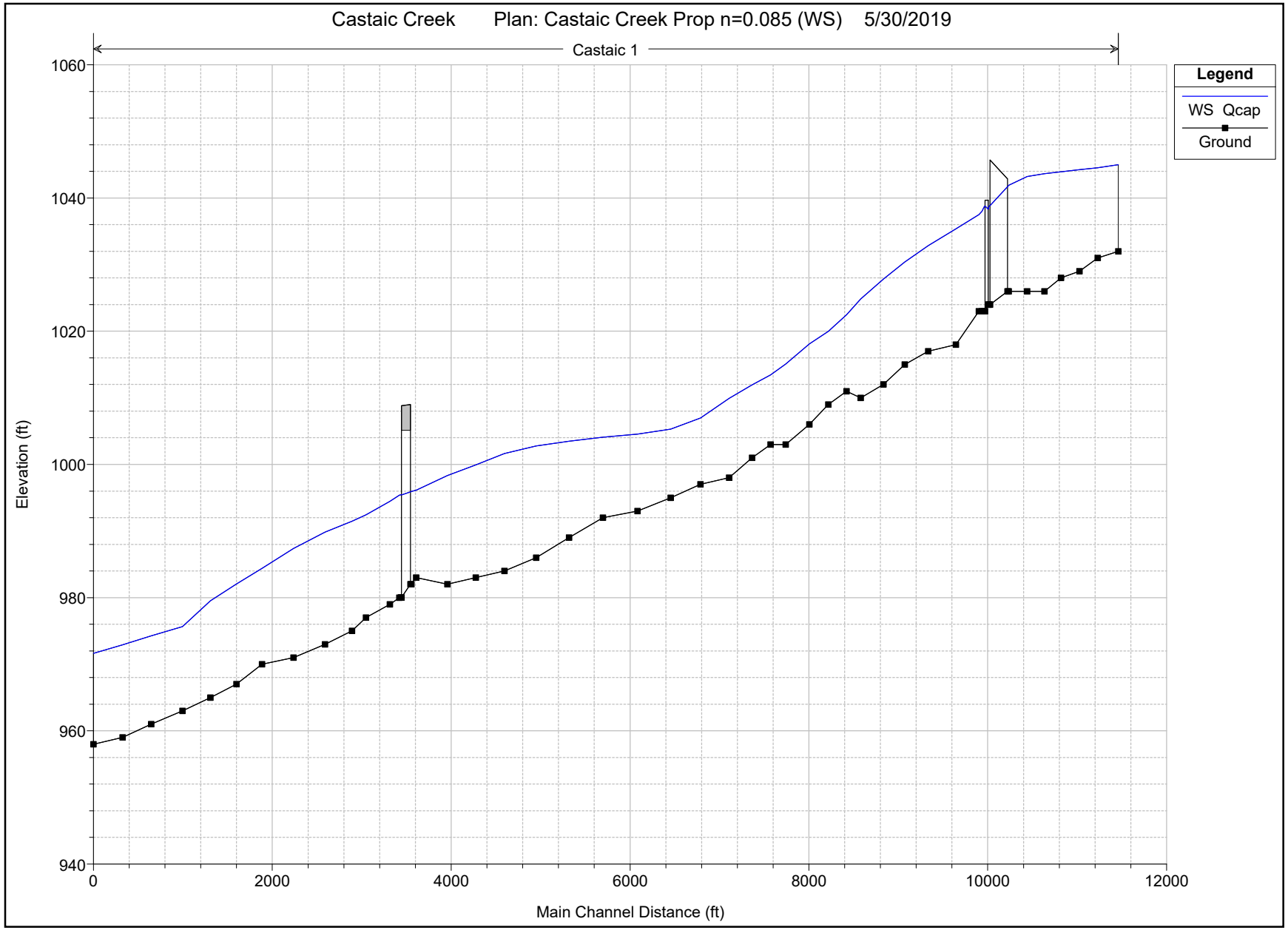
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta

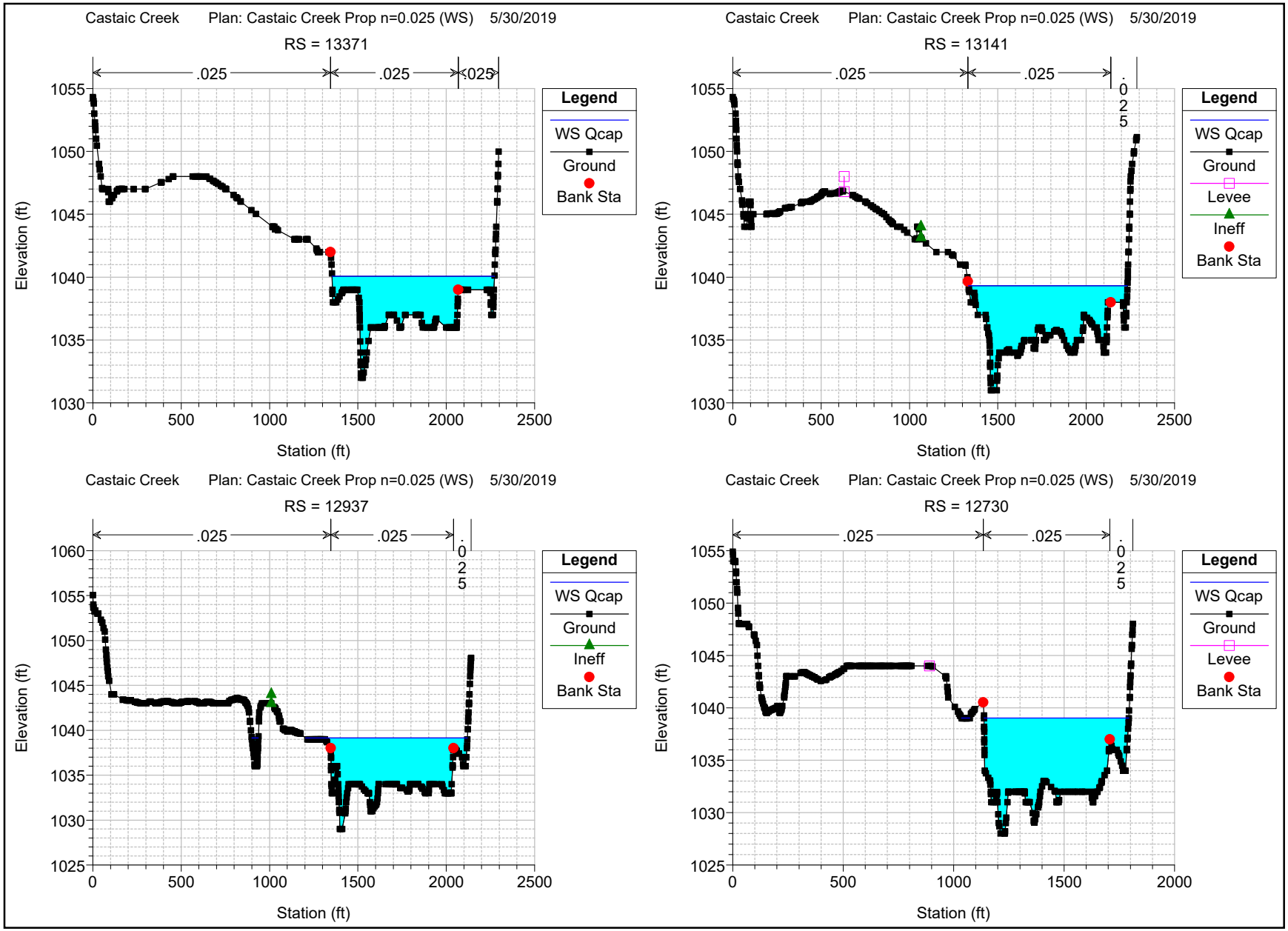

Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
RS = 1981.79
.085
.085
.085
Elevation (ft)
Station (ft)
Legend
WS Qcap
WS 2-YR
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.085 (WS) 5/30/2019
Castaic 1
Legend
WS Qcap
Ground
Elevation (ft)
1060
1040
1020
1000
980
960
940
0
2000
4000
6000
8000
10000
12000
Main Channel Distance (ft)

Appendix D – Proposed Condition HEC-RAS Model Results (n=0.025)

HEC-RAS Plan: Prop n=0.025 River: Castaic Reach: 1 Profile: Qcap

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)	
1	13371	Qcap	26500.00	1032.00	1040.07	1040.16	1041.68	0.006001	10.38	2676.52	919.48	0.99
1	13141	Qcap	26500.00	1031.00	1039.31	1038.45	1040.24	0.002624	7.80	3467.15	899.94	0.68
1	12937	Qcap	26500.00	1029.00	1039.16	1036.97	1039.81	0.001175	6.50	4187.48	953.47	0.48
1	12730	Qcap	26500.00	1028.00	1039.02	1035.81	1039.59	0.000772	6.15	4425.56	692.95	0.40
1	12543	Qcap	26500.00	1026.00	1038.82	1034.90	1039.45	0.000738	6.33	4186.01	533.51	0.40
1	12350	Qcap	26500.00	1026.00	1037.63	1034.86	1039.15	0.001539	9.92	2671.70	391.47	0.58
1	12146	Qcap	26500.00	1026.00	1036.45	1034.52	1038.71	0.002248	12.04	2200.36	355.71	0.71
1	12059.65		Bridge									
1	11934	Qcap	26500.00	1024.00	1035.79	1033.58	1037.95	0.002103	11.77	2251.63	237.27	0.67
1	11923	Qcap	26500.00	1024.00	1035.75	1033.59	1037.92	0.002138	11.83	2240.16	237.24	0.68
1	11917.82		Bridge									
1	11873	Qcap	26500.00	1023.00	1030.91	1032.64	1037.04	0.012124	19.85	1334.81	248.62	1.51
1	11846	Qcap	26500.00	1023.00	1030.91	1032.52	1036.58	0.011734	19.10	1387.67	268.84	1.48
1	11811.76	Qcap	26500.00	1023.00	1032.65	1032.65	1036.00	0.004894	14.68	1804.97	283.92	1.00
1	11551.18	Qcap	26500.00	1018.00	1028.70	1030.29	1033.91	0.012282	18.31	1447.16	441.18	1.49
1	11242.98	Qcap	26500.00	1017.00	1027.33	1027.80	1030.58	0.006688	14.48	1829.89	563.23	1.12
1	10982.86	Qcap	26500.00	1015.00	1026.41	1026.41	1029.22	0.005193	13.43	1973.40	669.81	1.00
1	10738.26	Qcap	26500.00	1012.00	1023.61	1024.64	1027.42	0.010356	15.67	1691.10	707.67	1.34
1	10490.82	Qcap	26500.00	1010.00	1020.69	1021.97	1024.59	0.011812	15.86	1671.37	482.51	1.42
1	10351.28	Qcap	26500.00	1011.00	1018.84	1019.83	1022.50	0.013805	15.36	1724.72	524.44	1.49
1	10141.82	Qcap	26500.00	1009.00	1016.39	1017.17	1019.28	0.015428	13.64	1942.44	770.26	1.51
1	9931.16	Qcap	26500.00	1006.00	1015.33	1015.37	1016.85	0.006817	9.88	2682.53	936.08	1.03
1	9670.57	Qcap	26500.00	1003.00	1013.08	1013.26	1014.77	0.009111	10.43	2541.60	1014.36	1.16
1	9501.81	Qcap	26500.00	1003.00	1009.75	1010.60	1012.58	0.016922	13.50	1963.48	849.16	1.56
1	9298.99	Qcap	26500.00	1001.00	1009.06	1009.06	1010.49	0.006724	9.60	2760.74	995.65	1.02
1	9042.81	Qcap	26500.00	998.00	1007.40	1007.46	1008.77	0.007470	9.41	2817.62	1134.57	1.05
1	8724.84	Qcap	26500.00	997.00	1004.83	1004.98	1006.14	0.008892	9.18	2887.04	1373.28	1.12
1	8396.13	Qcap	26500.00	995.00	1001.20	1001.59	1002.90	0.010467	10.44	2538.96	1127.64	1.23
1	8049.79	Qcap	26500.00	993.00	999.07	999.07	1000.29	0.007030	8.84	2997.74	1267.26	1.01
1	7688.71	Qcap	26500.00	992.00	998.71	996.57	999.19	0.001093	5.53	4795.87	1013.40	0.45
1	7326.46	Qcap	26500.00	989.00	997.85		998.68	0.001449	7.32	3621.18	620.32	0.53
1	6966.68	Qcap	26500.00	986.00	997.42		998.17	0.001188	6.96	3808.33	604.74	0.49
1	6623.5	Qcap	26500.00	984.00	995.77		997.52	0.002193	10.59	2502.94	331.45	0.68
1	6304.52	Qcap	26500.00	983.00	993.92		996.58	0.003188	13.09	2025.10	259.59	0.83
1	5992.38	Qcap	26500.00	982.00	992.70	992.70	995.27	0.005300	12.85	2062.88	400.65	1.00
1	5629.45	Qcap	26500.00	983.00	991.78	990.54	993.61	0.002693	10.87	2438.06	390.63	0.74
1	5557.68	Qcap	26500.00	982.00	992.19	989.28	993.29	0.001268	8.44	3138.77	473.51	0.52
1	5500		Bridge									
1	5453.98	Qcap	26500.00	980.01	992.29		992.97	0.000792	6.62	4002.53	503.20	0.41
1	5434.68	Qcap	26500.00	980.00	992.32		992.94	0.000682	6.31	4199.90	508.29	0.39
1	5340.04	Qcap	31400.00	979.00	989.91	989.91	992.57	0.005377	13.07	2403.28	460.02	1.01
1	5060.55	Qcap	31400.00	977.00	986.67	987.61	990.60	0.009117	15.90	1974.48	418.29	1.29
1	4899.94	Qcap	31400.00	975.00	984.45	985.74	989.06	0.009919	17.22	1823.34	364.66	1.36
1	4571.63	Qcap	31400.00	973.00	985.12	983.34	986.79	0.002171	10.36	3031.04	415.16	0.68
1	4220.87	Qcap	31400.00	971.00	982.64	982.64	985.54	0.005249	13.65	2301.19	404.22	1.01
1	3865.39	Qcap	31400.00	970.00	979.04	979.93	983.00	0.009337	15.96	1967.44	421.87	1.30
1	3579.81	Qcap	31400.00	967.00	976.72	977.44	980.37	0.008465	15.33	2048.18	432.05	1.24
1	3289.64	Qcap	31400.00	965.00	974.89	975.26	978.03	0.006806	14.22	2208.63	443.16	1.12
1	2974.88	Qcap	31400.00	963.00	972.63	973.16	975.64	0.008202	13.93	2254.20	537.08	1.20
1	2627.11	Qcap	31400.00	961.00	970.54	968.64	971.26	0.001488	6.83	4595.08	889.12	0.53
1	2307.93	Qcap	31400.00	959.00	968.42	968.42	970.30	0.005947	10.99	2856.14	765.71	1.00
1	1981.79	Qcap	31400.00	958.00	966.28	965.20	967.74	0.002774	9.68	3243.72	594.61	0.73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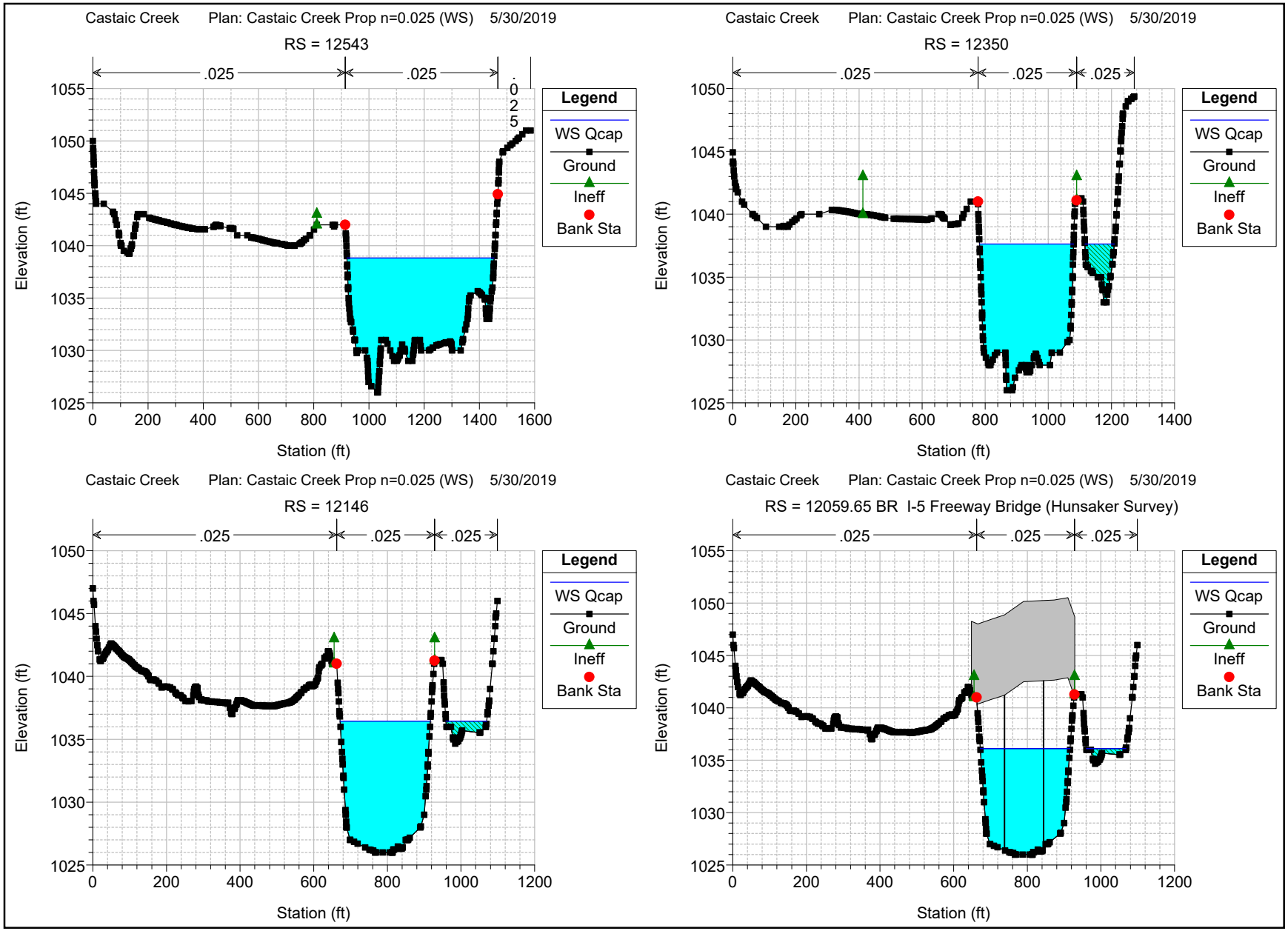
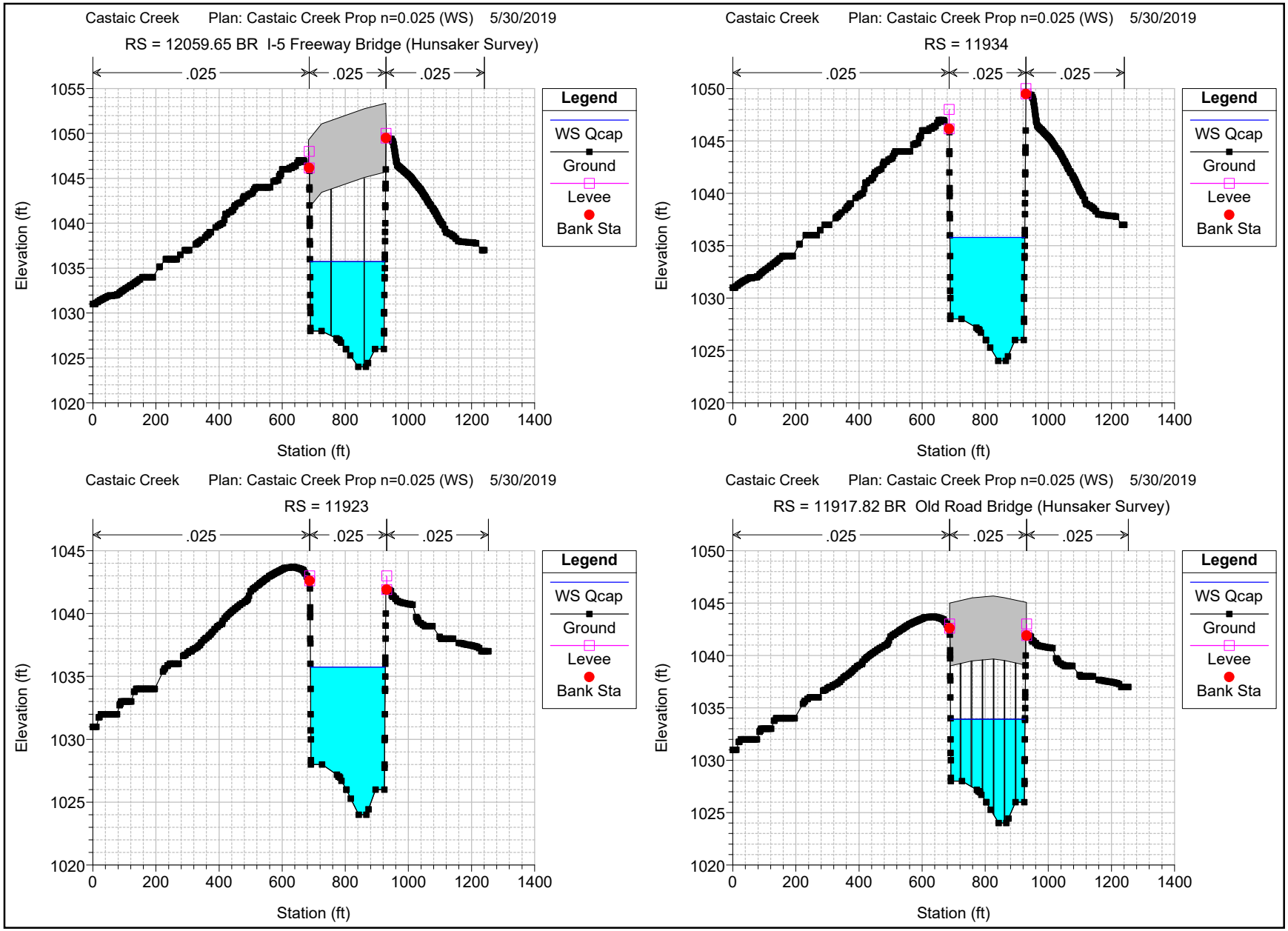
RS = 13371
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 13141
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Levee
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 12937
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 12730
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Levee
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 12543
.025
.025
.025
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 12350
.025
.025
.025
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 12146
.025
.025
.025
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 12059.65 BR I-5 Freeway Bridge (Hunsaker Survey)
.025
.025
.025
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)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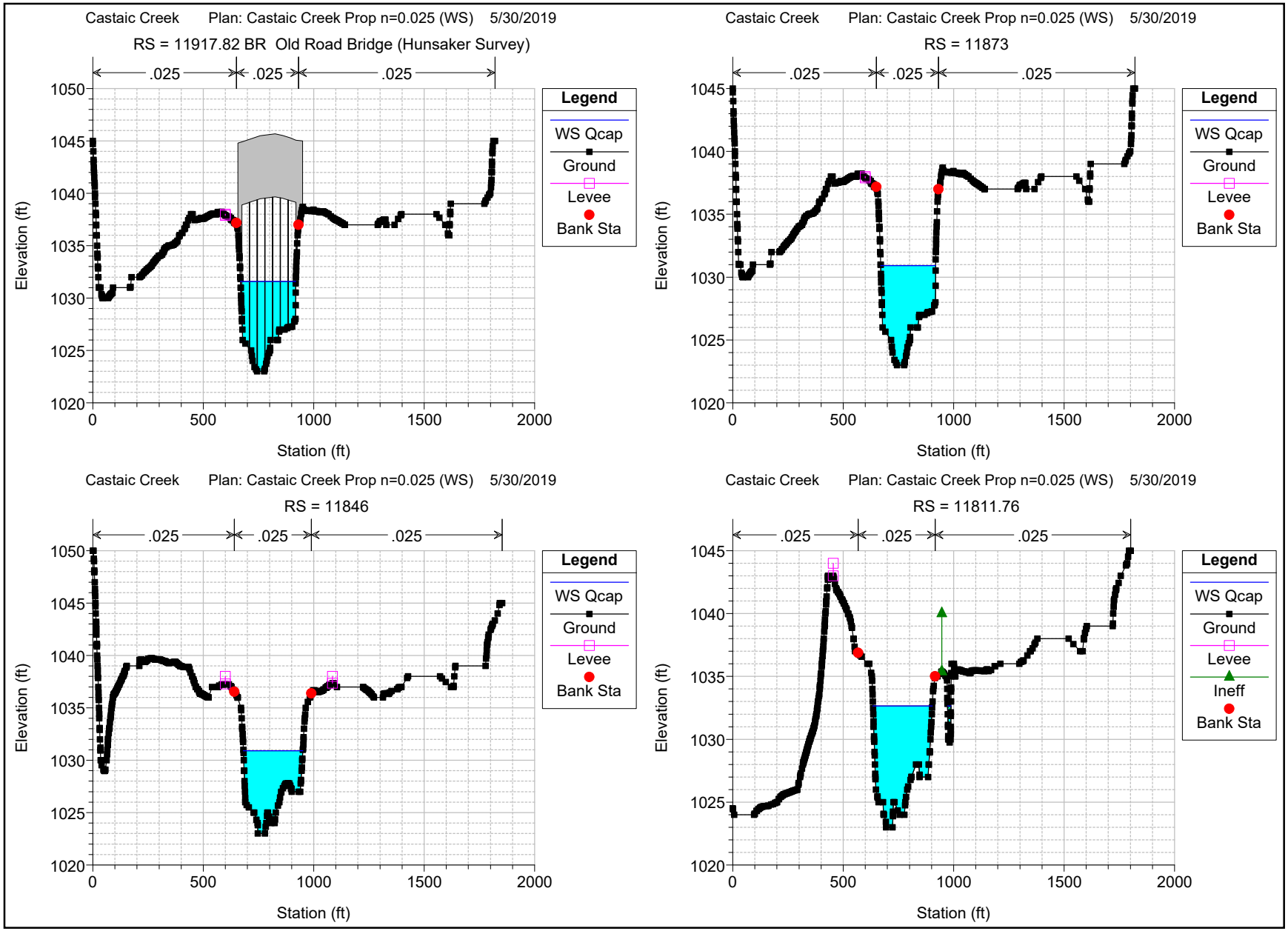
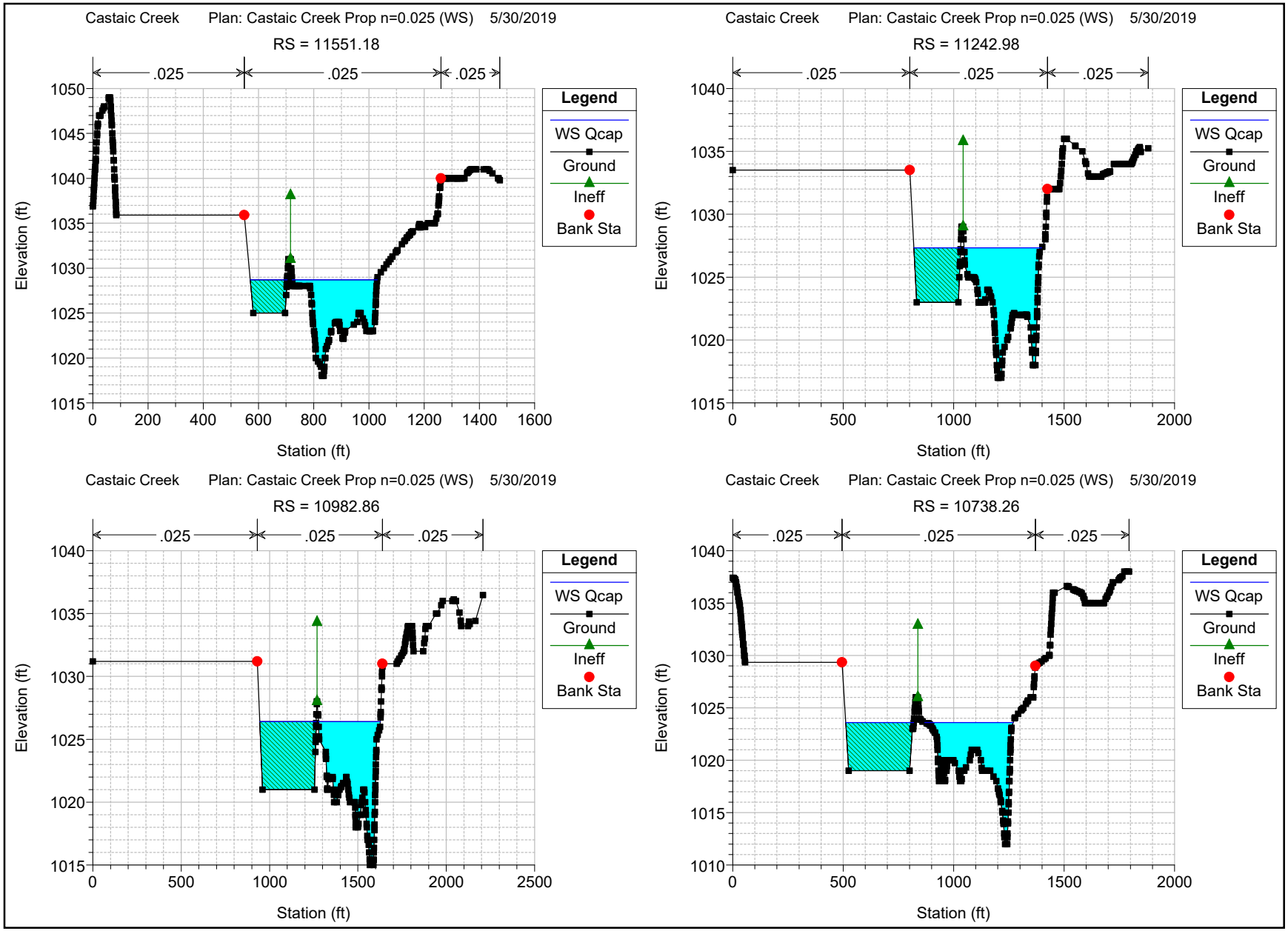
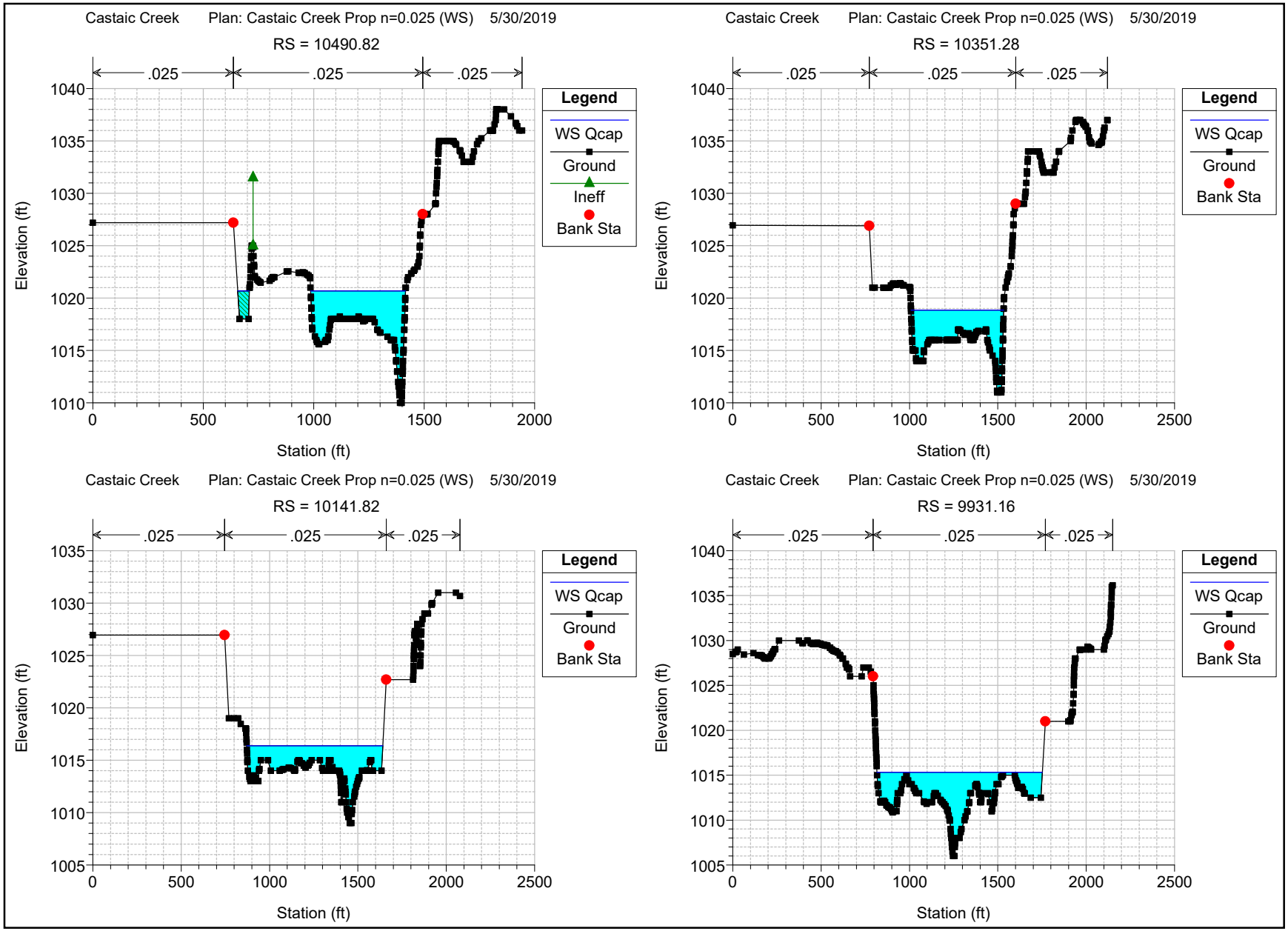
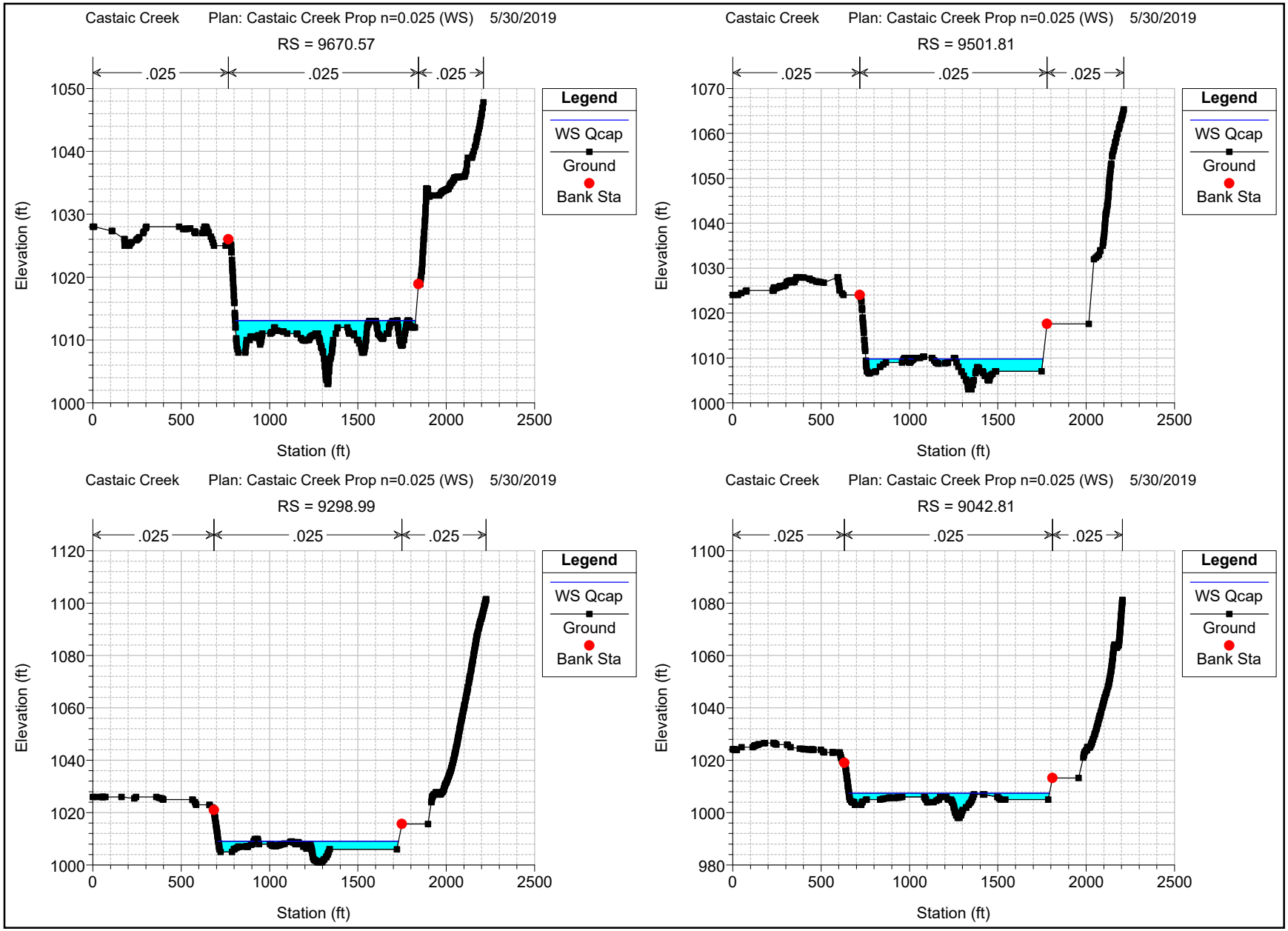
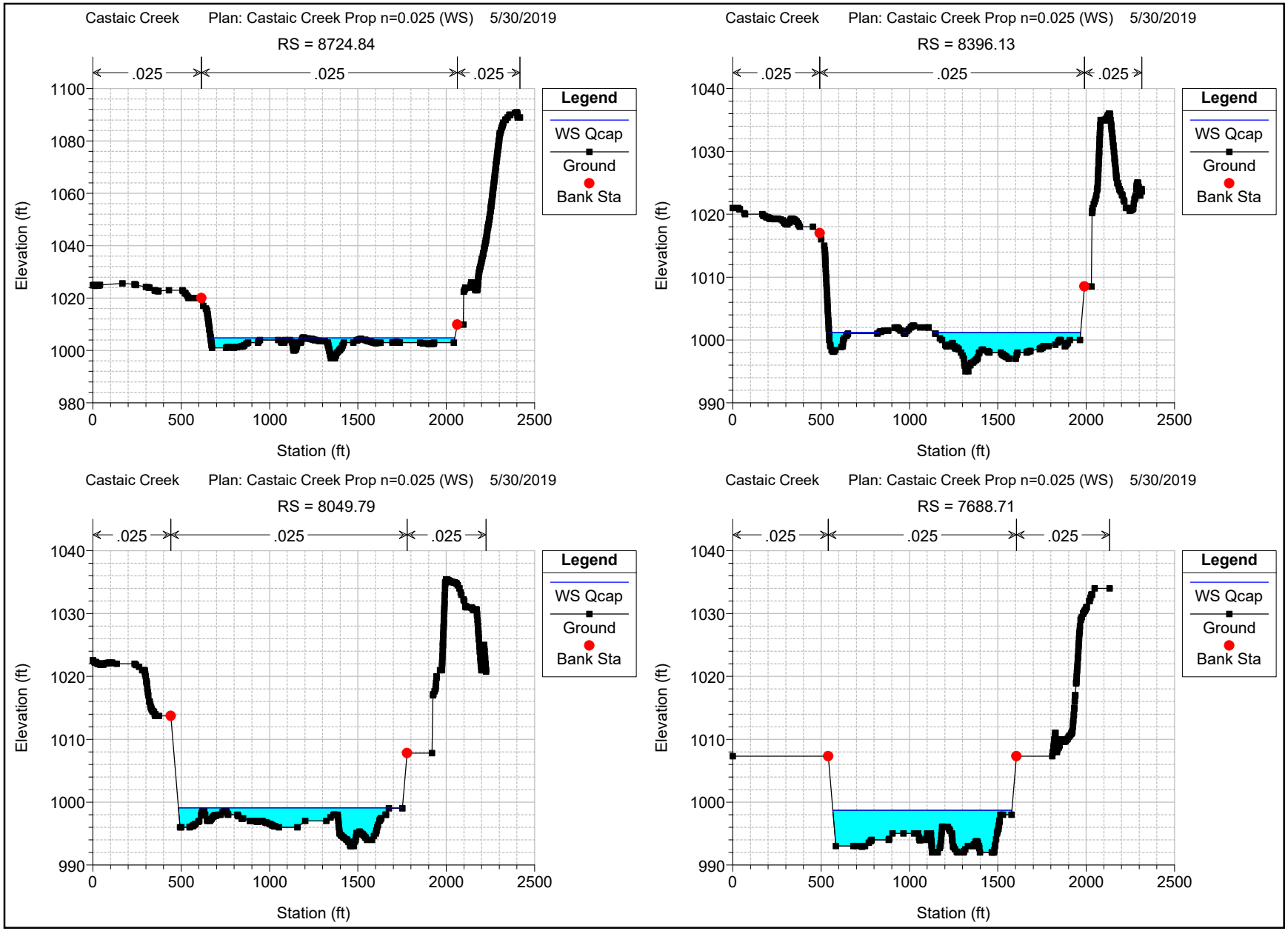
RS = 12059.65 BR I-5 Freeway Bridge (Hunsaker Survey)
.025
.025
.025
Legend
WS Qcap
Ground
Levee
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 11934
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 11923
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 11917.82 BR Old Road Bridge (Hunsaker Survey)

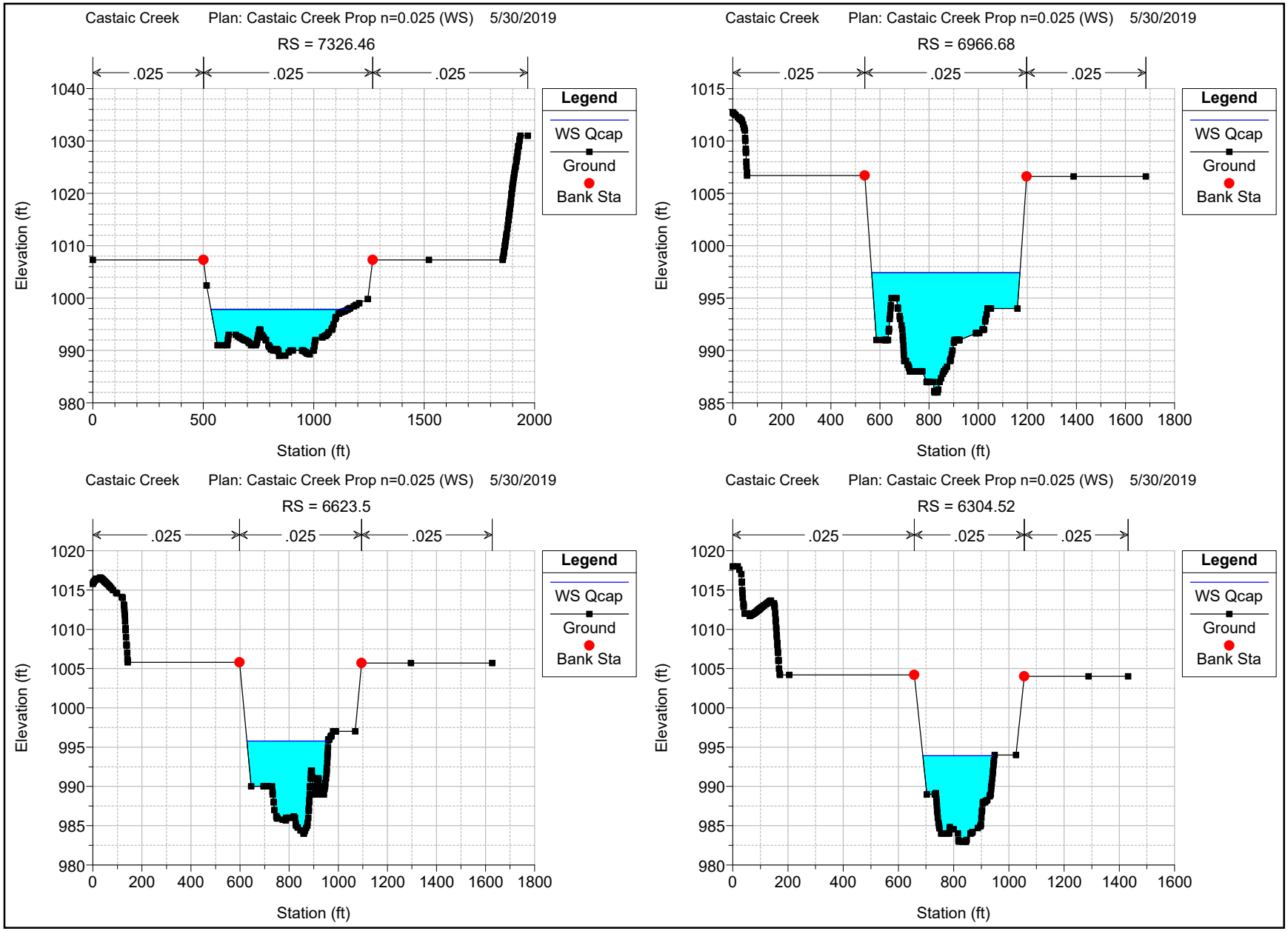

Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 11917.82 BR Old Road Bridge (Hunsaker Survey)
.025
.025
.025
Legend
WS Qcap
Ground
Levee
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 11873
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 11846
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 11811.76
Ineff

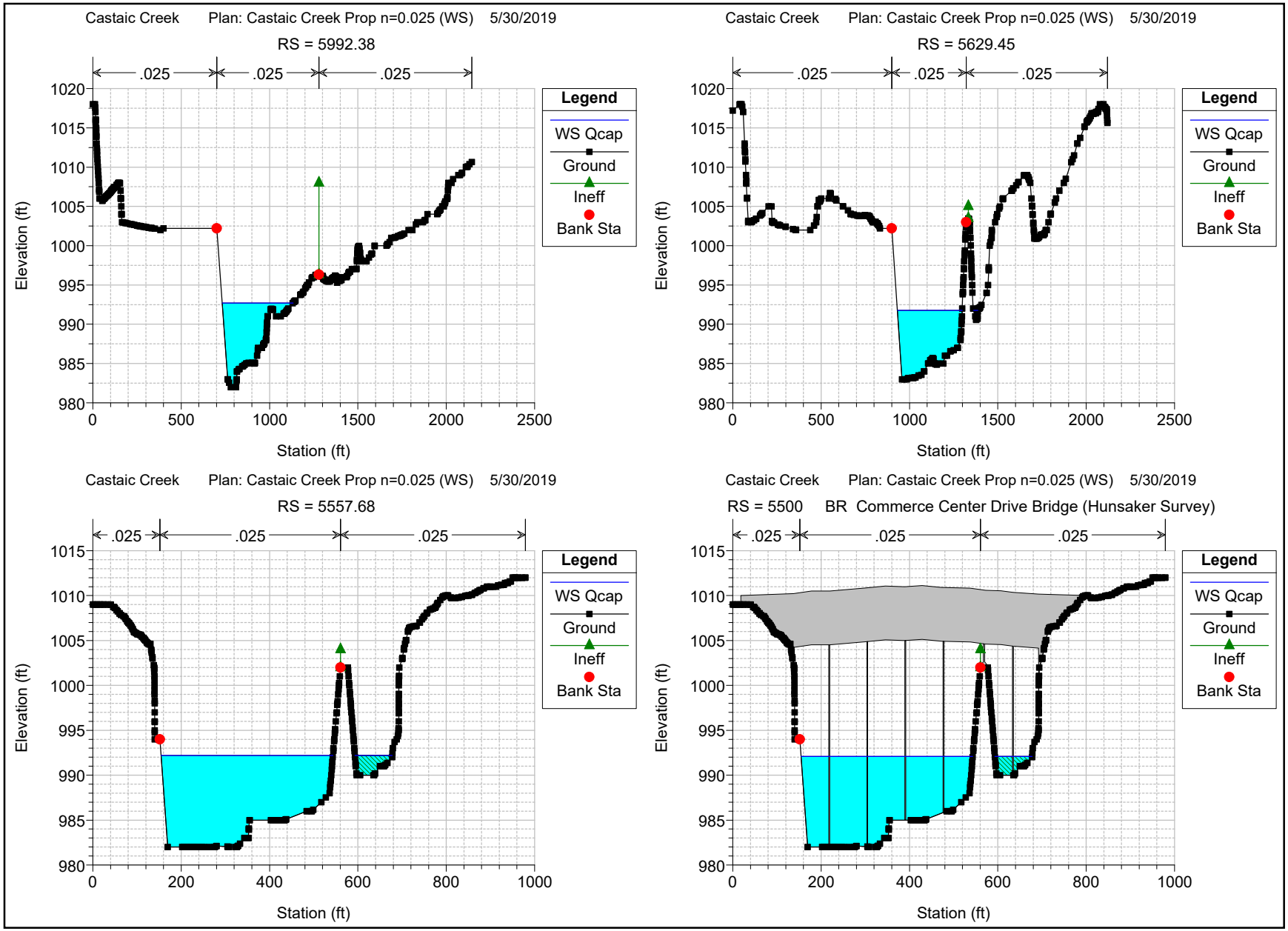

Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 11551.18
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 11242.98
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 10982.86
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 10738.26
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta

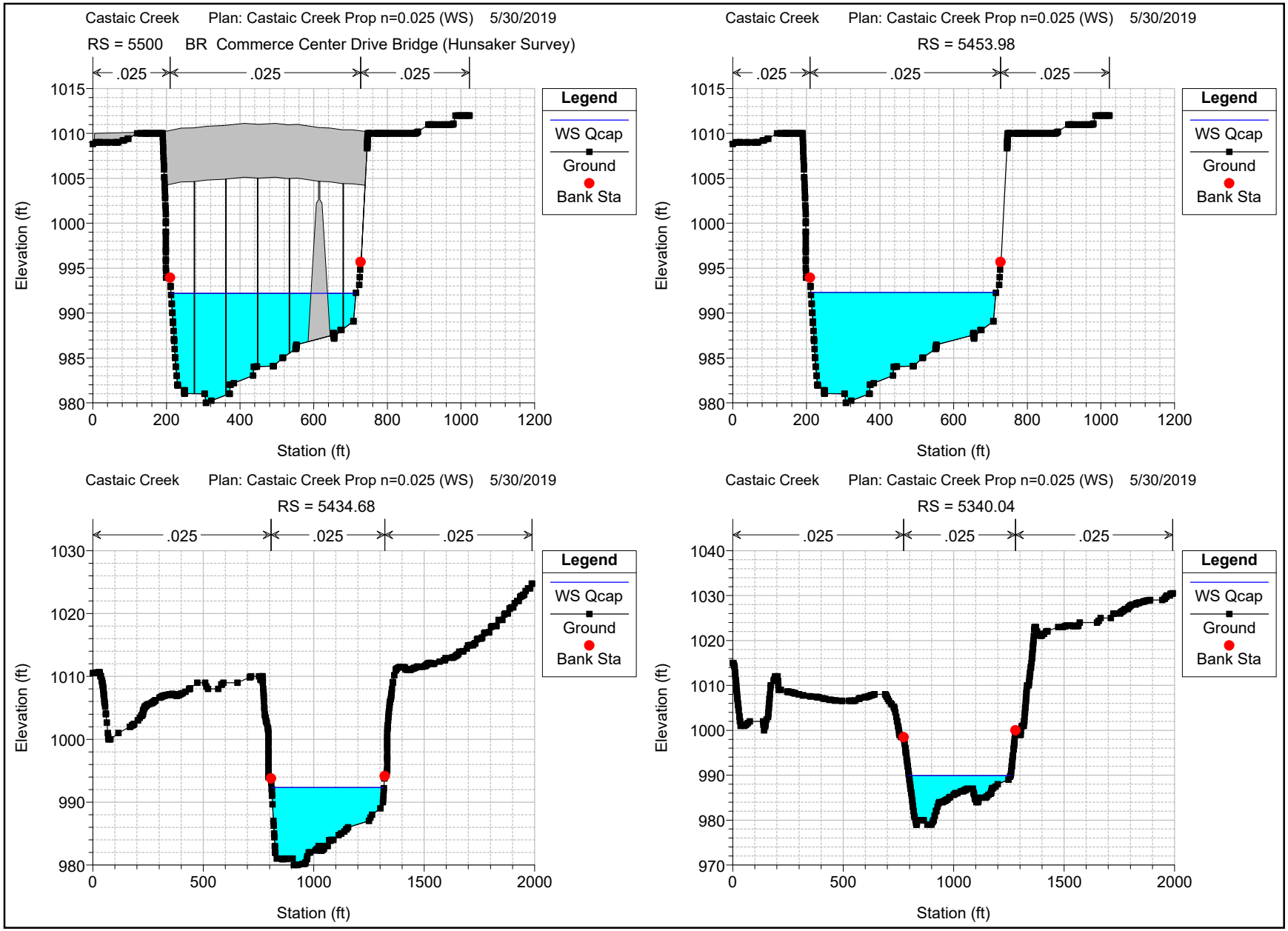

Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 10490.82
.025
.025
.025
Legend
WS Qcap
Ground
Ineff
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 10351.28
.025
.025
.025
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 10141.82
.025
.025
.025
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 9931.16
.025
.025
.025
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)

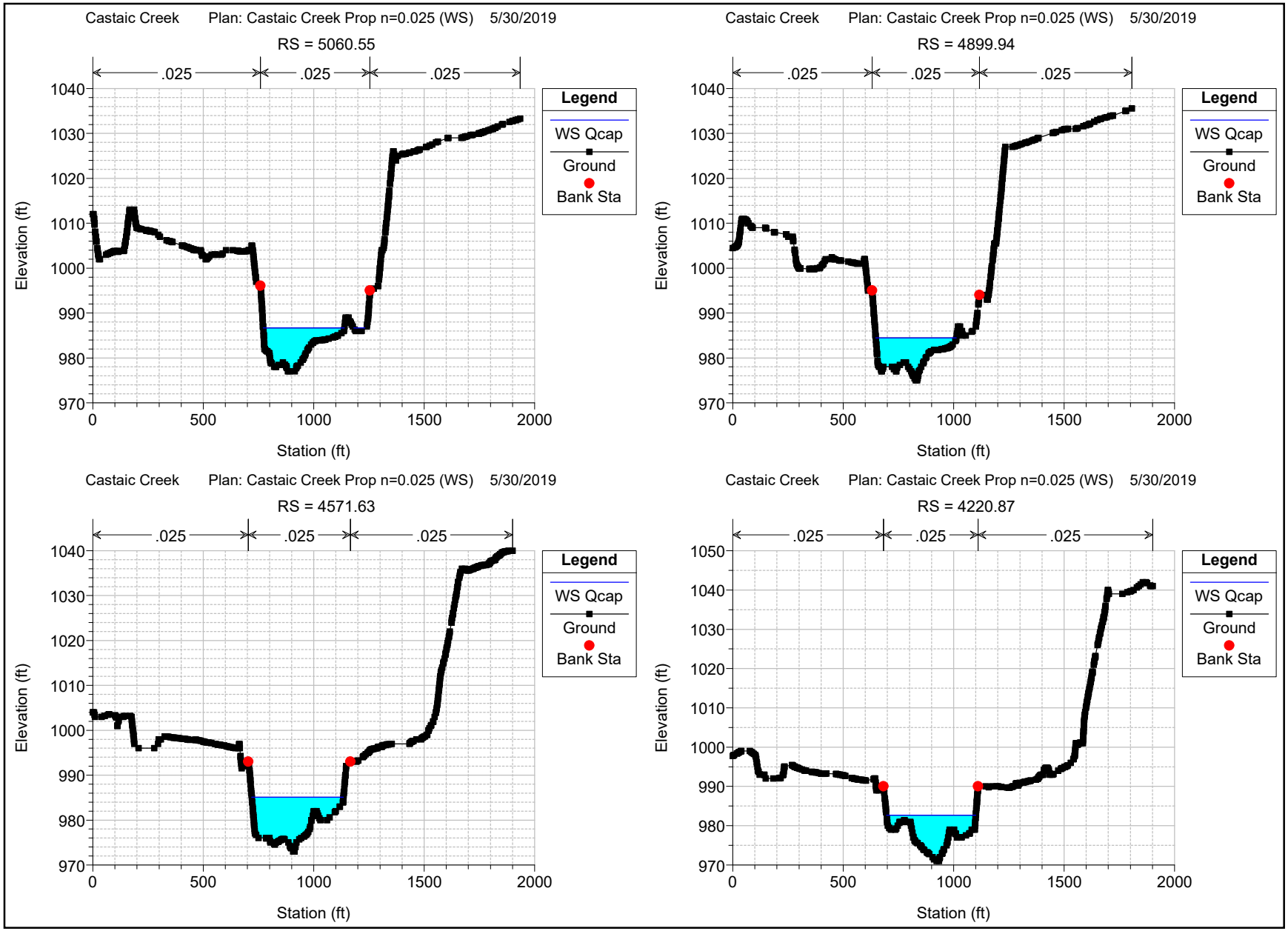

Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 9670.57
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 9501.81
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 9298.99
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 9042.81
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta

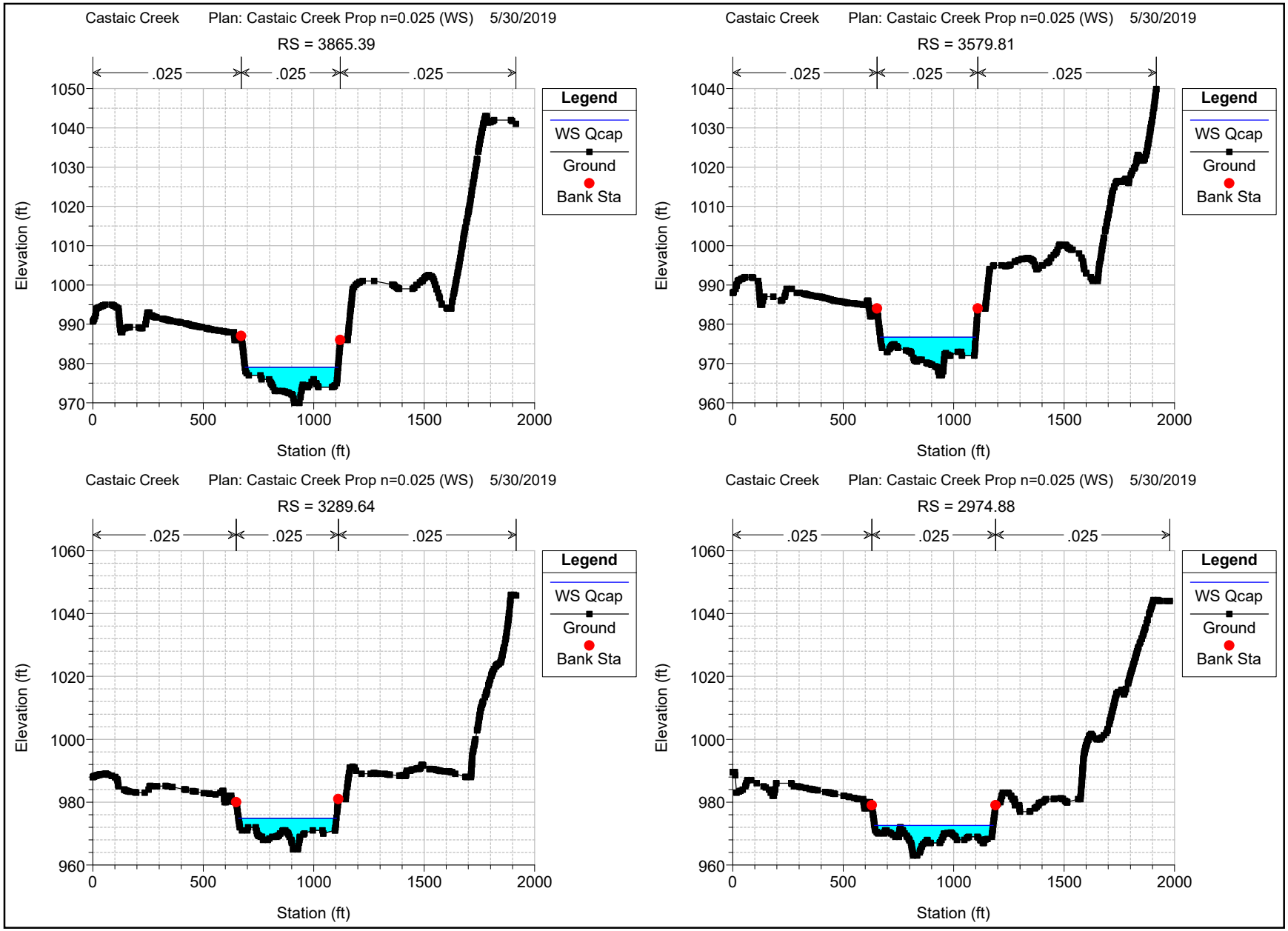

Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 8724.84
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 8396.13
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 8049.79
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 7688.71
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta

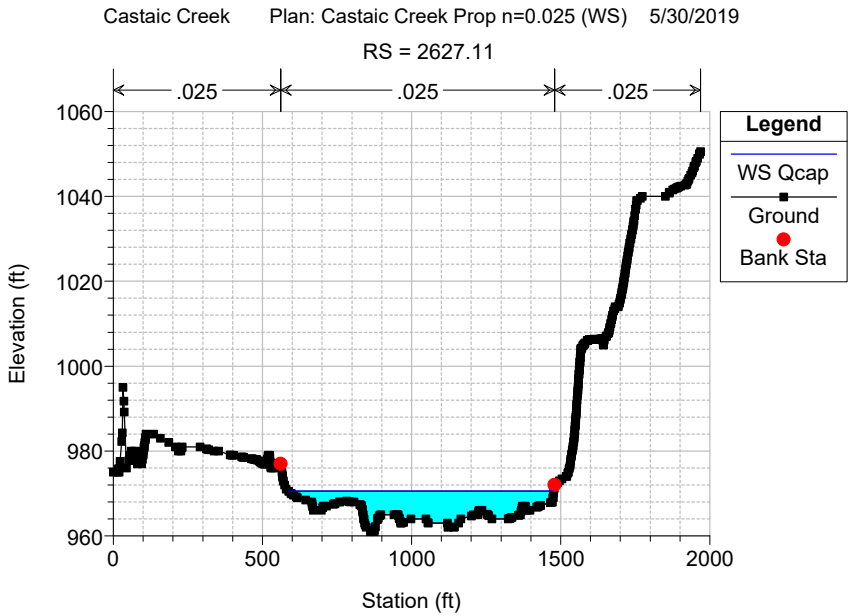

Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 7326.46
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 6966.68
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 6623.5
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 6304.52
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta

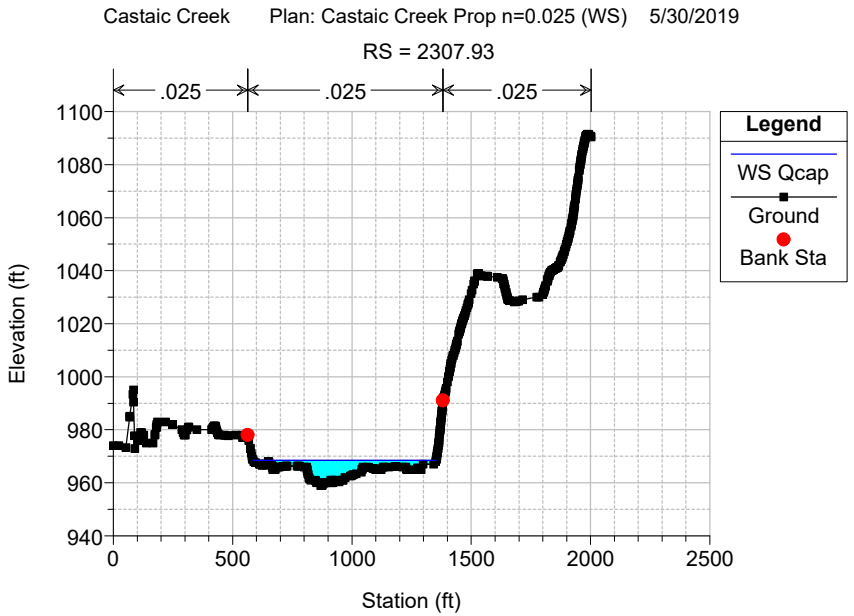

Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 5992.38
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Ineff
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 5629.45
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 5557.68
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 5500 BR Commerce Center Drive Bridge (Hunsaker Survey)

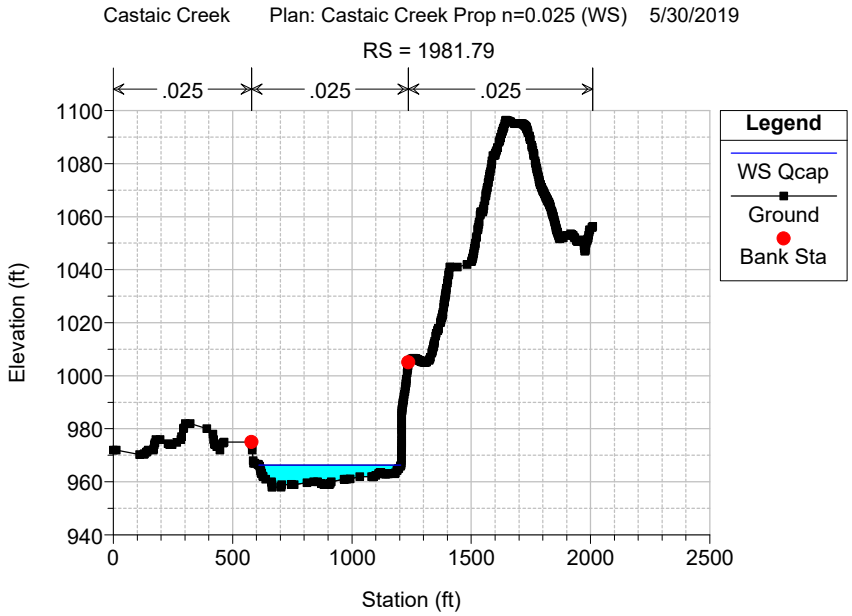

Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 5500 BR Commerce Center Drive Bridge (Hunsaker Survey)
.025
.025
.025
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 5453.98
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 5434.68
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 5340.04

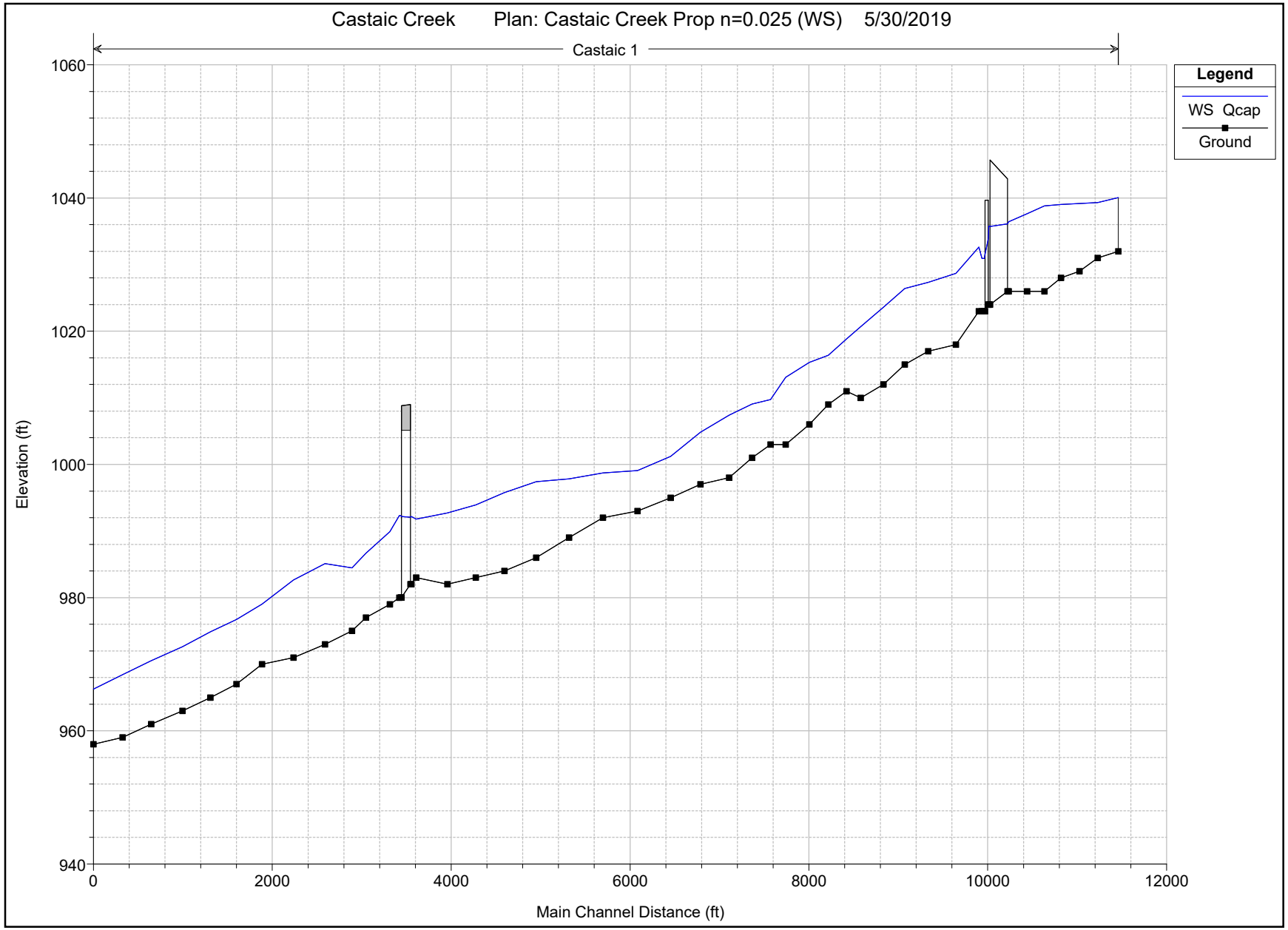

Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 5060.55
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 4899.94
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 4571.63
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 4220.87
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 3865.39
.025
.025
.025
Legend
WS Qcap
Ground
Bank Sta
Elevation (ft)
Station (ft)
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 3579.81
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 3289.64
Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 2974.88


Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 2627.11
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 2307.93
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
RS = 1981.79
.025
.025
.025
Elevation (ft)
Station (ft)
Legend
WS Qcap
Ground
Bank Sta


Castaic Creek Plan: Castaic Creek Prop n=0.025 (WS) 5/30/2019
Castaic 1
Legend
WS Qcap
Ground
Elevation (ft)
1060
1040
1020
1000
980
960
940
0
2000
4000
6000
8000
10000
12000
Main Channel Distance (ft)

Appendix E – Castaic Creek Freeboard and Toe-Down Calculation Spreadsheets

Table E-1: CALCULATIONS FOR FREEBOARD BASED ON LACDPW SEDIMENTATION MANUAL AND LACFCD HYDRAULIC DESIGN MANUAL - CASTAIC CREEK WEST BANK PROPOSED CONDITIONS FOR Q_{CAP} AND n=0.085

SECTION	Y_{MAX}	Y_{TOT}	V (FPS)	FLOW DEPTH (FT)	Y_{AGG}+	Y_{GA}+	CHANNEL TYPE	BOTTOM WIDTH (FT)	TOP WIDTH (FT)	Y_{SE}+	BEND COEFF	SIDE SLOPE	RADIUS	H/2	Y_{DM}
10351.28	2.5	1.6	6.7	11.5	1.0	0.0	2	520.6	777.5	0.0	0	1.5	1,250	0.6	2.5
10141.82	2.5	1.7	5.4	11.0	1.0	0.0	2	766.5	885.9	0.3	1	1.5	1,250	0.4	2.5
9931.16	2.5	1.6	5.0	12.1	1.0	0.0	2	794.9	950.2	0.3	1	1.5	1,250	0.3	2.5
9670.57	2.5	1.5	5.8	12.1	1.0	0.0	2	708.9	1,029.8	0.0	0	1.5	0	0.5	2.5
9501.81	2.5	1.3	4.7	10.4	1.0	0.0	2	790.9	1,021.0	0.0	0	1.5	0	0.3	2.5
9298.99	2.5	1.3	4.7	10.9	1.0	0.0	2	788.7	1,034.0	0.0	0	1.5	0	0.3	2.5
9042.81	2.5	1.6	4.7	11.9	1.0	0.3	2	1,043.4	1,147.0	0.0	0	1.5	0	0.3	2.5
8724.84	2.5	1.6	4.5	10.0	1.0	0.3	2	1,005.3	1,392.3	0.0	0	1.5	1,650	0.3	2.5
8396.13	2.5	1.4	3.2	10.3	1.0	0.3	2	890.2	1,444.4	0.0	0	1.5	1,650	0.1	2.5
8049.79	2.5	1.4	2.6	11.6	1.0	0.3	2	981.1	1,300.1	0.0	0	1.5	1,650	0.1	2.5
7688.71	2.5	1.4	2.6	12.1	1.0	0.3	2	891.0	1,045.6	0.0	0	1.5	1,650	0.1	2.5
7326.46	3.1	3.1	3.5	14.5	1.0	1.9	2	538.1	743.3	0.0	0	1.5	1,650	0.2	2.5
6966.68	3.1	3.1	3.7	16.8	1.0	1.9	2	418.4	636.9	0.0	0	1.5	1,650	0.2	2.5
6623.5	3.3	3.3	5.3	17.6	1.0	1.9	2	314.2	472.7	0.0	0	1.5	1,650	0.4	2.5
6304.52	3.4	3.4	6.4	16.9	1.0	1.9	2	242.5	373.2	0.0	0	1.5	0	0.5	2.5
5992.38	3.3	3.3	5.4	16.3	1.0	1.9	2	244.1	822.9	0.0	0	1.5	0	0.4	2.5
MAX=	3.4	3.4								0.3				0.6	2.5
MIN=	2.5	1.3								0.0				0.1	2.5

Notes:

1. Hydraulic data based on proposed condition HEC-RAS model for Qcap flowrate and Manning's "n" of 0.085 for entire reach.

2. Superelevation calculations for the river reach from RS 9931.16 to RS 10141.82, along the West Bank, are for outside curve of the reach.

DEFINITIONS

Y_{MAX} = MAXIMUM EMBANKMENT PROTECTION (FREEBOARD) VALUE IN FEET BETWEEN LACDPW SEDIMENTATION MANUAL AND LACFCD HYDRAULIC DESIGN MANUAL
Y_{TOT} = TOTAL EMBANKMENT PROTECTION (FREEBOARD) IN FEET BASED ON THE LACDPW SEDIMENTATION MANUAL
V(FPS) = VELOCITY IN FEET PER SECOND
FLOW DEPTH = WATER DEPTH IN CHANNEL IN FEET
Y_{AGG} = LONG TERM AGGRADATION IN FEET
Y_{GA} = GENERAL AGGRADATION IN FEET
CHANNEL TYPE = CHANNEL SHAPE/FLOW FACTOR:
IF Fr < 1, RECTANGULAR = **0**; IF Fr > 1, RECTANGULAR = **1**;
IF Fr < 1, TRAPEZOIDAL = **2**; IF Fr > 1, TRAPEZOIDAL = **3**.
Y_{SE} = SUPER ELEVATION IN FEET
BEND COEFF = BEND COEFFICIENT; IF NO BEND = **0**, BEND = **1**
SIDE SLOPE = CHANNEL SIDE SLOPE (H:V), UNITLESS
BOTTOM WIDTH = CHANNEL BOTTOM WIDTH IN FEET, 2-YEAR WATER SURFACE WIDTH
RADIUS = RADIUS OF CURVATURE TO CENTERLINE IN FEET
H/2 = HALF BEDFORM HEIGHT IN FEET, LIMITED TO FLOW DEPTH, AFTER KENNEDY (1963)
Y_{DM} = EMBANKMENT PROTECTION (FREEBOARD) IN FEET REQUIRED BY THE LACFCD HYDRAULIC DESIGN MANUAL

GENERAL

THIS SPREADSHEET IS DESIGNED TO CALCULATE EMBANKMENT PROTECTION (FREEBOARD) BASED ON THE LACDPW SEDIMENTATION MANUAL (2006), PAGES 59-60 AND ASSOCIATED APPENDICES. FREEBOARD VALUES BASED ON LACFCD HYDRAULIC DESIGN MANUAL ARE PRESENTED AS PART OF THE CALCULATIONS FOR COMPARISON. THE SPREADSHEET DETERMINES THE GREATER VALUE OF FREEBOARD BETWEEN THE TWO METHODOLOGIES (Y_{MAX}). ALL VELOCITIES ARE IN FEET PER SECOND (FPS), WITH NO MAXIMUM VALUE. THE USER SHOULD CONSIDER A PRACTICAL MAXIMUM VELOCITY OF APPROXIMATELY 20 TO 30 FPS. LONG TERM AGGRADATION IS USER SUPPLIED. GENERAL AGGRADATION IS ALSO USER SUPPLIED. SUPER ELEVATION AT BENDS IS BASED ON EQUATIONS IN SECTION C-3.1 (PAGE C-10) OF THE LACFCD HYDRAULIC DESIGN MANUAL. BEDFORM HEIGHT, LIMITED TO FLOW DEPTH AFTER KENNEDY (1963), IS BASED ON EQUATIONS IN APPENDIX C OF THE LACDPW SEDIMENTATION MANUAL. IF FLOW IS SUPERCRITICAL, SPREADSHEET REPORTS LACFCD HYDRAULIC DESIGN MANUAL TOTAL WALL HEIGHT INSTEAD OF FREEBOARD. BOTTOM WIDTH IS BASED ON THE 2-YEAR WATER SURFACE WIDTH AFTER U.S. ARMY CORPS OF ENGINEERS PROCEDURES.

LACDPW = LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
LACFCD = LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

COLOR CODES

OUTPUT
DATA FROM HEC-RAS
USER SUPPLIED DATA
INTERMEDIATE CALCULATIONS (INDIVIDUAL SHEETS ONLY)

DESIGNED BY DAVID A JAFFE, PHD, PE
PACIFIC ADVANCED CIVIL ENGINEERING, INC
AUGUST, 2005
FEBRUARY, 2009, REVISED
FEBRUARY, 2014, REVISED

PACE

TABLE E-2: CALCULATIONS FOR TOTAL TOE-DOWN BY INDIVIDUAL ADJUSTMENT COMPONENT BASED ON LACDPW SEDIMENTATION MANUAL AND LACFCD HYDRAULIC DESIGN MANUAL W/ SAM - CASTAIC CREEK WEST BANK PROPOSED CONDITIONS FOR Q_{CAP} AND n=0.025

SECTION	Z_{MAX}	Z_{TOT}	V (FPS)	FLOW DEPTH (FT)	Z_{DEG}+	Z_{GS}+	PIER TYPE	B	ABUT TYPE	A	BANK PROT	Z_{LS}+ BANK	Z_{LS}+ BRIDGE	BEND COEFF	HYD DEPTH	E SLOPE	TOP WIDTH	RADIUS	Z_{BS}+	Z_{I}+	H/2	Z_{DM}
10351.28	12.5	10.7	15.4	7.8	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	0	3.3	0.0138	524.4	1,250	0.0	2.0	3.2	12.5
10141.82	15.0	14.4	13.6	7.4	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	1	2.5	0.0154	770.3	1,250	4.4	2.0	2.5	15.0
9931.16	14.7	14.7	9.9	9.3	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	1	2.9	0.0068	936.1	1,250	5.9	2.0	1.3	12.0
9670.57	10.0	9.0	10.4	10.1	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	0	2.5	0.0091	1014.4	0	0.0	2.0	1.5	10.0
9501.81	10.0	10.0	13.5	6.8	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	0	2.3	0.0169	849.2	0	0.0	2.0	2.5	10.0
9298.99	8.7	8.7	9.6	8.1	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	0	2.8	0.0067	995.7	0	0.0	2.0	1.2	8.0
9042.81	8.0	7.2	9.4	9.4	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	2.5	0.0075	1134.6	0	0.0	2.0	1.2	8.0
8724.84	8.0	7.1	9.2	7.8	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	2.1	0.0089	1373.3	1,650	0.0	2.0	1.1	8.0
8396.13	10.0	7.5	10.4	6.2	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	2.3	0.0105	1127.6	1,650	0.0	2.0	1.5	10.0
8049.79	8.0	7.1	8.8	6.1	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	2.4	0.0070	1267.3	1,650	0.0	2.0	1.1	8.0
7688.71	6.4	6.4	5.5	6.7	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	4.7	0.0011	1013.4	1,650	0.0	2.0	0.4	6.0
7326.46	8.0	6.7	7.3	8.9	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	5.8	0.0014	620.3	1,650	0.0	2.0	0.7	8.0
6966.68	8.0	6.7	7.0	11.4	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	6.3	0.0012	604.7	1,650	0.0	2.0	0.7	8.0
6623.5	10.0	7.5	10.6	11.8	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	7.6	0.0022	331.5	1,650	0.0	2.0	1.5	10.0
6304.52	10.0	8.3	13.1	10.9	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	7.8	0.0032	259.6	0	0.0	2.0	2.3	10.0
5992.38	10.0	8.2	12.9	10.7	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	5.2	0.0053	400.7	0	0.0	2.0	2.2	10.0
MAX=	15.0	14.7			2.0	1.5							0.0						5.9		3.2	15.0
MIN=	6.4	6.4			2.0	0.0							0.0						0.0		0.4	6.0

Notes:

1. Hydraulic data based on proposed condition HEC-RAS model for Qcap flowrate and Manning's "n" of 0.025 for entire reach.

2. Bend scour calculations for the river reach from RS 9931.16 to RS 10141.82, along the West Bank, are for outside curve of the reach.

DEFINITIONS

V(FPS) = VELOCITY IN FEET PER SECOND
Z_{MAX} = GREATER OF **Z_{TOT}** AND **Z_{DM}**
Z_{TOT} = TOTAL POTENTIAL VERTICAL ADJUSTMENT IN FEET
Z_{DEG} = LONG TERM DEGRADATION IN FEET
Z_{GS} = GENERAL SCOUR IN FEET
Z_{LS} = LOCAL SCOUR IN FEET [**Z_{LS} BANK + Z_{LS} BRIDGE**]
Z_{LS} BRIDGE = BRIDGE LOCAL SCOUR IN FEET [PIER (APP C-4) + ABUTMENT (APP C-6)]
Z_{LS} BANK = LEVEE/BANK LOCAL SCOUR = **2.0'** IF BANK PROTECTION PRESENT
BANK PROT = BANK PROTECTION ON CHANNEL BANK
0 = YES; **1** = NO
PIER TYPE = PIER SHAPE FACTOR; IF NO PIERS = **0**
1.0 = SQUARE NOSE; **0.9** = ROUND NOSE; **0.9** = CYLINDER;
0.8 = SHARP NOSE; **0.9** = GROUP OF CYLINDERS
FLOW DEPTH = WATER DEPTH IN CHANNEL IN FEET
B = WIDTH OF PIERS IN FEET; NO PIER = **0**
ABUT TYPE = VERT WALL FACTOR; IF VERT = **2**; NON-VERT = **1**
A = ABUTMENT PROTRUSION INTO FLOW PATH IN FEET
Z_{BS} = BEND SCOUR IN FEET
BEND COEFF = BEND COEFFICIENT; IF NO BEND = **0**, BEND = **1**
HYD DEPTH = HYDRAULIC DEPTH IN FEET
E SLOPE = ENERGY SLOPE, UNITLESS
TOP WIDTH = CHANNEL TOP WIDTH IN FEET
RADIUS = RADIUS OF CURVATURE TO CENTERLINE IN FEET
Z_I = LOW-FLOW INCISEMENT IN FEET, MEASURED OR 2'; VALUE **NOT LESS THAN 2'**
H = BEDFORM HEIGHT IN FEET, LIMITED TO FLOW DEPTH AFTER KENNEDY (1963)
Z_{DM} = CUT-OFF DEPTH REQUIRED BY LACFCD HYDRAULIC DESIGN MANUAL

GENERAL

THIS SPREADSHEET IS DESIGNED TO CALCULATE SCOUR PROTECTION (TOE DOWN) BASED ON LACDPW COUNTY SEDIMENTATION MANUAL (2006) PAGES 51-59 AND ASSOCIATED APPENDICES (APPENDIX C). VALUES FOR TOE-DOWN BASED ON THE CUT-OFF DEPTHS TABLE (PAGE F-31) IN THE LACFCD HYDRAULIC DESIGN MANUAL ARE PRESENTED AS PART OF THE CALCULATIONS FOR COMPARISON. THE SPREADSHEET DETERMINES THE GREATER VALUE OF TOE-DOWN BETWEEN THE TWO METHODOLOGIES (Z_{MAX}). THE CALCULATION DOES NOT CONSIDER ADJUSTMENTS TO CUT-OFF DEPTH BASED ON FIGURE F-06 (PAGE F-37). LONG TERM DEGRADATION IS USER SUPPLIED. GENERAL SCOUR VALUES ARE INTERPOLATED FROM THE GENERAL DEGRADATION GRAPH IN APPENDIX C (PAGE C-3) OF THE LACDPW SEDIMENTATION MANUAL. BEND SCOUR AND LOCAL SCOUR AT BENDS AND ABUTMENTS ARE BASED ON APPENDIX C (PAGES C-6 TO C-8) OF THE LACDPW SEDIMENTATION MANUAL. A LONGITUDINAL EXTENT BASED ON SECONDARY CURRENTS IS NOT INCLUDED. BEDFORM HEIGHT IS BASED ON APPENDIX C-9 OF THE SEDIMENTATION MANUAL. THE OCTOBER, 2005 REVISION INCLUDES ADDITIONAL TOEDOWN AT BRIDGES/ABUTMENTS WITH SOFT BOTTOMS. THE OCTOBER, 2006 REVISION INCLUDES BLOCKAGES AT BRIDGE PIERS. THE MAY REVISION INCLUDES THE Z_{MAX} CALCULATION AND UPDATES FOR THE 2006 SEDIMENTATION MANUAL.

LACDPW = LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
LACFCD = LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

COLOR CODES
OUTPUT
DATA FROM HEC-RAS
USER SUPPLIED DATA
INTERMEDIATE CALCULATIONS (INDIVIDUAL SHEETS ONLY)

DESIGNED BY DAVID A JAFFE, PHD, PE
PACIFIC ADVANCED CIVIL ENGINEERING, INC
OCTOBER, 2004
OCTOBER, 2005, REVISED
OCTOBER, 2006, REVISED
MAY, 2008, REVISED
FEBRUARY, 2009, REVISED

Table E-3: CALCULATIONS FOR FREEBOARD BASED ON LACDPW SEDIMENTATION MANUAL AND LACFCD HYDRAULIC DESIGN MANUAL - CASTAIC CREEK EAST BANK PROPOSED CONDITIONS FOR Q_{CAP} AND n=0.085

SECTION	Y_{MAX}	Y_{TOT}	V (FPS)	FLOW DEPTH (FT)	Y_{AGG}+	Y_{GA}+	CHANNEL TYPE	BOTTOM WIDTH (FT)	TOP WIDTH (FT)	Y_{SE}+	BEND COEFF	SIDE SLOPE	RADIUS	H/2	Y_{DM}
11873	2.5	1.4	5.5	15.7	1.0	0.0	2	246.1	1,607.2	0.0	0	1.5	0	0.4	2.5
11846	2.5	1.8	7.5	15.0	1.0	0.0	2	262.1	485.0	0.0	0	1.5	0	0.8	2.5
11811.76	2.5	1.9	8.2	14.5	1.0	0.0	2	250.0	856.0	0.0	0	1.5	0	0.9	2.5
11551.18	2.5	1.6	6.5	17.4	1.0	0.0	2	362.1	691.7	0.0	0	1.5	1,250	0.6	2.5
11242.98	2.5	1.6	6.8	15.8	1.0	0.0	2	523.7	680.7	0.0	0	1.5	1,250	0.6	2.5
10982.86	2.5	1.8	7.8	15.4	1.0	0.0	2	589.6	704.1	0.0	0	1.5	1,250	0.8	2.5
10738.26	2.5	1.7	7.1	15.8	1.0	0.0	2	626.0	866.9	0.0	0	1.5	1,250	0.7	2.5
10490.82	2.5	1.5	6.2	14.8	1.0	0.0	2	472.3	829.2	0.0	0	1.5	1,250	0.5	2.5
10351.28	2.5	1.6	6.7	11.5	1.0	0.0	2	520.6	777.5	0.0	0	1.5	1,250	0.6	2.5
10141.82	2.5	1.4	5.4	11.0	1.0	0.0	2	766.5	885.9	0.0	0	1.5	1,250	0.4	2.5
9931.16	2.5	1.3	5.0	12.1	1.0	0.0	2	794.9	950.2	0.0	0	1.5	1,250	0.3	2.5
8396.13	2.5	1.4	3.2	10.3	1.0	0.3	2	890.2	1,444.4	0.0	0	1.5	1,650	0.1	2.5
8049.79	2.5	1.5	2.6	11.6	1.0	0.3	2	981.1	1,300.1	0.1	1	1.5	1,650	0.1	2.5
7688.71	2.5	1.5	2.6	12.1	1.0	0.3	2	891.0	1,045.6	0.1	1	1.5	1,650	0.1	2.5
7326.46	3.1	3.1	3.5	14.5	1.0	1.9	2	538.1	743.3	0.1	1	1.5	1,650	0.2	2.5
6966.68	3.2	3.2	3.7	16.8	1.0	1.9	2	418.4	636.9	0.1	1	1.5	1,650	0.2	2.5
6623.5	3.4	3.4	5.3	17.6	1.0	1.9	2	314.2	472.7	0.1	1	1.5	1,650	0.4	2.5
6304.52	3.4	3.4	6.4	16.9	1.0	1.9	2	242.5	373.2	0.0	0	1.5	0	0.5	2.5
5992.38	3.3	3.3	5.4	16.3	1.0	1.9	2	244.1	822.9	0.0	0	1.5	0	0.4	2.5
5629.45	3.5	3.5	6.5	13.1	1.0	1.9	2	345.6	481.5	0.0	0	1.5	0	0.6	2.5
5557.68	3.3	3.3	5.7	14.0	1.0	1.9	2	371.5	515.3	0.0	0	1.5	1,200	0.4	2.5
5453.98	3.2	3.2	4.7	15.4	1.0	1.9	2	427.8	528.6	0.0	0	1.5	1,200	0.3	2.5
5434.68	3.2	3.2	4.6	15.4	1.0	1.9	2	437.6	536.9	0.0	0	1.5	1,200	0.3	2.5
MAX=	3.5	3.5								0.1				0.9	2.5
MIN=	2.5	1.3								0.0				0.1	2.5

Notes:

1. Hydraulic data based on proposed condition HEC-RAS model for Qcap flowrate and Manning's "n" of 0.085 for entire reach.

2. Superelevation calculations for the river reach from RS 6623.5 to RS 8049.79, along the East Bank, are for outside curve of the reach.

DEFINITIONS

Y_{MAX} = MAXIMUM EMBANKMENT PROTECTION (FREEBOARD) VALUE IN FEET BETWEEN LACDPW SEDIMENTATION MANUAL AND LACFCD HYDRAULIC DESIGN MANUAL

Y_{TOT} = TOTAL EMBANKMENT PROTECTION (FREEBOARD) IN FEET BASED ON THE LACDPW SEDIMENTATION MANUAL

V(FPS) = VELOCITY IN FEET PER SECOND

FLOW DEPTH = WATER DEPTH IN CHANNEL IN FEET

Y_{AGG} = LONG TERM AGGRADATION IN FEET

Y_{GA} = GENERAL AGGRADATION IN FEET

CHANNEL TYPE = CHANNEL SHAPE/FLOW FACTOR:
IF Fr < 1, RECTANGULAR = **0**; IF Fr > 1, RECTANGULAR = **1**;
IF Fr < 1, TRAPEZOIDAL = **2**; IF Fr > 1, TRAPEZOIDAL = **3**.

Y_{SE} = SUPER ELEVATION IN FEET

BEND COEFF = BEND COEFFICIENT; IF NO BEND = **0**, BEND = **1**

SIDE SLOPE = CHANNEL SIDE SLOPE (H:V), UNITLESS

BOTTOM WIDTH = CHANNEL BOTTOM WIDTH IN FEET, 2-YEAR WATER SURFACE WIDTH

RADIUS = RADIUS OF CURVATURE TO CENTERLINE IN FEET

H/2 = HALF BEDFORM HEIGHT IN FEET, LIMITED TO FLOW DEPTH, AFTER KENNEDY (1963)

Y_{DM} = EMBANKMENT PROTECTION (FREEBOARD) IN FEET REQUIRED BY THE LACFCD HYDRAULIC DESIGN MANUAL

GENERAL

THIS SPREADSHEET IS DESIGNED TO CALCULATE EMBANKMENT PROTECTION (FREEBOARD) BASED ON THE LACDPW SEDIMENTATION MANUAL (2006), PAGES 59-60 AND ASSOCIATED APPENDICES. FREEBOARD VALUES BASED ON LACFCD HYDRAULIC DESIGN MANUAL ARE PRESENTED AS PART OF THE CALCULATIONS FOR COMPARISON. THE SPREADSHEET DETERMINES THE GREATER VALUE OF FREEBOARD BETWEEN THE TWO METHODOLOGIES (Y_{MAX}). ALL VELOCITIES ARE IN FEET PER SECOND (FPS), WITH NO MAXIMUM VALUE. THE USER SHOULD CONSIDER A PRACTICAL MAXIMUM VELOCITY OF APPROXIMATELY 20 TO 30 FPS. LONG TERM AGGRADATION IS USER SUPPLIED. GENERAL AGGRADATION IS ALSO USER SUPPLIED. SUPER ELEVATION AT BENDS IS BASED ON EQUATIONS IN SECTION C-3.1 (PAGE C-10) OF THE LACFCD HYDRAULIC DESIGN MANUAL. BEDFORM HEIGHT, LIMITED TO FLOW DEPTH AFTER KENNEDY (1963), IS BASED ON EQUATIONS IN APPENDIX C OF THE LACDPW SEDIMENTATION MANUAL. IF FLOW IS SUPERCRITICAL, SPREADSHEET REPORTS LACFCD HYDRAULIC DESIGN MANUAL TOTAL WALL HEIGHT INSTEAD OF FREEBOARD. BOTTOM WIDTH IS BASED ON THE 2-YEAR WATER SURFACE WIDTH AFTER U.S. ARMY CORPS OF ENGINEERS PROCEDURES.

LACDPW = LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
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COLOR CODES

OUTPUT
DATA FROM HEC-RAS
USER SUPPLIED DATA
INTERMEDIATE CALCULATIONS (INDIVIDUAL SHEETS ONLY)

DESIGNED BY DAVID A JAFFE, PHD, PE
PACIFIC ADVANCED CIVIL ENGINEERING, INC
AUGUST, 2005
FEBRUARY, 2009, REVISED
FEBRUARY, 2014, REVISED

PACE
Advanced Water Engineering

TABLE E-4: CALCULATIONS FOR TOTAL TOE-DOWN BY INDIVIDUAL ADJUSTMENT COMPONENT BASED ON LACDPW SEDIMENTATION MANUAL AND LACFCD HYDRAULIC DESIGN MANUAL W/ SAM - CASTAIC CREEK EAST BANK PROPOSED CONDITIONS FOR Q_{CAP} AND n=0.025

SECTION	Z_{MAX}	Z_{TOT}	V (FPS)	FLOW DEPTH (FT)	Z_{DEG}+	Z_{GS}+	PIER TYPE	B	ABUT TYPE	A	BANK PROT	Z_{LS}+ BANK	Z_{LS}+ BRIDGE	BEND COEFF	HYD DEPTH	E SLOPE	TOP WIDTH	RADIUS	Z_{BS}+	Z_I+	H/2	Z_{DM} =
11873	27.5	27.5	19.9	7.9	2.0	2.4	0.9	2.5	1	0.0	0	2.0	15.1	0	5.4	0.0121	248.6	0	0.0	2.0	4.0	14.0
11846	14.0	12.4	19.1	7.9	2.0	2.4	0	0.0	1	0.0	0	2.0	0.0	0	5.2	0.0117	268.8	0	0.0	2.0	4.0	14.0
11811.76	11.3	11.3	14.7	9.7	2.0	2.4	0	0.0	1	0.0	0	2.0	0.0	0	6.7	0.0049	283.9	0	0.0	2.0	2.9	10.0
11551.18	14.0	12.9	18.3	10.7	2.0	2.4	0	0.0	1	0.0	0	2.0	0.0	0	4.7	0.0123	441.2	1,250	0.0	2.0	4.5	14.0
11242.98	11.2	11.2	14.5	10.3	2.0	2.4	0	0.0	1	0.0	0	2.0	0.0	0	5.2	0.0067	563.2	1,250	0.0	2.0	2.8	10.0
10982.86	10.8	10.8	13.4	11.4	2.0	2.4	0	0.0	1	0.0	0	2.0	0.0	0	5.6	0.0052	669.8	1,250	0.0	2.0	2.4	10.0
10738.26	12.5	10.8	15.7	11.6	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	0	4.2	0.0104	707.7	1,250	0.0	2.0	3.3	12.5
10490.82	12.5	10.9	15.9	10.7	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	0	3.9	0.0118	482.5	1,250	0.0	2.0	3.4	12.5
10351.28	12.5	10.7	15.4	7.8	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	0	3.3	0.0138	524.4	1,250	0.0	2.0	3.2	12.5
10141.82	10.0	10.0	13.6	7.4	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	0	2.5	0.0154	770.3	1,250	0.0	2.0	2.5	10.0
9931.16	8.8	8.8	9.9	9.3	2.0	1.5	0	0.0	1	0.0	0	2.0	0.0	0	2.9	0.0068	936.1	1,250	0.0	2.0	1.3	8.0
8396.13	10.0	7.5	10.4	6.2	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	2.3	0.0105	1127.6	1,650	0.0	2.0	1.5	10.0
8049.79	12.0	10.8	8.8	6.1	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	1	2.4	0.0070	1267.3	1,650	3.8	2.0	1.1	12.0
7688.71	9.7	9.7	5.5	6.7	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	1	4.7	0.0011	1013.4	1,650	3.3	2.0	0.4	9.0
7326.46	12.0	9.9	7.3	8.9	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	1	5.8	0.0014	620.3	1,650	3.2	2.0	0.7	12.0
6966.68	12.0	10.7	7.0	11.4	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	1	6.3	0.0012	604.7	1,650	4.0	2.0	0.7	12.0
6623.5	15.0	9.8	10.6	11.8	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	1	7.6	0.0022	331.5	1,650	2.3	2.0	1.5	15.0
6304.52	10.0	8.3	13.1	10.9	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	7.8	0.0032	259.6	0	0.0	2.0	2.3	10.0
5992.38	10.0	8.2	12.9	10.7	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	5.2	0.0053	400.7	0	0.0	2.0	2.2	10.0
5629.45	10.0	7.6	10.9	8.8	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	6.7	0.0027	390.6	0	0.0	2.0	1.6	10.0
5557.68	24.8	24.8	8.4	10.2	2.0	0.0	0.9	10.0	1	0.0	0	2.0	17.8	0	8.1	0.0013	473.5	1,200	0.0	2.0	1.0	8.0
5453.98	8.0	6.6	6.6	12.3	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	8.0	0.0008	503.2	1,200	0.0	2.0	0.6	8.0
5434.68	8.0	6.5	6.3	12.3	2.0	0.0	0	0.0	1	0.0	0	2.0	0.0	0	8.3	0.0007	508.3	1,200	0.0	2.0	0.5	8.0
MAX=	24.8	24.8			2.0	2.4							17.8						4.0		4.5	15.0
MIN=	8.0	6.5			2.0	0.0							0.0						0.0		0.4	8.0

Notes:

1. Hydraulic data based on proposed condition HEC-RAS model for Qcap flowrate and Manning's "n" of 0.025 for entire reach.

2. Bend scour calculations for the river reach from RS 6623.5 to RS 8049.79, along the East Bank, are for outside curve of the reach.

DEFINITIONS

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Z_{DEG} = LONG TERM DEGRADATION IN FEET
Z_{GS} = GENERAL SCOUR IN FEET
Z_{LS} = LOCAL SCOUR IN FEET [Z_{LS} **BANK** + Z_{LS} **BRIDGE**]
Z_{LS} **BRIDGE** = BRIDGE LOCAL SCOUR IN FEET [PIER (APP C-4) + ABUTMENT (APP C-6)]
Z_{LS} **BANK** = LEVEE/BANK LOCAL SCOUR = **2.0'** IF BANK PROTECTION PRESENT
BANK PROT = BANK PROTECTION ON CHANNEL BANK
0 = YES; **1** = NO
PIER TYPE = PIER SHAPE FACTOR; IF NO PIERS = **0**
1.0 = SQUARE NOSE; **0.9** = ROUND NOSE; **0.9** = CYLINDER;
0.8 = SHARP NOSE; **0.9** = GROUP OF CYLINDERS
FLOW DEPTH = WATER DEPTH IN CHANNEL IN FEET
B = WIDTH OF PIERS IN FEET; NO PIER = **0**
ABUT TYPE = VERT WALL FACTOR; IF VERT = **2**; NON-VERT = **1**
A = ABUTMENT PROTRUSION INTO FLOW PATH IN FEET
Z_{BS} = BEND SCOUR IN FEET
BEND COEFF = BEND COEFFICIENT; IF NO BEND = **0**, BEND = **1**
HYD DEPTH = HYDRAULIC DEPTH IN FEET
E SLOPE = ENERGY SLOPE, UNITLESS
TOP WIDTH = CHANNEL TOP WIDTH IN FEET
RADIUS = RADIUS OF CURVATURE TO CENTERLINE IN FEET
Z_I = LOW-FLOW INCISEMENT IN FEET, MEASURED OR 2'; VALUE **NOT LESS THAN 2'**
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Z_{DM} = CUT-OFF DEPTH REQUIRED BY LACFCD HYDRAULIC DESIGN MANUAL

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LACDPW = LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
LACFCD = LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

COLOR CODES

OUTPUT
DATA FROM HEC-RAS
USER SUPPLIED DATA
INTERMEDIATE CALCULATIONS (INDIVIDUAL SHEETS ONLY)

DESIGNED BY DAVID A JAFFE, PHD, PE
PACIFIC ADVANCED CIVIL ENGINEERING, INC
OCTOBER, 2004
OCTOBER, 2005, REVISED
OCTOBER, 2006, REVISED
MAY, 2008, REVISED
FEBRUARY, 2009, REVISED
FEBRUARY, 2014, REVISED

Appendix F – Approved Castaic Creek Fluvial Study Phase I Final Draft

FYI MEK

COUNTY OF LOS ANGELES

DEPARTMENT OF PUBLIC WORKS

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DONALD L. WOLFE, Director

ADDRESS ALL CORRESPONDENCE TO:
P.O. BOX 1460
ALHAMBRA, CALIFORNIA 91802-1460

April 18, 2006

IN REPLY PLEASE
REFER TO FILE: LD-0

Mr. Mark Subbotin
Vice President of Community Development
Newhall Land/Lennar
23823 Valencia Boulevard
Valencia, CA 91355-2194

Dear Mr. Subbotin:

HEC-RAS AND PHASE 1 FLUVIAL ANALYSIS FOR:
CASTAIC CREEK I-5 TO SANTA CLARA RIVER CONFLUENCE
SANTA CLARA RIVER I-5 TO VENTURA COUNTY LINE

This letter has been prepared to memorialize and clarify the conclusions and approvals of the County of Los Angeles Department Public Works relative to the completed review of the following reports prepared by PACE, Inc.

- Newhall Ranch Santa Clara River HEC-RAS Modeling Report dated December 2005 (I-5 to Ventura County Line).
- Castaic Creek HEC-RAS Modeling Report dated December 2005 (I-5 to Hwy 126).
- Newhall Ranch–Santa Clara River Phase 1 Fluvial Study dated March 6, 2006, (final date pending).
- Castaic Creek Phase 1 Fluvial Study dated January 20, 2006.

Based on our review of the reports provided to date, we will regard the following items as approved:

1) Reestablishment of a revised Capital Floodplain HEC-RAS model and water surface profile for design and ML map revision (n=0.06 per the approved reports).

2) The proposed bank protection horizontal alignment, as presented in the reports, for Newhall Ranch–Santa Clara River (I-5 to Ventura County line) and Castaic Creek (I-5 to Santa Clara River) confluence.

3) The top and toe elevations of the bank protection for Castaic Creek from I-5 to the Santa Clara River confluence and for the Santa Clara River from I-5 to the Chiquito Canyon Creek confluence (Landmark, Entrada, Mission Village, and Commerce Center). Refer to the enclosed exhibit for proposed bank protection

locations. The approved toe-down depth for these proposed bank protection projects is the greater (deeper) of the Los Angeles County Hydraulic Design Manual or the Los Angeles County Hydrology and Sedimentation Manual (utilizing the results of the Phase 1 Fluvial Study long-term aggradations/degradation analysis and the SAM model general degradation modeling results). In addition, the top of bank elevation for these proposed projects is as determined by the results of the approved Phase 1 Fluvial Analysis.

The remaining proposed projects within Newhall Ranch, where the top and toe elevations are not approved, will require further analysis of the Newhall Ranch Development within the tributary drainage areas and the evaluation of the resulting fluvial impacts (if any) to the Santa Clara River as related to changes in tributary sediment delivery pre- versus post-development condition.

The final design and permitting for each individual development requiring bank protection will be based upon the final results as concluded in the aforementioned reports and the Phase 2 Fluvial Studies. The procedure for the use of these reports and bank protection project construction phasing will be as follows:

1) Conceptual Design Report–Public Works approval
2) Hydrology (Final Design Report)–Public Works approval
3) Construction Plans and Specifications–Public Works approval
4) County ML Map Revisions–County Conditional and Final Approval

Please confirm that you are in agreement with the conclusions and approvals noted above. If you have any further questions, please contact Mr. Steven Sheridan of my staff at (626) 458-7151.

Very truly yours,

DONALD L. WOLFE
Director of Public Works

Dennis Hunter

DENNIS HUNTER
Assistant Deputy Director
Land Development Division

SDS:la
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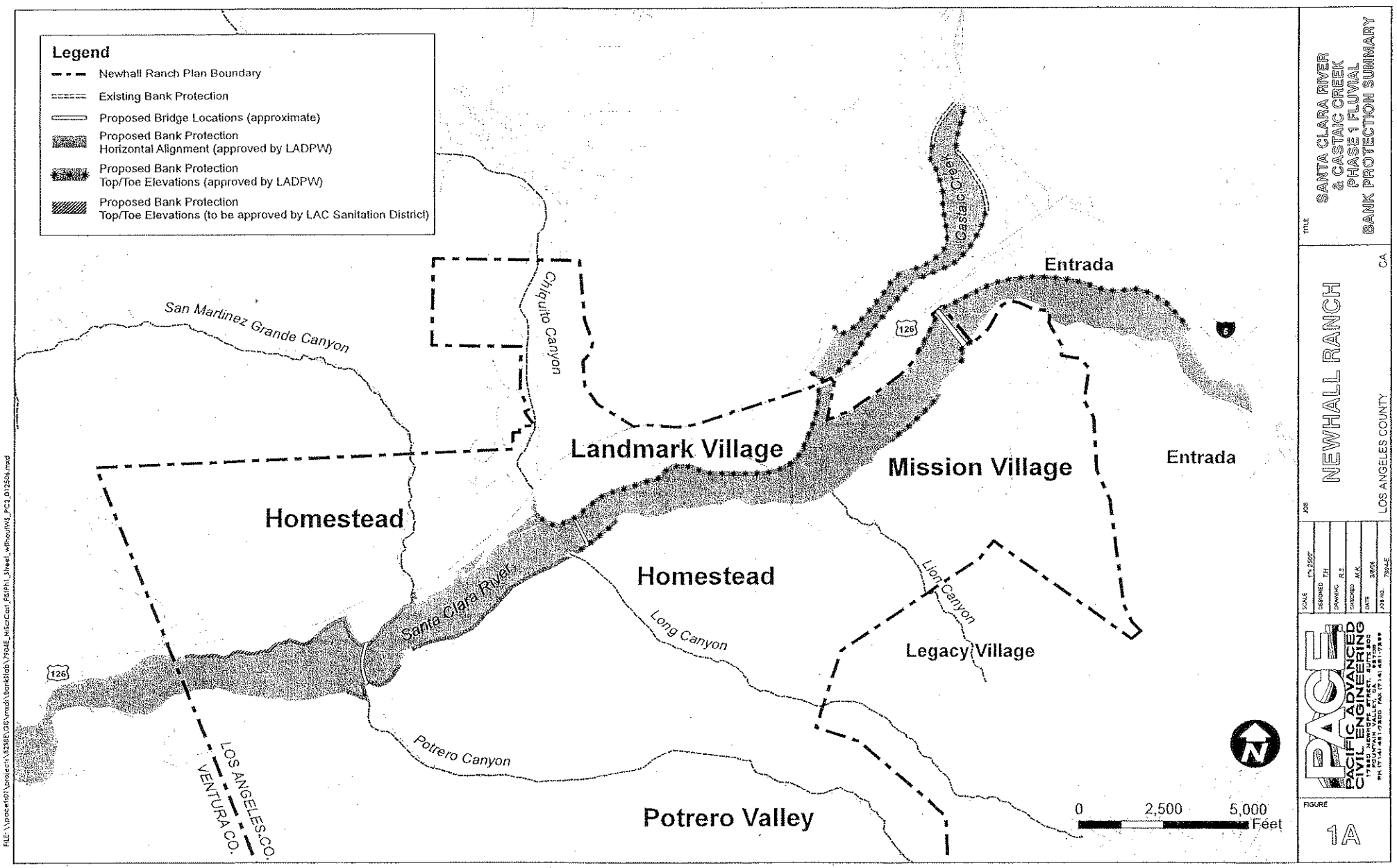
Enc.

cc: PACE Engineering (Mark Krebs)


Legend
Newhall Ranch Plan Boundary
Existing Bank Protection
Proposed Bridge Locations (approximate)
Proposed Bank Protection
Horizontal Alignment (approved by LADPW)
Proposed Bank Protection
Top/Toe Elevations (approved by LADPW)
Proposed Bank Protection
Top/Toe Elevations (to be approved by LAC Sanitation District)
San Martinez Grande Canyon
Chiquito Canyon
Castaic Creek
Entrada
Landmark Village
Mission Village
Entrada
Homestead
Homestead
Santa Clara River
Long Canyon
Lion Canyon
Legacy Village
Potrero Canyon
Potrero Valley
LOS ANGELES CO.
VENTURA CO.
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SANTA CLARA RIVER & CASTAIC CREEK PHASE 1 FLUVIAL BANK PROTECTION SUMMARY
NEWHALL RANCH
LOS ANGELES COUNTY
CA
PACE
PACIFIC ADVANCED CIVIL ENGINEERING
FIGURE
1A

PACIFIC ADVANCED CIVIL ENGINEERING, INC.

17520 Newhope Street, Suite 200 ▪ Fountain Valley, California 92708 ▪ 714.481.7300 ▪ fax: 714.481.7299

January 20, 2006

Paul Risher
Water Resources Division
Los Angeles County Department of Public Works
900 S. Freemont Avenue
Alhambra, CA 91803

Re: Response to Land Development Comments for the Review of Newhall Ranch Castaic Creek Fluvial Analysis SAM Model and Long-Term Topography Review dated November 22, 2005 **#8065E**

Dear Mr. Risher:

Pacific Advanced Civil Engineering, Inc. (PACE) is pleased to provide the following response to the Land Development Division review comments on the Fluvial Analysis. LADPW comments on the Fluvial Analysis are addressed in the submittal of the "Creek Fluvial Study – Castaic Creek" dated November 22, 2005, prepared by PACE.

Items To Be Addressed

Comment 1. *PACE response to Comment 4, a legible copy of the report is available in the DPW Technical Library, which can be referenced in the report.*

PACE Response

The text has been updated to reflect this comment.

Comment 2. *Page iii, first paragraph, last sentence states that the subreaches were determined by examining the hydraulic parameters. Add discussion on how the subreach boundaries were determined. See Comment 38.*

PACE Response

As per Comment 38 sections and stations have been differentiated throughout the text. A complete description of the how the subreaches were chosen is located in Chapter 4.1.2.

Comment 3. *Page iii, a typo in the first line of the second paragraph. Should read, "…percent finer data form obtained from sieve analysis…"*

PACE Response

The text has been updated to reflect this comment.

Comment 4. *Page iv, a typo in the fifteenth line of the first paragraph. Should read, "…which the bed will not degrade."*

PACE Response

The text has been updated to reflect this comment.

Comment 5. *Page iv, a typo in the twenty-second line of the first paragraph. "channelizeation" should be 'channelization'. Also, a sentence could be added to explain how the dam has created this effect on the creek.*

PACE Response

The text has been updated to reflect this comment. Additionally, a reference to the explanation of the Dam's effect on Creek in Chapter 5 has been included.

Comment 6. *Page v, fourth line, second paragraph, "Bend scour occurs because velocity gradients around curves…" Grammar needs to be corrected.*

PACE Response

The text has been updated to reflect this comment.

Comment 7. *Page v, a typo in the twelfth line of the second paragraph. Should read, "For both the existing condition…"*

PACE Response

The text has been updated to reflect this comment.

Comment 8. *Page v, a typo in the third line of the fourth paragraph. Should read, "…a shallower toedown in for both…"*

PACE Response

The text has been updated to reflect this comment.

Comment 9. *Page 1, Introduction, line seven. State "The Castaic Creek lies within…" to avoid confusion with Hasley Creek mentioned above.*

PACE Response

The text has been updated to reflect this comment.

Comment 10. *Figure 1.1, the subreach labels should be changed to be consistent with the rest of the report i.e., SRB1 instead of RB1.*

PACE Response

The text has been updated to reflect this comment.

Comment 11. *Page 2-3, technical framework 1-11, state which of the objectives were addressed by this study, and which will be done later. The other alternative is to remove the reference to HEC-6, which appears to be the only part of the plan not completed.*

PACE Response

The text has been updated to reflect this comment and a note has been added that this work will be included in a Phase 2 report.

Comment 12. *Page 2, point 6 states that different storm periods will be run with SAM to determine sediment inflow parameters. This point is still of concern to DPW and will be addressed at a later time along with all analysis utilizing HEC-6.*

PACE Response

The text has been updated to reflect this comment and the HEC-6 reference removed.

Comment 13. *Page 4, section 2.2 last sentence is confusing with use of "although" twice. Revise for readability.*

PACE Response

The text has been updated to reflect this comment.

Comment 14. *Page 5, a typo in the seventh line of the first paragraph. Should read, "…feet in for…"*

PACE Response

The text has been updated to reflect this comment.

Comment 15. *Page 5, section 2.4, state the flow rate used in the Tetra Tech report for comparison purposes with the other reports.*

PACE Response

The text has been updated to reflect this comment.

Comment 16. *Page 6, Table 3.1, explain how the D_{50} reach averages were computed. According to my calculation check, the 3 samples in each cross section were averaged and then the reaches averages were averaged. Which means, the one sample from CS section was weighted equal to the average of each of the other sections (CH, HW, and HS). Notice should be taken that the stations HS and CS are beyond the boundary of the reach modeled by SAM. They happen to be quite different from the samples taken within SRB, and change the average to be heavier than if they were not considered. The change in the gradations should not affect the SAM calculations significantly.*

PACE Response

The D_{50} in Table 3.1 for subreach SRB contained a typographical error. This error has been corrected. Moreover, that one sample from CS was weighted equal to the other averages is not correct. The average D_{50} is calculated from the average of all samples for a given reach. The corrected D_{50} should indicate this. A footnote was added to Table 3.1 for further clairification. Moreover, that HS and CS are included in the average was enacted at the request of the County in prior communication. Station CS only has lower percent passing for sieve sizes larger than approximately Number 16, while for lower sieve sizes CS is approximately the average. The text has been updated to reflect this response.

Comment 17. *Page 7, Section 3.2 the explanation of sampling here states, "Averaging is accomplished by taking the mean of the three samples from a given station." It should continue to explain that the stations are then averaged to get one average gradation curve for the whole reach according to Figures 3.2 A and B.*

PACE Response

This is not correct. Please see response to Comment 16. Average gradation is the average of individual samples, not the average of averages. The text has been updated to reflect this comment.

Comment 18. *Page 7, Section 3.3 line three "samples ranged from 0.51 0.46 to 4.24mm."*

PACE Response

The text has been updated to reflect this comment.

Comment 19. *Figure 3.2B, the average gradation used in the SAM analysis for reach B is consistently about 2 percent below the curve in the figure. This should not influence the validity of the SAM modeling. However, the curve should be evaluated to determine the final gradation curve, or the figure should be changed to match the models.*

PACE Response

The typographical error in the referenced file has been corrected. Please see response to Comment 16. The text has been updated to reflect this comment.

Comment 20. *Page 10, Section 4.1.1b, second paragraph, eleventh line, remove the word "approved" before "Newhall Ranch Specific Plan." Parts of it have not been approved, specifically the hydraulic models.*

PACE Response

The text has been updated to reflect this comment.

Comment 21. *Page 10, Section 4.1.1b, second paragraph, lines 15 -17, revise sentence for grammar, i.e., "…stationing have…" Also, the whole second paragraph becomes confusing by virtue of its length and detail: it should be revised to improve readability.*

PACE Response

The text has been updated to reflect this comment.

Comment 22. *Page 11, Section 4.1.2, first paragraph, lines 5-6 state that 5 separate subreach lengths were run for the sensitivity analysis (1200, 1400, 1750, 2300, and 3500 feet). However, in Appendix 4.5, only 4 tables are shown. Please explain the discrepancy or correct it.*

PACE Response

In personal communication with the County (B. Willardson, 1-11-06) it was decided to resolve this comment by removing the 3500 foot reach length from the analysis because when using this length only two subreaches remained in an entire reach for analysis. It is believed that with the longest subreach removed the convergence of SAM values based on varied reach lengths for the existing and proposed conditions will be evident. The text has been updated to reflect this comment.

Comment 23. *Page 11, Section 4.1.2, first paragraph, lines 9-12 state that the "…calculated general adjustment converges to a single value of adjustment for each of the subreaches..." The convergence was not recognizable in Appendix chapter 4.5. Some discussion of the convergence and how to recognize convergence should be included either here or in the referenced appendix to clarify how the final reach lengths were selected.*

PACE Response

Please see response to Comment 22.

Comment 24. *Page 11, Section 4.1.2, first paragraph, tenth line, make the reference to "Appendix Chapter 4.5".*

PACE Response

The text has been updated to reflect this comment.

Comment 25. *Page 11, Section 4.1.2, second paragraph, seventh line, change to "…shifting in lateral discharge tributary positions…" This should be further clarified by explaining the reason for doing so, i.e., Hasley Canyon Creek outflanking the flow trainer.*

PACE Response

This issue was resolved in revisions to the HEC-RAS model during the LAC review process and the text in question has been removed.

Comment 26. *Figure 12, a typo in the ninth line of the second paragraph. Should read, "…choosing and an individual…"*

PACE Response

The text has been updated to reflect this comment.

Comment 27. *Page 12, a typo in the third line of the last paragraph. Should read, "…Yang and MPN MPM equations…"*

PACE Response

The text has been updated to reflect this comment.

Comment 28. *Page 13, Section 4.1.3, second paragraph, tenth line, make the reference to "Appendix Chapter 4.2".*

PACE Response

The text has been updated to reflect this comment.

Comment 29. *Page 14, first paragraph, give a reference for the general adjustment equation or state that it was derived by PACE.*

PACE Response

The general adjustment calculation is merely a chain calculation. The text has been updated to reflect this comment.

Comment 30. *Page 14, second paragraph, ninth line should read "…as computed by the HEC-2 conversion from the HEC-RAS model…"*

PACE Response

The text has been updated to reflect this comment.

Comment 31. *Page 14, second paragraph, last sentence needs a period at the end.*

PACE Response

The text has been updated to reflect this comment.

Comment 32. *Page 14, a typo in the sixth line of the last paragraph. Should read, "…Subreaches SRA4 and SRB1 changing changed…"*

PACE Response

The text has been updated to reflect this comment.

Comment 33. *Page 15, a typo in Tables 4.2A, 4.2B, and 4.3 in the footnote of each "significatnly" should read "significantly."*

PACE Response

The text has been updated to reflect this comment.

Comment 34. *Page 15, Table 4.4 is misleading because it compares the SAM adjustment for a whole subreach to the LACH&SM for only the upstream cross section. Revise the table to include all the subreaches, or an average of the LACH&SM adjustment for the whole subreach length. If the reach average method is used, please add the phrase "Reach Averaged" to the Existing and Proposed Conditions titles.*

PACE Response

As per personal communication with LACDPW (B. Willardson, 1-3-06) footnotes have been added to the tables to indicated that values calculated based on LACH&SM are for single cross-sections only and directs the reader to Appendix Chapter 6 for the complete data set. The text has been updated to reflect this comment.

Comment 35. *Figure 4.1A, the subreach labels (SRA1, etc…) are not correct, shown in the opposite order. Also, Comment 34 applies to the LACH&SM data. A note should be added that the figure depicts reach average data, not a continuous bed profile as the solid line suggests. Since the data is reach averaged, the value should be shown at the centroid of each subreach, not at the upstream cross section.*

PACE Response

As per personal communication with LACDPW (B. Willardson, 1-3-06) to indicate that the data is not continuous individual data points have been placed on the figures connecting the lines on the graphs and representing the specific location of the data values. The full data set is available in Appendix Chapter 6.

Comment 36. *Figure 4.1B, see Comments 34 and 35 above.*

PACE Response

Please see response to Comment 35.

Comment 37. *Page 17, Section 5, eleventh line, states that certain cross sections were chosen for the analysis. Discussion should be added as to why they were chosen. Are they representative of the subreach? Does something significant happen at that station? etc. This can be done in the commentary on each section later in Section 5.*

PACE Response

The text has been updated to note that historical sections were chosen based on locations of engineering interest. The important feature that warrant each section's choice is outlined in the individual description of the respective section.

Comment 38. *Pages 17-25 Section 5, in total, clarification on 'stations' is needed. Throughout the report the terms 'section' and 'station' were used to refer to the HEC-RAS cross sections made along the centerline of the creek. In Section 5 only, 'section' refers to the HECRAS cross sections and 'station' refers to the horizontal stations perpendicular to the center line and shown on the x-axis of Figures 5.2A-I. There should be consistency throughout the entire report of what the terms refer to. The simplest*

answer would probably be to call the lateral station an 'x-station' or just 'lateral station' to differentiate between lateral and longitudinal stationing and leave the rest as-is. Define the term in the report.

PACE Response

Please see response to Comment 2.

Comment 39. *Figure 5.1, the subreach labels should be changed to be consistent with the rest of the report i.e., SRB1 instead of RB1.*

PACE Response

The figure has been updated to reflect this comment.

Comment 40. *Page 18, second paragraph, last sentence, revise. This sentence is somewhat confusing, maybe an example would make it clearer. How is the vertical elevation the same? do they have the same datum?*

PACE Response

The text has been updated to reflect this comment.

Comment 41. *Page 18, third paragraph, second sentence, the reference to Figure 5.2I should be 5.2A for section 9299.*

PACE Response

The text has been updated to reflect this comment.

Comment 42. *Page 19, Tables 5.1 A and B, make a footnote with reference to Appendix 5.3 for the historical thalweg analysis.*

PACE Response

The historical thalweg analysis provides a less detailed picture of the historical trends of the creek bed than does the presently included sectional analysis. The thalweg analysis is only included for completeness. As per personal communication with LACDPW personnel (B. Willardson, 1/3/06) no footnote has been added.

Comment 43. *Page 19, second paragraph, first sentence, the reference "(1.3 feet, Table 5.1)" is inconsistent with the text which states that it is for "1963 to the present." In fact, the reference is for 1999 to 2005.*

PACE Response

The text has been updated to reflect this comment.

Comment 44. *Page 19, second paragraph, fifth line, states "the bed is at or approaching the armoring depth." Figure 5.2C shows a bed that fluctuates in the period 1999-2005.*

Provide more evidence or commentary on why section 6624 is approaching the armor depth.

PACE Response

Any analysis of the armoring depth will be considered in the second phase fluvial analysis. The reference has been removed.

Comment 45. *Page 20, third paragraph, for sections 1982, 1645, and 1531, discussion should be added concerning the apparently large bank erosion on the east bank of section 1531. Figure 5.2I shows a large lateral change between 300 and 450. Also, more discussion needs to be added concerning how the thalweg moved to the inside of the curve through these sections. The shift in the thalweg is contrary to river mechanics. Was there channelization, an upstream meander, earthmoving, etc?*

PACE Response

The analysis of the section and relevant topography has been revised figure updated. The revised figure and the original topographic data do not show any erosion. The County has indicated that they believe the movement of the thalweg from one side of the creek to the other is a function of possible earth moving activities, although the movement may also result from the confinement of the channel by the bridge abutment. The text has been updated to reflect this comment.

Comment 46. *Page 22, Table 5.2, include the WSE up to the hundredths place so that the Difference column will match the difference calculated from the two WSE's. Rounding the difference column to tenths is acceptable, but with the extra decimal place the reader will be able to see the rounding error.*

PACE Response

The table has been updated by rounding the raw data used in the table instead of including additional decimal places in table. The table has been updated to reflect this comment.

Comment 47. *Page 23, Table 5.3, see previous comment.*

PACE Response

See response to Comment 46.

Comment 48. *Page 24, Section 5.2, last sentence, revise to say that SAM calculated general adjustment will be used for preliminary design for tentative tract map approval, and that final design will include information from the Phase 2 study.*

PACE Response

The text has been updated based on a 1-12-06 meeting with LACDPW staff. The text has been updated to read: "Final design will be based on the deeper of LACH&SM using SAM and LACFCDDM criteria."

Comment 49. *Page 28, Section 7, fifteenth line states that "The figure shows…" which figure is being referred to?*

PACE Response

The text has been updated to reflect this comment.

Comment 50. *Page 28-29, Tables 7.1 A-D, Z_{deg} for SRB2 should be updated to be –2.8 according to the changes on Table 5.1B (per phone call with David Jaffe on 12/12/05). A footnote should be added referring to this table and the fact that the Z_{deg} values are taken at differing cross sections within the subreach, not necessarily the upstream cross section. Also, similar to Comment 34 above, Z_{other} is taken at only the upstream cross section of each subreach. This data should be averaged for the cross section or the most conservative Z_{other} within the subreach should be used to make a better comparison. Either way, a footnote should be added stating what data was used.*

PACE Response

The typographic error has been corrected in the table. The footnote has been added to the table. Also, please see response to Comment 34.

Comment 51. *Page 30-31, Tables 7.2 A-D, update the LACH&SM columns according to the previous comment. Update the LACFCDDM columns similarly, using an average value for the subreach, or the most conservative within the subreach.*

PACE Response

Please see response to Comments 34 and 50.

Comment 52. *Page 32, Tables 7.3A-B, include some discussion stating that SAM calculated degradation will not be included in the general adjustment column for aggradation. Also, similar to previous comments use reach averaged data for comparisons by subreach.*

PACE Response

The text has been updated to reflect this comment.

Comment 53. *Figures 7.1A-D, update according to Comment 50 above and place value in the centroid of the subreach. Re-label subreaches that are placed on the figure in reverse order.*

PACE Response

Subreaches have been re-labeled. Please see response to Comment 35.

Comment 54. *Figures 7.2A-D, update according to Comment 51 above and place value in the centroid of the subreach. Re-label subreaches that are placed on the figure in reverse order.*

PACE Response

Please see response to Comment 35 and 53.

Comment 55. *Figures 7.3A-B, update according to Comment 51 above and place value in the centroid of the subreach. Re-label subreaches that are placed on the figure in reverse order.*

PACE Response

Please see response to Comments 35 and 53.

Comment 56. *Page 34, Subreach SRB1, second line states that "long-term analysis has shown aggradation of 2.1 feet as the bed recovers from historic gravel mining." Table 5.1B shows a degradation of 2.1 feet and neither the cross-section nor the historical topography maps show any evidence of gravel mining in this area.*

PACE Response

The text and table have been updated to reflect this comment.

Comment 57. *Table 8, Total Degradation should be updated according to Comment 67 below.*

PACE Response

The table has been updated based on a 1-12-06 meeting with LACDPW staff to indicate include data in final design as the greater of LACH&SM and LACFCDDM criteria. The original table has been re-labeled and included in Appendix Chapter 6 as per personal communication with County staff (B. Willardson, 1-17-06).

Comment 58. *Appendix Chapter 4.1, please include output from the Existing Conditions analysis of n=0.085 that is mentioned elsewhere in the report.*

PACE Response

The data is included in the draft.

Comment 59. *Appendix Chapter 4.3 please include existing conditions n=0.025 model.*

PACE Response

The data is included in the draft.

Comment 60. *Appendix Chapter 4.4, Table A4.4, add a note stating which 'n' value analysis is being compared.*

PACE Response

The appendix has been updated to reflect this comment.

Comment 61. *Appendix Chapter 4.4, the text should be fully justified.*

PACE Response

The appendix has been updated to reflect this comment.

Comment 62. *Appendix Chapter 4.4, Tables A4.4B1-2, add a column showing the percent change from the HEC-RAS mixed flow to the HEC-RAS subcritical flow. Also add a column showing the total percent change from HEC-RAS mixed to the HEC2 subcritical import.*

PACE Response

The table has been updated to reflect this comment.

Comment 63. *Appendix Chapter 4.5, Tables A4.5A1-4, update according to Comment 22. Also add a comment in the title of each table stating which reach length it is for. Correct the spelling of "Conditions" in each table title and "significantly" in the footnote. See also Comment 23.*

PACE Response

The table has been updated to reflect this comment. Also, please see response to Comment 23.

Comment 64. *Appendix Chapter 4.5, Tables A4.5B1-4, update according to Comment 22. Also add a comment in the title of each table stating which reach length it is for. Correct the DS Stations in the first table for SRB4 and in the last table for SRA1 and SRB2. Correct the spelling of "Stability" in each table title and "significantly" in the footnote. See also Comment 23.*

PACE Response

The table has been updated to reflect this comment. Also, please see response to Comment 23.

Comment 65. *Appendix Chapter 4.5, Tables A4.5C1-4, update according to Comment 22. Also add a comment in the title of each table stating which reach length it is for. Correct the Subreach column of table C2 to make it consistent with the others. See also Comment 24.*

PACE Response

The table has been updated to reflect this comment. Also, please see response to Comment 24.

Comment 66. *Appendix Chapter 4.5, Tables A4.5E1-4, labeling is non-sequential unless Table A4.5D was not included with the report. Also add some commentary to this page explaining what we are looking at. Convergence is not evident. Existing Reach A and Proposed Reach B appear to diverge with smaller subreach lengths.*

PACE Response

The table has been updated to reflect this comment. Also, please see response to Comment 23.

Comment 67. *Appendix Chapter 6.2, all Tables, add a column after the Z_{gs} to show the SAM General Bed Adjustment value for each cross-section. Also add another Z_{tot} column that replaces the Z_{gs} contribution with the SAM General Bed Adjustment. Correct Z_{deg} according to Comment 50 and correct the value for Section 2627 which is included in subreach SRB2 (2.8 instead of 2.0). Update the footnote reference to "Table 5.x."*

PACE Response

The tables have been updated based on a 1-12-06 personal communication with County staff (B. Willardson, 1-17-06). The updated includes a maximum column to indicate the greatest toe-down value. Additionally, a separate table has been included to show SAM values in place of LACH&SM estimates of general adjustment.

If you have any questions regarding the above responses, please do not hesitate to contact us.

Sincerely,
PACIFIC ADVANCED CIVIL ENGINEERING, INC.

David A. Jaffe, PhD, PE
Project Engineer

PACIFIC ADVANCED CIVIL ENGINEERING, INC.

17520 Newhope Street, Suite 200 ▪ Fountain Valley, California 92708 ▪ 714.481.7300 ▪ fax: 714.481.7299

November 22, 2005

Mr. Ben Willardson
Water Resources Division
Los Angeles County Department of Public Works
900 S. Freemont Avenue
Alhambra, CA 91803

Re: Response to Land Development Comments for the Review of Newhall Ranch Castaic Creek Fluvial Analysis SAM Model and Long-Term Topography Review dated June 6, 2005 **#8065E**

Dear Mr. Willardson:

Pacific Advanced Civil Engineering, Inc. (PACE) is pleased to provide the following response to the Land Development Division review comments on the Fluvial Analysis. LADPW comments on the Fluvial Analysis are addressed in the submittal of the "Valencia Commerce Center – Castaic Creek Fluvial Study Report" dated September 19, 2005, prepared by PACE.

Previous Comments

Comment 1. *Figure 1.1, No bank protection is shown on the northwest bank as the creek turns to go under the SR-126 bridge. The longitudinal profile traces for this portion of the river show that for larger events, the river cuts the existing agricultural land just upstream so that it can make the turn through the bridge. Once that is constrained, it is likely that the river will erode the northwest bank upstream of the bridge opening. David has responded that the river analysis shows that there is no increase in velocities through this reach from existing to proposed conditions. This may be the result of not having erosion shown in the existing versus proposed condition HEC-RAS cross-sections. The existing HECRAS cross-section may not show the actual flow area that occurs under existing conditions since the cross-section is replaced each time it fails. It may also be related to the fact that HEC-RAS cannot model curved reaches. More analysis is required to determine bank protection needs.*

PACE Response

The text in Chapter 5 has been updated to include the following:

LACDPW has noted that they believe the placement of the soil cement on the banks of Castaic Creek upstream of Highway 126 may impact the abutment on the northwest bank. This belief was partially predicated by the partial erosion of agricultural fill observed by LACDPW personnel following large discharge events. No data pertaining to any erosion of fill is available at the time of writing. LACDPW believes that the erosion of fill is related to the flow of the Creek trying to maintain a large-radius bend from the Creek into the Santa Clara River confluence. LACDPW has noted concern that if the agricultural fill is protected behind the proposed soil cement bank protection that the flow of the Creek will be directed at the northwest bank adjacent to the Highway 126 Bridge abutment causing erosion that will eventually outflank the abutment and thereby cause failure of the bridge. In previous drafts of this report HEC-RAS numerical modeling was included that compared the existing and proposed conditions water surface elevation and velocity. The modeling indicated that no significant change in water surface elevation or velocity was expected. Another version of the same model was included that modified the existing condition so that erosion of the northwest bank fill was accounted for by removing 50 feet of fill for approximately 2500 feet of channel length. This modeling found no significant changes between the existing-eroded condition and the proposed condition water surface elevation and velocity. LACDPW indicated that they believed the one-dimensional nature of HEC-RAS made it insufficient to address the velocity and water surface elevation changed between the existing and proposed conditions.

To resolve the issue of the protection of the Highway 126 Bridge abutment an extension of the abutment will be constructed. This extension, shown in Figure 1.1, will continue from the existing abutment to the northwest and tie in to existing Saugus formation sandstone. Allan Seward Engineering Geology has conducted a preliminary investigation of the soils in the area and indicated that there is most likely sufficient sandstone in the vicinity of the Bridge to complete the tie-in of the abutment extension. The draft report is included in Appendix Chapter 5. A final draft of the Seward report will be made available prior to construction to verify the location of sandstone deposits. The extension of the abutment will be buried following construction and will only be utilized if the fill material covering the extension is eroded by hydraulic action of the Creek. The topography of the area at the tie-in is sufficiently steep to ensure that the extension will not be out flanked by Creek flow (See Exhibit). The Saugus sandstone is a moderately cemented material that is much less erodible than the fluvial deposits that form the creek bed. This resistant material can be seen at section 9299, as noted above, where cross-section data indicates that exposed Saugus sandstone has been persistent for the period of record. Communications with LACDPW's Geotechnical & Materials Engineering Division (Charles Nestle, personal communication, 11/2/05) has concurred with this analysis. To address LACDPW concerns as to the curvature of the existing-eroded condition, a curved path based on the 1947 historical thalweg was used for existing conditions calculations in Chapter 6 below. This path and its radius are illustrated in Appendix Chapter 6.

Comment 4. *Page 4, Section 2.2 discusses the Simons, Li, and Associates report for the Santa Clara River and major tributaries. The first paragraph states that sediment gradation data is not available. However, as discussed for the Santa Clara River, it is available in the copies of the report provided to PACE and in the Public Works technical library.*

PACE Response

The copy of the report provided by the County is illegible with respect to identifying the locations of sample sites. Boring and gradation data are present, however, the position data in its current state is unusable. Approximate data locations have been noted. The text has been updated to reflect this response.

Comment 8. *Page 10, more discussion needs to be added about the conversion process of HEC-RAS to SAM input data. More discussion is needed on the method of analyzing the stream reach in SAM using the reach length specification from the subreach cross-section to the cross-section just downstream of the next subreach cross-section. (Revised in response to comments, but not actually incorporated in report). The discussion states that the Sikand model was used for HEC-RAS with minor changes in flow rates. This no longer applies. The model has been changed and should be considered a PACE model now. The PACE model will eventually be accepted, but the Sikand model still has unresolved problems.*

PACE Response

The text has been updated to incorporate this change.

Comment 25. *Page 20, more discussion needs to be added on the agricultural practices in the Castaic Creek reach being analyzed. The river reclaims large portions of agricultural fill every time a large flood occurs. This is evidenced by the 1999 thalweg on top of the 2004 aerial photography. This spring, field reconnaissance showed that the river moved back towards the position shown as the 1999 thalweg. The agricultural area was being replaced this spring. This is the area for one of the projects on Castaic Creek and needs to be analyzed more carefully.*

PACE Response

Please see response to Previous Comment 1, above.

Comment 26. *Table 5.1 needs to be updated to include the elevation data for each of the 5 traces at each cross-section along the longitudinal profile and the median and mean of these values. The median and mean of all of these values can then be determined to show what the average elevation has been over the long-term. The current method of analysis only utilizes 2 of the 5 traces and is not consistent with the analysis of average cross-section elevation as discussed in June. Instead of cross-sections, PACE was allowed to create longitudinal traces, to expedite the work. In order for these traces to provide comparable information, all traces must be used in the averaging process. The profiles need to be analyzed to show if the bed is oscillating or if it is continuing to aggrade or*

degrade. This analysis needs to be discussed in the document. (This work was completed, but resulted in new problems). Please see comments in the new comments section.

PACE Response

In the current draft of the report the historic thalweg analysis has been replaced with a historic cross-section analysis. The sections provide a more clear and complete picture of the trends of the creek bed in a given section. An analysis of the trends of the various sections is included in Chapter 5. This analysis separagely considers the periods prior to and following the completion of the upstream dam in 1971. The data from the thalweg analysis is included for completeness in Appendix Chapter 5.

Comment 35. *Page 26, the HEC-RAS model submitted by Sikand was never approved and must be reviewed for use with both SAM and HEC-6. (Update text to reflect response to comment)*

PACE Response

The text has been updated to reflect this change.

Comment 39. *Page 27, the second paragraph states that it is impossible to know the affect of agriculture on the long-term channel behavior due to a short record. The record is sufficiently long to indicate that agricultural infilling of the river has occurred and at times causes problems within the river system. More discussion is needed on this topic.*

PACE Response

Please see response to Previous Comment 1, above.

Present Comments

Comment 1. *Page 12, the last sentence of the third paragraph states that calculations considered outliers for the sediment transport capacity are discarded. More information needs to be provided on how the outlier is determined. Another sentence discusses physical viability of the transport capacity. More discussion on viability determination is required.*

PACE Response

The text has been updated to include the following:

In some simulations and with some transport functions SAM analysis provides transport potential calculations three or more orders of magnitude higher or lower than the median. This is because the transport function employed may have been based on sediment and hydraulic parameters very different than those examined in the present study despite being selected by SAM.AID. These values have been rejected and generally the median value of transport is employed.

Comment 2. *Page 17, an outline for the HEC-6 study is being presented on this page. Section 3.2.1 discusses conversion of HEC-RAS models to HEC-6 models. The approved*

HEC-RAS model of Castaic Creek needs to be used for the conversion. The section also states that the County will provide hydrologic data for all modeling in the form of hydrographs. Public Works provided design storm peak flows for the 2-, 5-, 10-, 25- and 50-year events. The hydrographs associated with these modeled events can be provided to PACE. However, these do not represent flow rates on the Creek since they are design storms with design storm assumptions including a high antecedent soil moisture condition, evenly distributed rainfall, etc… PACE needs to submit a plan on how to provide long-term hydrographs for Castaic Creek.

PACE Response

As per a personal communication of 10/17/05 with Ben Willardson of the County, any and all HEC-6 related comments and responses will be delayed until the second phase of the fluvial analysis following the approval of the Tentative Tract Map(s).

Comment 3. *Page 17, other comments on the HEC-6 study are as follows: 3.2.3 The study plan calls for assigning roughness coefficients based on aerial photography and site visits. Typically, roughness coefficients are set during the fixed-bed water-surface calibration of the model. 3.2.4 Sediment properties of the bed should be based on the bed sampling program conducted for Castaic Creek by Seward. 3.2.5 Setting sediment inflow at the upstream boundaries will be challenging. Using SAM calculations in a stable alluvial reach upstream from the boundary as suggested in the PACE outline will be a good first approximation. Be sure to use the same sediment transport equation that is used in the HEC-6 model. Another option is to use iterative HEC-6 sediment transport calculations at the upstream five cross sections. 3.2.6 The model will simulate the development of an armor layer. You should not have to externally calculate a bed-sediment reservoir depth in armored reaches. However, the initial bed gradation in armored reaches should be coarser than in unarmored reaches. 3.2.7 and 3.2.10, The hydrograph in HEC-6 consists of a series of steady state discharges with a specified duration. For lack of a better term, this is referred to as a histograph. The durations may be relatively long at low discharges and relatively short at high discharges and when the discharge is changing rapidly between events. Numerical instability in the explicit solution of the Exner equation can be eliminated by reducing the time step duration. This may be required during the calibration of the model. For convenience, the mean daily flow would probably be used as a maximum time step. At higher discharges durations may need to be reduced to 15 minutes or less.*

PACE Response

Please see response to Present Comment 2, above.

Comment 4. Page 18, More information needs to be provided regarding changes to sediment inflow based on the development of the watershed.

PACE Response

Please see response to Present Comment 2, above.

Comment 5. The HEC-6 studies needs to include historic cross-sections for comparison during the model calibration phase.

PACE Response

Please see response to Present Comment 2, above.

Comment 6. *Page 19, The second paragraph discusses the completion of the Lake Hughes Dam in 1971 and how it has changed the system. Although the creek bed has degraded over time, the 2005 storm event has deposited a sediment layer in the creek that is 5 feet deep in the reach downstream of the Hasley Creek confluence. This section begins the analysis of long-term trends in the river. After mention of the dam, the data is analyzed as if it is a continuous record. This is incorrect. The dam changed the system and the two data sets are discontinuous. More analysis is needed to determine the trends in the creek relative to the more recent data set. Since the data is representative of a 6-year period, some uncertainty is involved. All of this needs to be analyzed and discussed. The analysis needs to include recommendations on how the observed data will affect freeboard and scour calculations and these influences need to be included in* Z_{DEG} *and* Y_{AGG} *calculations.*

PACE Response

Please see response to Previous Comment 26, above.

Comment 7. *Page 20 discusses the fact that the Santa Clara River has been in a state of equilibrium since 1947. Looking at the tables updated for the revised calculations of mean bed elevation, this statement may not be accurate. The analysis needs to be revisited with the latest calculations of mean bed elevation from the topographic traces.*

PACE Response

Please see response to Previous Comment 26, above.

Comment 8. *Figures 5.3A-B should not cover a vertical scale of 800 to 1100 feet since the elevation only ranges from 930 to 1050 feet. Reducing the vertical range will make the yearly changes more apparent and the scale easier to interpret graphically. The aggradation/degradation trends in Figure 5.3B need to be addressed, especially the period after 1971. The spike at station 9671 should also be explained.*

PACE Response

Please see response to Previous Comment 26, above. The scale of the figure has been adjusted.

Comment 9. *Page 23, The special note on this page should include the discussion requested in comment #1 from the previous comments and address the potential impacts to the CalTrans right-of-way.*

PACE Response

Please see response to Previous Comment 1, above.

Comment 10. *Page 24, The analysis shown in Table 5.4 is not the analysis required to determine the effects of erosion on the CalTrans right-of-way upstream of the Highway 126 Bridge. HEC-RAS does not compute effects of curved reaches. The tables in Appendix 6 show scour values ranging from 9 to 18 feet between stations 1982 and 1462. Since no lining is to be placed on the bank in this area, the results of this scour must be quantified, and Capital Flood protection must be provided for CalTrans.*

PACE Response

Please see response to Previous Comment 1, above.

Comment 11. *Page 25, the values in Table 5.5 do not match the mean values shown in Table 5.1. The values appear similar to the minimum topographic value for the 2004 and 2005 data. This table should use the mean data for the years instead of the minimum bed elevation for each year. Table 5.5 and Figure 5.4 do not represent the values shown in Table 5.1 that there is only degradation occurring in Castaic Creek. More analysis and discussion of these tables needs to be included in the report.*

PACE Response

Please see response to Previous Comment 26, above.

Comment 12. *Page 29, references are implied in the text, but no reference page is found in the report. Please add a reference page.*

PACE Response

A reference page has been included in the report.

Comment 13. *Page 32, this page states that freeboard ranges from 2.5 to 4.5 feet and that the data is found in Appendix 6. The tables in Appendix 6 only show freeboard of 2.5 to 2.8 feet. The freeboard needs to include deposition of sediment. More analysis of the deposition downstream of Hasley Canyon needs to be added and the results need to be reflected in the report and appendices. Downstream of Hasley, deposition of 5 or 6 feet occurred between 2004 and 2005. Freeboard needs to include this possibility. The models currently use the 1999 bed elevation, which is higher than the 2004 bed elevation and lower than the 2005 bed elevation. When the channel width is reduced by the proposed project, the volume of sediment may cause changes in depth that exceed what was experience during the last storm season if velocities are not high enough to move sediment out of the reach. Again, more analysis and discussion in the report is needed.*

PACE Response

Please see response to Previous Comment 26, above.

Comment 14. *Page 33, the effects on CalTrans and any mitigation measures need to be discussed in the summary.*

PACE Response

Please see response to Previous Comment 1, above.

Comment 15. *Page 34, the second paragraph states that the Yang equation was best suited for the study. However, earlier comments and the models seem to indicate that the Meyer-Peter and Muller equation is preferred.*

PACE Response

The typographical error has been corrected in the text.

Comment 16. *Page 34, the last paragraph needs to discuss the discontinuous nature of the topographic data for use with sediment transport calculations. The discontinuity is cause by the construction of the dam on Castaic Creek.*

PACE Response

The following text has been changed to read the following:

> Apparent degradation is expected to be caused by the construction of the dam upstream of Interstate 5, which was completed in 1971. It is important to recall that the analysis of long-term sediment fluctuation is conducted using discrete time increments. Any abrupt changes in bed elevation between the 1963 and 1999 topographies are likely the result of the completion of the dam.

Comment 17. *Page 35, the third and fourth paragraphs indicate that the existing model was modified to reflect 50 feet of erosion on the western bank, but it was actually modified on the northern bank before the river curved south under the Highway 126 bridge. The top width of the channel for the existing condition is approximately 1000 feet wide at this location and so no change was noted. The proposed width is approximately 500 feet, and changes should be noted. However, this is not the portion of the riverbank that requires analysis, since it will be lined. The analysis is needed between stations 1982 to 1462 where the CalTrans easement is currently unprotected.*

PACE Response

Please see response to Previous Comment 1, above.

Comment 18. Page 35, the fourth paragraph states that aggradation ranges from 0 to 4.6 feet. This needs to be reflected in the various portions of the text already mentioned.

PACE Response

Please see response to Previous Comment 26, above.

Comment 19. *Page 36, the effect of the armoring layer is discussed related to toe-down depths. The armoring layer does not affect the required toe-down depths since construction may displace or disrupt the balance and leave the lining vulnerable to scour.*

PACE Response

Armoring layer calculations have been removed from this draft of the report because it is believed that armoring will be reflected in HEC-6 modeling, if present, in phase two of this report.

Comment 20. *Preliminary review of the HEC-RAS to HEC-2 conversion showed that there were discrepancies between the HEC-RAS model submitted for review and the HEC-2 model. The HEC-2 model is used as input to SAM and eventually to HEC-6. The discrepancies included 10 to 20 percent differences in top-width, depth, velocity, etc… at many crosssections. The text indicates that the conversion of HEC-RAS to HEC-2 is a powerful tool, but that there are things that need to be corrected during the conversion process. The models need to be checked and discrepancies need to be explained since the discrepancies were not consistent at each cross-section. The HEC-2 model will be compared against the approved model. Comments on the HEC-RAS model were provided on October 5, 2005.*

PACE Response

HEC-RAS and HEC-2 employ different numerical schemes to calculate hydraulic parameters. The schemes are described in the user's manuals for each program. These differences result in variations in hydraulic output between the two models, even for the same geometry and hydrology. Moreover, the conversion process, as described in the text, notes that the number of stations in each cross-section must be reduced to no more that 100 points, and fewer if ineffective flows or levees are present in a given section, and bridge removal. The removal of stations is both a manual process (in the overbank areas) and an automated one (minimal area change method in the HEC-RAS geometry editor), although several methods can be utilized by a modeler. Finally, HEC-2 requires that bank stations be located at levees and ineffective flow areas, unlike HEC-RAS, and SAM modeling includes no bridges. In any case, the changes in geometry resulting from the requirements of the HEC-RAS to HEC-2 process will introduce even more hydraulic variations between models, which prevent direct hydraulic comparison. It is not expected that these hydraulic will significantly alter the calculation of total potential transport. The text has been updated to reflect this comment.

Comment 21. *Analysis of the SAM sediment input data shows that there is still a problem with the sediment size distribution in Reach B. No other analysis on SAM is possible until the HEC-RAS and HEC-2 conversions are checked as discussed in comment #20.*

PACE Response

The numerical error noted is a product of conversion of HEC-2 to SAM. The error has been corrected and the results updated accordingly.

If you have any questions regarding the above responses, please do not hesitate to contact us.

Sincerely,
PACIFIC ADVANCED CIVIL ENGINEERING, INC.

David A. Jaffe, PhD, PE
Project Engineer

PACIFIC ADVANCED CIVIL ENGINEERING, INC.

17520 Newhope Street, Suite 200 ▪ Fountain Valley, California 92708 ▪ 714.481.7300 ▪ fax: 714.481.7299

September 19, 2005

Mr. Ben Willardson
Water Resources Division
Los Angeles County Department of Public Works
900 S. Freemont Avenue
Alhambra, CA 91803

Re: Response to Land Development Comments for the Review of Newhall Ranch Santa Clara River and Castaic Creek Hydraulic Analysis Dated August 23, 2005 **#8065E / #8197E**

Dear Mr. Willardson:

Pacific Advanced Civil Engineering, Inc. (PACE) is pleased to provide the following response to the Land Development Division review comments on the Hydraulic Analysis. LADPW comments on the Hydraulic Analysis are addressed in the submittal of the "Newhall Ranch-Santa Clara River HEC-RAS Modeling Report and the Valencia Commerce Center – Castaic Creek HEC-RAS Modeling Report" dated September 2005, prepared by PACE.

General Comments:

1. *Comment- A total of three hydraulic models shall be submitted for each river: a) Manning's n-value of 0.085 with proposed bank improvements to determine the top of the levee elevations; b) Manning's n-value of 0.025 with proposed bank improvements to determine velocities for toe down scour depth calculations; c) Manning's n-value of 0.060 with proposed bank improvements for floodway and floodplain delineation.*

   PACE Response:

   The following models are included in both the Santa Clara River and Castaic Creek submittals on CD and in the Appendices, and are listed below for each project:

   Santa Clara River:

   Existing Conditions n = 0.060 (Appendix A)
   Proposed Conditions n = 0.060 (Appendix B)
   Existing Conditions n = 0.085 (Appendix I)
   Proposed Conditions n = 0.085 (Appendix C)
   Proposed Conditions n = 0.025 (Appendix D)
   Proposed Conditions n = 0.025 with interpolated Cross Sections (Appendix E)

Castaic Creek:

Existing Conditions n = 0.060 (Appendix A)
Proposed Conditions n = 0.060 (Appendix B)
Existing Conditions n = 0.085 (Appendix I)
Proposed Conditions n = 0.085 (Appendix C)
Proposed Conditions n = 0.025 (Appendix D)
Proposed Conditions n = 0.025 with interpolated Cross Sections (Appendix E)

2. *Comment- Provide a topographic map showing all cross-section locations.*

PACE Response:

A topographic work map showing all cross section locations as well as ineffective flow areas and floodplain lines have been included in the Castaic Creek and Santa Clara River HEC-RAS Modeling Reports. See Exhibit 1 – "Existing condition Topo with HEC-RAS sections" and "Exhibit 2 – Proposed condition Topo with HEC-RAS sections."

3. *Comment – Due to problems with the hydraulic models, top of levee elevations, scour depths, and revised floodway delineation were not reviewed at this time.*

PACE Response:

As previously discussed during our August 23, 2005 meeting, your comment text "floodway" is incorrect and should be replaced with "floodplain." The current submittals are for HEC-RAS Floodplain Analyses for both the Santa Clara River and Castaic Creek Proposed Projects. Adequate models are now included to evaluate:

a) Top of Levee – See Table 2.0, 2.1 and Appendix C in both reports.
b) Scour Depths – See Table 3.0, 3.1 and Appendix D in both reports.
c) Floodplain delineation – See Table 1.0 and Appendix A&B and Exhibits 1 and 2.

Santa Clara River:

1. *Comment – Flows were overtopping several user-defined cross-sections. Cross-sections should be extended to fully contain the flow.*

PACE Response:

All cross sections have been extended to fully contain the flow in the river. There is an exception in the Castaic Creek model for cross sections below Hwy 126 where the creek and the river confluence. For evaluation of the Creek within the River, the HEC-RAS model still includes artificial levee markers as the model would be incorrect if the cross-sections were extended.

2. *Comment – The current analysis yielded water surface elevations defaulting to critical depth. Additional cross-sections and shorter reach lengths should be used to reevaluate the channel flow depths. HEC-RAS is a steady gradually varied flow model and additional cross-sections are necessary for calculating steady flow water surface profiles.*

   PACE Response:

   See Appendix E of the Santa Clara River Report. This Appendix has been included to show that for the proposed n=0.025 model, the cross sections indicating critical depth have been verified as correct. The verification has been accomplished by modifying the n=0.025 Proposed Condition Model (Appendix D) with cross-sections interpolated every 50 feet in order for the model to be able to determine an accurate water depth. This analysis showed that even with the interpolated cross sections every 50 feet, critical depth still occurs along the river. Interpolated model results as shown in Appendix E verifies original model results for the Proposed n=0.025 model, verifying critical depth does occur in the river and creek (Appendix D).

3. *Comment – Additional cross-sections are needed to model the bridge crossings properly. The bridge routine uses four user-defined cross-sections in the computation of energy losses. Refer to the HEC-RAS Hydraulic Reference Manual pages 5-2 to 5-5 for cross-section locations.*

   PACE Response:

   The three proposed structures, Commerce Center Bridge, Long Bridge, and Potrero Bridge have been modeled as blocked obstructions. See Santa Clara River HEC-RAS Floodplain Analyses exhibits and proposed HEC-RAS models for locations and details of how the structures were modeled.

4. *Comment – Two bridges were modeled using the multiple block obstructions feature. Verify that the computed water surface elevation is below the bridge deck low chord elevation. Otherwise, the bridge would have to be modeled using the bridge editor to model flow through the bridge. To properly model the bridges refer to the HEC-RAS Hydraulic Reference Manual pages 5-2 to 5-5 for cross-section locations.*

   PACE Response:

   As previously mentioned, there are no structures modeled as bridges, but rather as blocked obstructions. The computed water surface elevations for all three proposed structures (Commerce Center Dr., Long, & Potrero) are well below the bridge deck low chord. See Santa Clara River HEC-RAS Floodplain Analyses for table which shows amount of freeboard for each bridge. For the Santa Clara River model, preliminary bridge information is included in Appendix H.

5. *Comment – Ineffective flow areas should be used to define areas of the cross-section that will contain water that is not actively being conveyed, specifically at bridges, contractions, and expansions.*

   PACE Response:

   Ineffective flow areas are included in both the existing and proposed models for Castaic Creek and Santa Clara River and are shown on Exhibits 1 & 2.

6. *Comment – Proposed bank protection was not shown in several user-defined cross-sections.*

   PACE Response:

   The proposed bank protection locations shown in plan on the exhibits have been verified and the proposed banks have been properly located and modeled.

7. *Comment – Artificial levees may be used to restrict flow splitting within a cross-section.*

   PACE Response:

   Ineffective flow areas or artificial levees have been reevaluated and placed in the model at locations where split flow occurs within a cross section. See attached HEC-RAS model cross sections.

8. *Comment – For the floodway revision hydraulic model, proposed bank protection should be modeled as it will be constructed to ensure accurate floodway delineation. The current hydraulic models show vertical levees for the proposed bank protection.*

   PACE Response:

   As discussed in our August 23, 2005 meeting, the review comment should be "floodplain" and not "floodway." The proposed condition models have been revised to replace the levee markers with the 2:1 sloped proposed bank protection on the Santa Clara River.

Castaic Creek:

1. *Comment – Due to confluence with Santa Clara River, the downstream boundary condition for Castaic Creek should be set using the known water surface elevation at the confluence point from the Santa Clara River hydraulic model.*

   PACE Response:

   The downstream boundary condition for the Castaic Creek model was taken from the Santa Clara River with maximum water surface elevation from the n=0.085 proposed project model at Santa Clara River station 30445 with a water surface elevation of 961.4. In addition, we have provided Appendix H, which includes Proposed Condition Castaic Creek Model with a normal depth downstream starting water surface of 949.5. For each model, the water surface upstream of the Hwy 126 Bridge is the same 966.6 even though the starting water surface is different by nearly 12 feet. This concludes that the Creek is hydraulically independent from the river upstream of Hwy 126.

2. *Comment – The current analysis yielded water surface elevations defaulting to critical depth. Additional cross-sections and shorter reach lengths should be used to reevaluate the channel flow depths. HEC-RAS is a steady gradually varied flow model and additional cross-sections are necessary for calculating steady flow water surface profiles.*

   PACE Response:

   See Appendix E of the Castaic Creek report. This Appendix has been included to show that for the proposed n=0.025 model, the cross sections indicating critical depth have been verified as correct. The verification has been accomplished by modifying the n=0.025 Proposed Condition Model (Appendix D) with cross-sections interpolated every 50 feet in order for the model to be able to determine an accurate water depth. This analysis showed that even with cross sections interpolated every 50 feet, critical depth still occurs along the river. Interpolated model results as shown in Appendix E verifies original model results that for the Proposed n=0.25 model, critical depth does occur in Creek (Appendix D).

3. *Comment – The Capital Flood flow rates for Castaic Creek are 26,500 cfs upstream of the confluence with Hasley Creek and 31,400 cfs downstream of the confluence with Hasley Creek. These flow rates should be used for all Castaic Creek model.*

   PACE Response:

   The capital flood flow rates have been updated to match for both the existing and proposed conditions models.

4. *Comment – Bridges should span the entire width of the cross-section. Abutments should be modeled using the abutment editor and not as part of the deck or roadway.*

   PACE Response:

   The Commerce Center Dr. Bridge where this problem occurs has been updated to properly model the abutments in the abutment editor.

5. *Comment – Artificial levees may be used to restrict flow splitting within a cross-section.*

   PACE Response:

   Ineffective flow areas or artificial levees have been reevaluated and placed in the model at locations where split flow occurs within a cross section. See attached HEC-RAS model cross sections.

6. *Comment – For the floodway revisions hydraulic model, proposed bank protection should be modeled as it will be constructed to ensure accurate floodway delineation. The current hydraulic models show vertical levees for the proposed bank protection.*

   PACE Response:

   As discussed in our August 23, 2005 meeting, the review comment should be "floodplain" and not "floodway." The proposed condition models have been revised to replace the levee markers with the 2:1 sloped proposed bank protection on Castaic Creek.

If you have any questions regarding the above responses, please do not hesitate to contact us.

Sincerely,
PACIFIC ADVANCED CIVIL ENGINEERING, INC.

Mark E. Krebs, P.E.
President

MEK/db

Creek Fluvial Study Phase 1 Final Draft

Castaic Creek

January 20, 2006

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#8065E

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Executive Summary

This technical study provides an evaluation of the existing and proposed fluvial characteristics and long-term stability of Castaic Creek between Interstate 5 and the Santa Clara River. Development along the Creek within Valencia Commerce Center will potentially modify the fluvial mechanics of the Creek. The proposed buried soil cement bank protection on both the east and west banks of the Creek is intended to provide long-term erosion protection from lateral migration of the bank and flood protection for the adjacent proposed development areas. This analysis evaluates impacts from build-out of Valencia Commerce Center from (1) fluvial modifications of the creek bed from single hypothetical storm events, and (2) changes in the floodplain fluvial operation over the long term.

Three more recent sediment transport studies have been conducted within the study reach between Interstate 5 and the Santa Clara River (Chapter 2). The first study was conducted by Simons, Li & Associates (SLA) and sought to provide information for flood and fluvial process management in Santa Clara River from Solidad Canyon to Castaic Creek. The study is comprised of multiple tasks including data collection, quantitative and analysis and mathematical modeling, alternatives evaluation and other assignments. A second study, also produced by SLA, provided fluvial analysis of lower Castaic Creek including sedimentation analysis. The intent of the study was to provide technical data in support of design criteria. A third study prepared by Tetra-Tech conducted bed scour analysis for proposed bridge widening at Interstate 5.

Sediment collection for Castaic Creek along the study reach was conducted by Allan Seward Engineering Geology, Inc. (Chapter 3, Appendix Chapter 3). Twenty samples were collected at eight different locations positioned along River subreaches. All sampling was conducted using grab samples of the upper five feet of the active or recently active portion of the bed. No fine material is included in the sediment analysis because fine material is generally transported as wash load. A review of the raw gradation curves indicated that most samples were comprised of poorly graded sands and gravels with cobbles. The D_{50} values for all samples ranged from 0.46 to 4.24 mm (Figure 3.1). Additionally, previous studies noted above also found similar characteristics, although specific distribution data is not presently available.

Modifications to the creek bed are measured as bed adjustment in feet. Positive adjustment indicates aggradation and negative adjustment indicates degradation. Several types of adjustment are considered in this study including general adjustment, long-term adjustment, and other scour. General adjustment is scour that occurs in an individual discharge event, and is calculated as the difference between sediment inflow and outflow of a given River reach. Long-term adjustment consists of fluvial processes that occur over several years. Other scour is made up of local scour, bend scour, low-flow incisement, and bed form formation.

General adjustment was estimated in this study using the US Army Corps of Engineers (ACOE) SAM steady-state zero-dimensional numerical model (Chapter 4). SAM is utilized to provide a first approximation of sediment transport potential for subreaches within Castaic Creek. The SAM numerical model is built upon hydraulic and fluvial representations of the study bed. The hydraulic component includes representations

of bed characteristics and discharge. The fluvial component includes representation of bed gradation and sediment transport functions. SAM's hydraulic component utilized average cross-section data imported from HEC-2 numerical models of the river down-converted from HEC-RAS numerical models. Both the existing and proposed conditions HEC-RAS models were originally developed by Sikand Engineering in the *Newhall Ranch Castaic Creek HEC-RAS Study* (June, 2000) and submitted as part of the Newhall Ranch Specific Plan. The HEC-RAS model that is the basis of the SAM modeling has been refined by PACE and has been submitted (concurrently with this Fluvial Study) to LADPW for review and approval. The current model resembles the Sikand model but has been refined to address current LACDPW modeling standards and updated proposed project conditions. River subreaches that make up the SAM model are determined by examining the hydraulic parameters of the individual HEC-RAS cross-sections and identifying correlations between those hydraulic parameters and the longitudinal position in the channel as described in 4.1.2.

Representation of sediment grain size distribution in SAM is percent finer data obtained from sieve analysis of channel sediment samples (Chapter 3). At each sample location, multiple samples are collected and the average data is input into the model. Sediment transport equations used in all SAM modeling were chosen with the assistance of the Army Corps' SAM.AID subroutine (Chapter 4). The SAM.AID subroutine determines the most representative transport function based on the hydraulic parameters and percent finer data by comparing model data with peer-reviewed sediment transport studies. The study found that MPM was the representative transport function for all subreaches for both existing and proposed conditions because it produced adjustment values within physical reason.

SAM was run for all River reaches and bed stability and was estimated based on the change in potential transport between adjacent channel subreaches for the Q_{CAP} discharge. General adjustment based on SAM modeling is presented in Chapter 4. No pattern of aggradation/degradation is apparent in the model results. General adjustment was also calculated using the equation specified in the Los Angeles County Hydrology and Sedimentation Manual (LACH&SM). The LACH&SM general adjustment calculation is based only on flow mean velocity. Modeling indicates that SAM general adjustment ranges from -3.3 to +2.4 for the proposed condition (Table 4.3). No differences greater than one foot between the existing and proposed conditions general adjustment were calculated since the majority of the length of the proposed bank protection does not alter the hydraulics. The exception to this being at the Commerce Center Drive Bridge where degradation increased by 2.0 feet because of the channelization of the Creek in the proposed condition (Table 4.3). Finally, an overall trend in general adjustment for the study reach as indicated by SAM modeling is not apparent for either the existing condition or proposed condition. These depths may be further reduced (become more shallow) due to the likely proximity of the existing bed to the armoring layer, as indicated by historical data presented in Chapter 5.

Long-term adjustment was calculated based on historical records in the form of topographic data. Topographic data dating from 1930, 1947, 1963, 1999, 2004 and 2005, was digitized. Cross-sections were cut at the locations of select HEC-RAS sections for each historical topography. At least one cross-section was chosen for each subreach established in Chapter 4. Areas of the 1999, 2004 and 2005 cross-

sections (sections) are calculated and these areas are used to calculate the average change in bed elevation over time (Chapter 5). The other topographies are not used to calculate average change in bed because the trends in bed change that occurred during these years occurred prior to the construction of the Dam upstream of the project site. Several events within the available historical record (1930 to present) have had an impact on the Creek bed and fluvial mechanics. These events include construction of the upstream dam, construction of bridges spanning the Creek, agricultural infill along the Creek banks, gravel mining of the Creek bed and banks, and periodic burning of surrounding vegetation during forest fires. The most significant historical event in the formation of the present bed condition was the completion of the Creek's Dam and subsequent formation of Castaic Lake. Within the project reach the construction of the Dam appears to have resulted in the continual degradation of the bed. The sectional analysis finds that some historical sections (9299, 8050) show almost no discernible change between 2004 and 2005 suggesting an armored condition of the bed below which the bed will not degrade. Between 1999 and 2005, 0.7 and 0.6 feet (Figure 5.2), respectively, of aggradation appears to have occurred on these sections. The reason for this aggradation is unclear, however, it appears that the bed is recovering from historical gravel mining operations. Sections 6624 to 1531 show continuous degradation over the period of record from 1963 to the present which was likely the result of the completion of the upstream dam (Figure 5.2). The small change in average bed height between 2004 and 2005 for sections 6624 to 1982 suggests the bed is at or approaching the armoring depth. In these sections continued channelization and incisement of the bed is most likely caused by the construction of the dam upstream as described in Chapter 5. Additionally, agricultural activities occur in these sections so it is possible that some of the channelization may have resulted from these activities. It is important to note, however, that agricultural activities were commenced prior to the construction of the dam and data from pre-construction times does not appear to indicate incisement. The two downstream sections, section 1645 and section 1531, both aggraded between 2004 and 2005 by 1.6 and 0.4 feet on average, respectively (Figure 5.2). While it is unclear why the observed aggradation occurred it is presently believed to be the result of the fires of Summer, 2004 and the heavy rains of the 2004/2005 rainy season. This combination had the potential to produce high sediment runoff loading into Hasley Canyon Creek, which confluences with Castaic Creek at Commerce Center Drive.

To determine what impacts the proposed improvements may have on the the existing Travel Village development at Castaic Creek/Santa Clara River confluence, a model of Castaic Creek was run with a Manning's number of n=0.085 for the Q_{CAP} discharge for the existing and proposed condition (Chapter 5). The model results show that the water surface elevation does not change more than approximately 0.1 feet between the two conditions (Table 5.2). Because the impacts of this analysis are negligible an additional model with Manning's number of n=0.025 was run for the Creek for both the existing and proposed condition. The results show that the velocities in the existing and proposed conditions do not vary more than 0.2 fps (Table 5.3).

LACDPW has noted that they believe the placement of the soil cement on the banks of Castaic Creek upstream of Highway 126 may impact the bridge abutment on the northwest bank. LACDPW believes that the erosion of fill is related to the flow of the Creek trying to maintain a large-radius bend from the Creek into the Santa Clara River confluence. LACDPW has noted concern that if the agricultural fill is protected behind

the proposed soil cement bank protection, the flow of the Creek will be directed at the northwest bank adjacent to the Highway 126 Bridge abutment causing erosion that will eventually outflank the abutment and cause failure of the bridge. To resolve the issue of the protection of the Highway 126 Bridge abutment, an extension of the abutment will be constructed. This extension will extend from the existing abutment to the northwest and tie in to existing Saugus formation sandstone. Allan Seward Engineering Geology has conducted a preliminary investigation of the soils in the area and indicated that there is most likely sufficient sandstone in the vicinity of the Bridge to complete the tie-in of the abutment extension (Appendix Chapter 5.2). The topography of the area at the tie-in is sufficiently steep to ensure that the extension will not be out flanked by Creek flow.

Other scour considered in this study is comprised of four sub-categories: local scour, bend scour, low-flow incisement, and bed form height (Chapter 6). Local scour occurs in the vicinity of flow obstructions including piers and abutments. Bend scour occurs because velocity gradients occur around curves in fluvial systems. Four distinct bends are located in the study reach. Low flow incisement is included to represent thalweg or low flow channel depth. On-site inspection and review of historic data of this feature suggest a thalweg depth of approximately two feet. Finally, bed form height represents the dunes and anti-dunes that develop in active soft-bottomed channels during flow events. In this study, bed form height has been limited after Kennedy (1963). Results of calculations of bend scour vary from 0.0 to 9.2 feet (Appendix Chapter 6). Change in bend scour is observed at the bend upstream of the Santa Clara River confluence between the existing and proposed condition. For the existing condition the bed form height ranges from 0.2 feet to 3.5 feet and in the proposed condition the bed form height ranges from 0.2 feet to 4.3 feet. Local scour ranges from 0.0 to 17.6 in the existing condition and 0.0 to 16.0 feet in the proposed conditions at the various creek crossings (Appendix Chapter 6).

General adjustment, long-term adjustment and other scour are summed to determine total potential bed adjustment following LACS&HM methodology (Chapter 7). For cross-sections where SAM modeling predicts aggradation the general adjustment contribution to total bed adjustment is not included. Calculations in the existing condition predict that the combined bed adjustment ranges from approximately -5.0 to -16.1 feet for the outside of curved reaches and -5.0 to -8.8 feet for the inside of curved and straight reaches (Table 7.1). The proposed condition is predicted to have a combined bed adjustment of approximately -5.1 to -16.6 feet for the outside of curved reaches and -4.6 to -10.1 feet for the inside of curved and straight reaches (Table 7.1).

A comparison of total bed adjustment estimated by both the summed methodology and the LACFCDDM methodology shows that the more intensive LACH&SM methodology predicts a shallower toe-down for both the existing and proposed conditions than does the LACFCDDM methodology except for sections in the vicinity of Commerce Center Bridge and downstream of Interstate 5. LACH&SM methodology utilizing SAM calculations predicts a deeper toe-down than does the LACFCDDM at these locations methodology because the LACFCDDM does not account for the effects of long-term degradation as effectively (Table 7.2).

Freeboard is considered for the purposes of this report to be the additional height required above the top of a levee or other bank protection to prevent overtopping. Freeboard elevation is calculated in this study based on LACH&SM Chapter 5A-3, and includes LACFCDDM calculations. The freeboard for Castaic ranges from approximately 2.5 to 4.3 feet for curved and straight reaches (Table 7.3). Total toe-down, total freeboard, toe-down elevation and freeboard elevation based on the greater of LACH&SM and LACFCDDM criteria are presented in Table 8.

The Tentative Tract Map shows a toe-down of 15± feet, and the preliminary design report closely matches the LACFCDDM calculations. Differences between the LACFCDDM and the preliminary Design Report are a function of local effects of reach curvature on the interior and exterior arcs of the channel bends, and the Report is slightly shallower in some locations. Analysis based on the SAM modeling suggests a toe-down from approximately 5 to 16.5 feet. The difference between the SAM and LACFCDDM is expected because the large quantity of data and depth of analysis required of the SAM modeling. The depth estimated by the SAM procedure is also somewhat lower than the values on the Map. It is expected that the more intensive HEC-6 procedure will further reduce the depth of calculated general adjustment and further reduce the toe-down depth. Previous studies presented in Chapter 2 found that a toe-down of approximately 5 feet would be required for most reaches.

1 Introduction

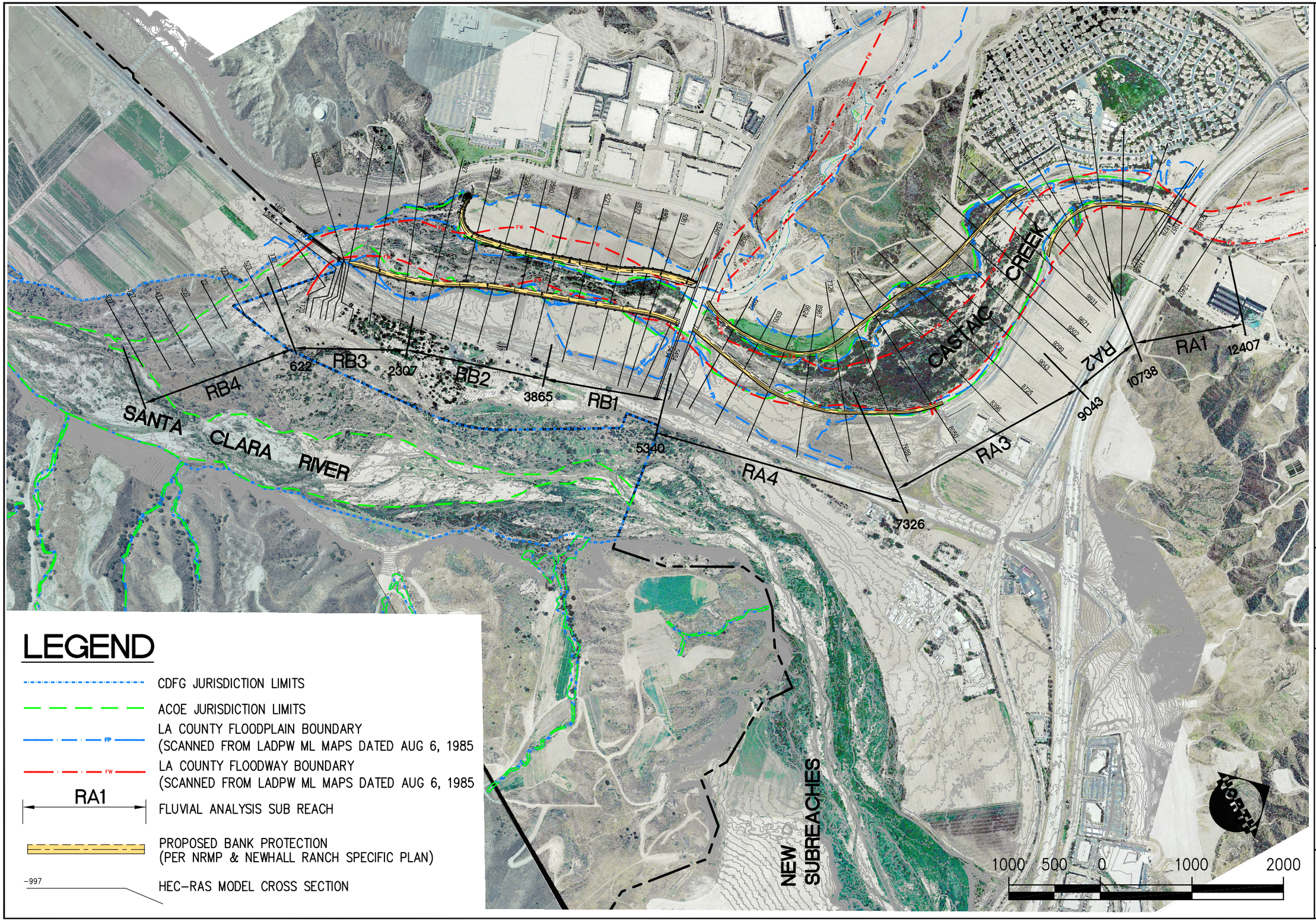
The following technical investigation provides a detailed and focused evaluation of the fluvial characteristics and long-term stability of Castaic Creek in the vicinity of tentative parcel maps 26363 and 18108. The Creek study reach is located on Newhall Ranch in northern Los Angeles County, California, between Interstate 5 and the Santa Clara River (Figure 1.1). The study reach is bounded by the River on the south and Interstate 5 on the north and confluences with Hasley Canyon Creek (Figure 1.1). Castaic Creek lies within a contributing drainage of the 202 square-mile Castaic Creek basin watershed. The existing floodplain generally consists of a natural alluvial river system extending from Castaic Lake in the north to the Santa Clara River, the Creek's natural terminus, in the south. Adjacent development along the Creek within the Newhall Ranch will potentially modify the fluvial response of the watershed through changes in the runoff and reduction in the sediment supply from the developed areas. The proposed buried soil cement bank protection on both the north and south banks of the Creek is intended to provide long-term erosion protection from lateral migration of the bank and flood protection for the adjacent proposed development areas. These modifications to the stream system may result in adjustment to the fluvial operation of the floodplain and changes to the stream mechanics. The intent of this analysis is to evaluate these impacts from (1) fluvial modifications of the riverbed from single hypothetical storm events, and (2) changes in the floodplain fluvial operation over the long term.

1.1 Types of Adjustment

Modifications to the Creek system are measured as bed adjustment in feet. Positive adjustment indicates bed aggradation while negative adjustment indicates bed degradation. Several types of adjustment are considered in this study including general adjustment, long-term adjustment, and other scour. General adjustment consists of scour that occurs in an individual discharge event, and may be considered as the difference between sediment inflow and outflow. That is, if sediment inflow into a given reach is higher than sediment outflow for the same reach, aggradation will occur. In contrast, if sediment outflow exceeds inflow for a given reach, degradation in the form of scour will occur. Long-term adjustment consists of fluvial processes that occur over many rainy seasons and contribute fluctuation of bed elevation of a river or creek. Other scour is comprised of local scour, bend scour, low-flow incisement, and bed form formation. These are discussed in detail below.

1.1.1 Study Objectives

The primary objective of this report is to develop the technical engineering analysis to assess creek bed impacts from potential modifications of fluvial operation from the proposed Newhall Ranch development. The intent is to provide a comprehensive assessment of short- and long-term bed adjustment based on the level of information available. This report provides technical analysis for (1) general adjustment, (2) long-term adjustment, (3) other scour, (4) study reach gradation, (5) SAM modeling and analysis, (6) HEC-6 modeling and analysis, (7) total soil cement bank protection toe-down and freeboard design and (8) determination of any impacts to off-site property. The



LEGEND

- CDFG JURISDICTION LIMITS
- ACOE JURISDICTION LIMITS
- LA COUNTY FLOODPLAIN BOUNDARY (SCANNED FROM LADPW ML MAPS DATED AUG 6, 1985
- LA COUNTY FLOODWAY BOUNDARY (SCANNED FROM LADPW ML MAPS DATED AUG 6, 1985
- RA1 — FLUVIAL ANALYSIS SUB REACH
- PROPOSED BANK PROTECTION (PER NRMP & NEWHALL RANCH SPECIFIC PLAN)
- -997 — HEC-RAS MODEL CROSS SECTION

TITLE

HEC-RAS
CROSS SECTION
LOCATION MAP

JOB

CASTAIC CREEK
FLUVIAL STUDY
LOS ANGELES COUNTY CA

SCALE	1"= 1000'
DESIGNED	DAJ
DRAWN	DP
CHECKED	D.J.
DATE	07/21/05
JOB NO.	8065E

PACE
PACIFIC ADVANCED CIVIL ENGINEERING
17520 NEWHOPE STREET, SUITE 200
FOUNTAIN VALLEY, CA 92708
PH (714) 481-7300 FAX (714) 481-7299

FIGURE

1.1

P:\8197E\Engineering\GlobalExhibits\8197 Castaic Creek-Fluvial Studydwg - Tab: Layout1 By: ddrahn on Nov. 03, 2005 at 12:26 pm
Xrefs: 7841E-Plan_Jurisdiction.dwg; 7841-PSR\dwg; P-bank protection_REV.dwg; 7142-Lining.dwg; 7142-FloodResource.dwg; 7142-river-sections1.dwg; NR-cdfg_2004.dwg;
7104-Santa Clara Reach Analysis.dwg; 7904-E-LACO Floodway map (12-29-04).dwg; 8197-3860-EXH-TBLK.dwg; 8197-HEC-SECS.dwg; 7142-Lining Sections.dwg; 7142-Lining...
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objectives of the fluvial assessment for the proposed development project include the following:

1. Quantify the fluvial parameters that are representative of the creek bed characteristics.
2. Model the existing and proposed conditions creek bed and fluvial processes.
3. Provide preliminary assessment of the streambed stability through determination of the sediment transport capacities within different reaches of the floodplain.
4. Provide toe-down depth and freeboard assessment throughout the study reach.

A variety of engineering analysis and tasks were associated with both the different aspects of the watershed hydrology and floodplain hydraulics. A technical framework was developed to guide the analysis of the system. These major task areas of study reflected the various objectives of the study and included the following:

1. Floodplain field investigations – Perform field reconnaissance of the existing watershed conditions as well as ground photo survey along the entire existing creek system within the fluvial study boundary. (See Appendix Chapter 3.1)
2. Baseline digital floodplain cross-section geometry – Layout appropriate spacing and location of cross-sections to establish the representative channel geometry. Digitally develop extremely accurate cross-section coordinate points using topographic digital terrain models (DTM) and CAD subroutines suitable for hydraulic model format. Adjust cross-section data to include horizontal variation of roughness and other attributes.
3. Baseline HEC-RAS hydraulic model – Prepare floodplain model in HEC-RAS based on the digital geometry and existing condition flow rates. Evaluation based on single storm event and steady flow conditions.
4. HEC-2 model creation – Conversion of HEC model formats for use in SAM modeling.
5. Floodplain reach characterization and parameter estimation – Prepare an assessment of the hydraulic parameters and evaluate the statistics. Determine hydraulic subreaches based on hydraulic statistics. The analysis involves determining the average hydraulic properties for each reach and then applying the appropriate sediment transport relationship to each grain size fraction.
6. Sediment transport capacity analysis – Prepare steady state sediment transport capacity analysis through dividing the channel system into different reaches and comparing the capacity within each reach.
7. Analyze historic trends in creek bed adjustment – Consider available historic data to gain insight into changes in bed characteristics throughout the period of record.

8. Analysis of local bed adjustment components – Study of the individual components of local bed adjustment
9. Calculate toe-down depth and freeboard height – Calculate the total toe-down depth and freeboard height of proposed soil cement bank protection based on the analysis of individual bed adjustment components.

2 Previous Fluvial Analyses for Study Reach

2.1 Introduction

Three sediment transport studies have been conducted within the study reach between Interstate 5 and the Santa Clara River in the last 20 years. The first study was conducted by Simons, Li & Associates (SLA) and submitted November, 1990. The report sought to provide information for flood and fluvial process management in the Santa Clara River from Soledad Canyon to Castaic Creek. The study included multiple tasks including data collection, quantitative analysis and mathematical modeling, alternatives evaluation, and other assignments.

The second study produced by SLA and submitted January, 1992, provided fluvial analysis of lower Castaic Creek including sedimentation analysis. The intent of the study was to provide technical data in support of design criteria. A third study conducted by Tetra-Tech dated November, 2003, conducted bed scour analysis related to proposed bridge widening at Interstate 5.

2.2 Simons, Li & Associates, 1990

The 1990 SLA study is broad in reach and included use of the proprietary QUASED numerical model for fluvial analysis. At present, it is not clear if this model is still available or how it compares to other fluvial modeling software such as HEC-6. The study included a large volume of data analysis including flood flow frequency analysis, HEC-2 numerical modeling, historic photograph analysis, and sediment yield calculation. Supplemental design criteria are provided for subsequent studies. Sediment gradation data was presented in an interim version of the study. The location map of the sediment sample is vague with exact location of the single sample, although the gradation curve is similar to that presented in Chapter 2.3, below, but extends only to ¾" sieve size. The County has indicated that a copy of the report resides in the DPW Technical Library.

The study suggests a Capital Flood peak discharge in the Creek of 33,820 cfs downstream of the Interstate 5 Bridge. For the portion of the Creek downstream of Interstate 5, the bed was found to be adjusting between +1.0 to +2.0 following a Capital Flood (Figure 3.4, page 3-12). The study notes that the Creek appears to exhibit "degradational trends (p. 3-16)." The study also noted that the reservoir trapped upstream sediments, prohibiting their release downstream. Additionally, the report recommends the construction of bank protection on the west bank (p.3-23). QUASED modeling suggests that the study reach (D3) will not be altered with respect to degradational behaviors by the proposed improvements listed in the study. A recommended toe-down depth is not apparent.

2.3 Simons, Li & Associates, 1992

The 1992 SLA study covers the hydraulic and sedimentation analysis associated with proposed improvements to the Creek. The study utilized a peak Capital discharge of 50,100 cfs. It also utilized the MPM bed load function and the Einstein method for sediment transport and suspended load respectively. ACOE HEC-2 model to assess hydrology was employed, and the QUASED model was used for estimating dynamic sediment routing. The Capital discharge was run for sediment calculations and the study found that, depending on vegetation conditions assumed, general adjustment ranged from +1.1 to -3.9 feet for a single event and +0.4 to -2.3 feet in the long term. The middle reaches were found to be aggrading, both for single events and over the long-term, while the up- and downstream limits of the study were degrading (Table 5.2, p. 37). Total adjustment depth ranged from -5.4 to -11.7 feet and toe-down depth ranged from -5 to -12 feet. The report notes that according to the Los Angeles County design standards, toe-down may be required from -12 to -18 feet.

2.4 Tetra Tech, 2003

The Tetra Tech November 2003, report is specifically concerned with the scour associated with bridge widening. The report does note, however, that general scour in the vicinity of the Interstate 5 Bridge is 1.13 meters (3.71 feet). Maximum local scour at the bridge is expected to be 5.51 meters (18.08 feet). Total adjustment included 0.60 meters (2 feet) of low-flow incisement with no long-term adjustment expected. For the purposes of the present study, total adjustment predicted by the Tetra Tech report estimated 5.68 feet, neglecting the affects of the bridge itself, in the vicinity of the Interstate 5 Bridge. These estimates of scour are based on HEC-18 numerical modeling. The design discharge used in the report is 41,500 cfs.

3 Sediment Characterization and Analysis

3.1 Sediment Data Collection

To characterize the sediment of the creek bed and by extension the possible bed load of sediment during discharge events, a sediment grain size analysis was conducted. The goal of the analysis is to gain a statistical representation of the size distribution of soil components of the Creek bed. Grain size distribution analysis is a powerful tool because the results can represent both a qualitative description of soil make up as well as quantitative input for further predictive measures, such as fluvial modeling.

High quality sediment collection for the Castaic Creek and its tributaries along the study reach was conducted by Allan E. Seward Engineering. Within the Creek, 20 samples were collected at eight different locations positioned along various Creek subreaches. These sampling locations are compiled in Table 3.1. All sampling was conducted in the upper five feet of the active or recently active portion of the bed with the noted exceptions. At least three samples within a sampling station were taken. Sampling locations for the study are presented in an September 15, 2005 report, reproduced in Appendix Chapter 3.

Table 3.1 Location and Average D_{50} of Sediment Samples -- Castaic Creek

Reach	HEC-RAS Section	Sample Number	Reach Average[1] D_{50}
A	11242	OR	1.9
	8396	OC	
	5992	CC	
	5557	HC	
B	2974	CH	1.6
	1530	HW	
	NA	HS	
	NA	CS	

1: Reach average is the average of all samples in the reach.

3.2 Sediment Gradation Analysis

Generally grain size distribution analysis is broken down into three distinct steps. The first step is to dry the samples. Drying is accomplished in a desiccator or similar apparatus. The second step is to sieve or otherwise separate the sediment by particle size. Finally, fine material (smaller than standard mesh 200) is analyzed using hydrometric techniques. The sediment distributions are plotted on semi-log plots by percent finer for a given sample size. For this study, no fine material is included in analysis because fine material is generally transported as wash load, which is not of concern here. Distribution data has been averaged on a station-by-station basis. Averaging

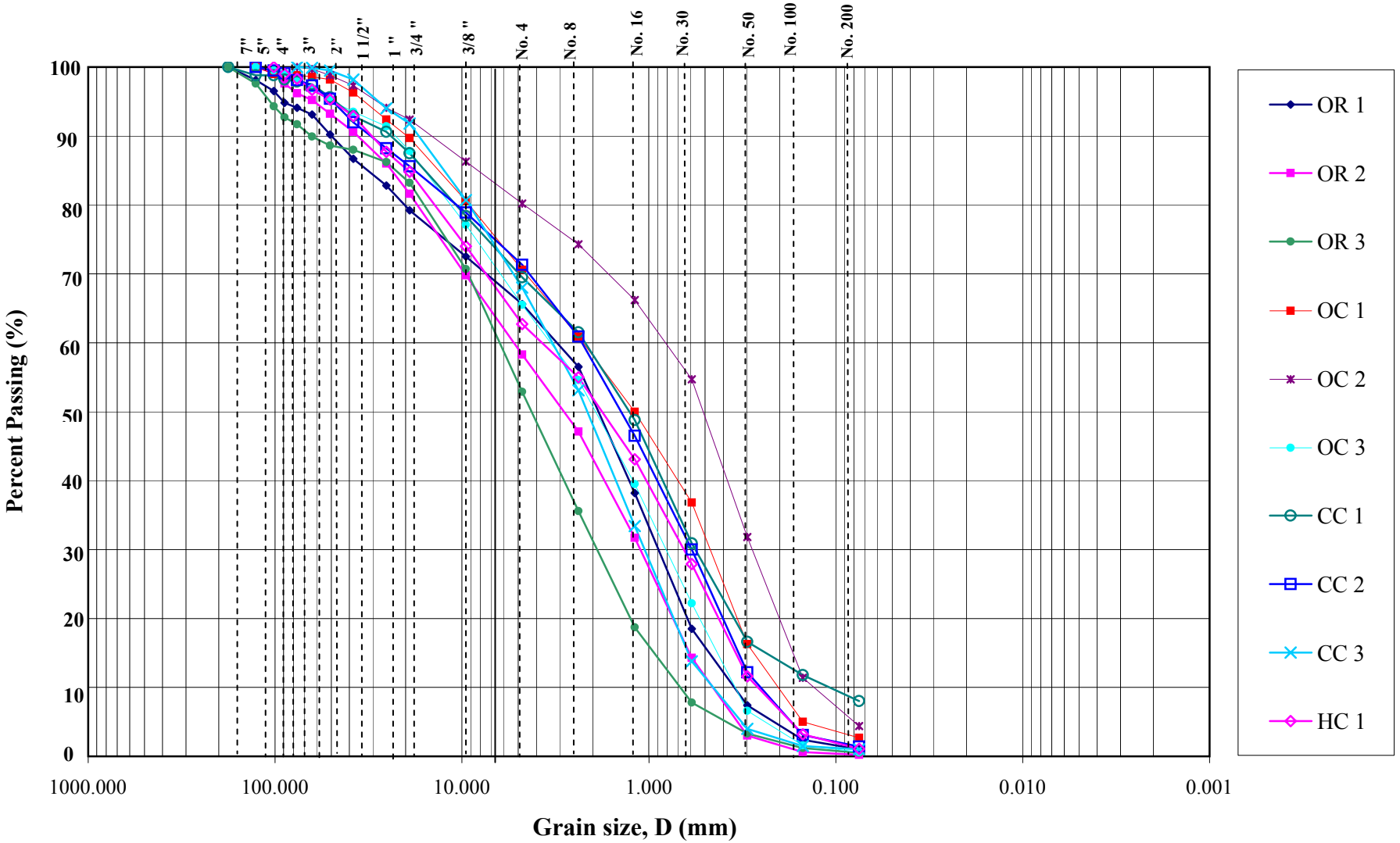

Figure 3.1A Castaic Creek Streambed Grain Size Reach A Gradation Curves

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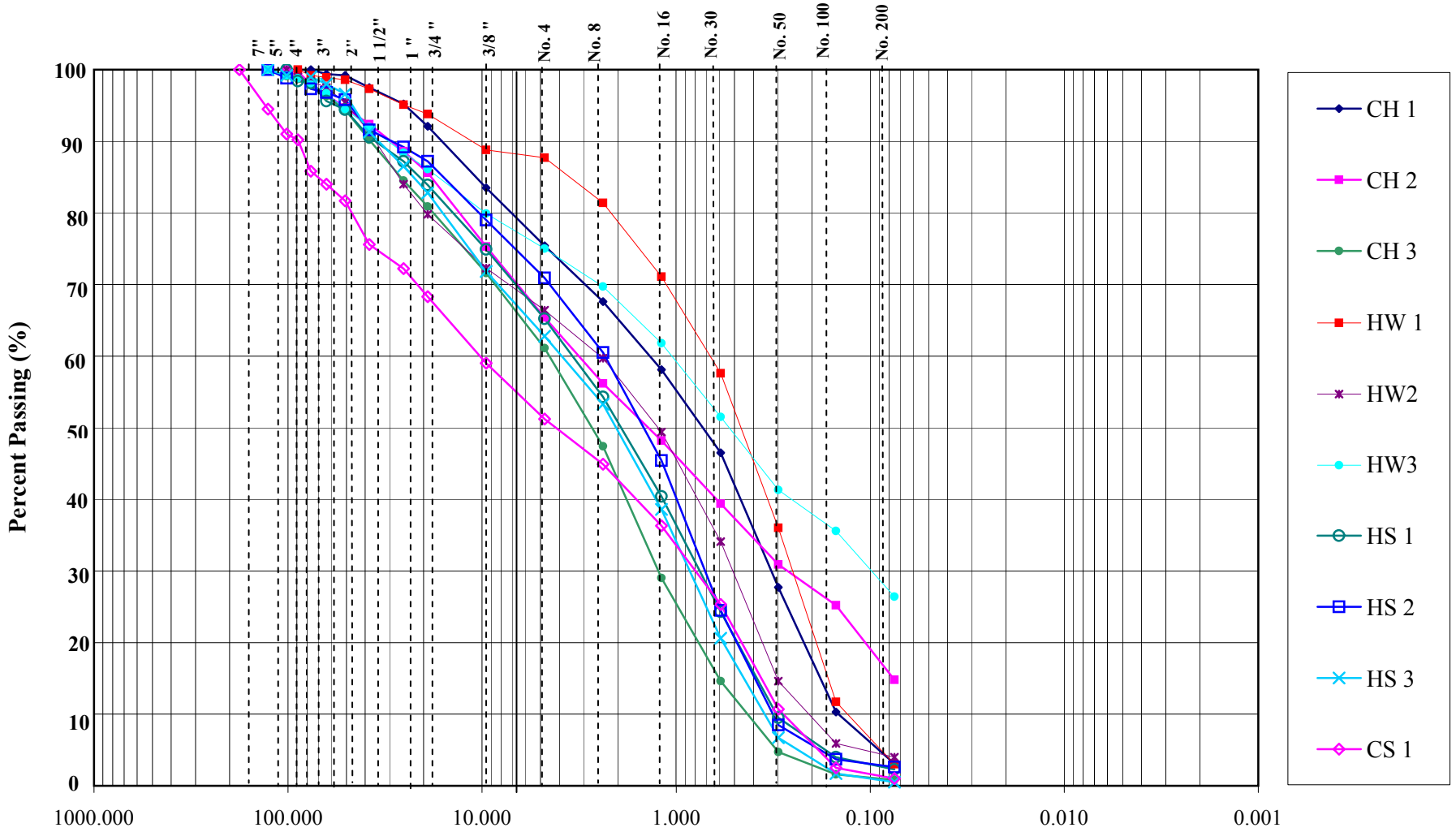

Figure 3.1B Castaic Creek Streambed Grain Size Reach B Gradation Curves

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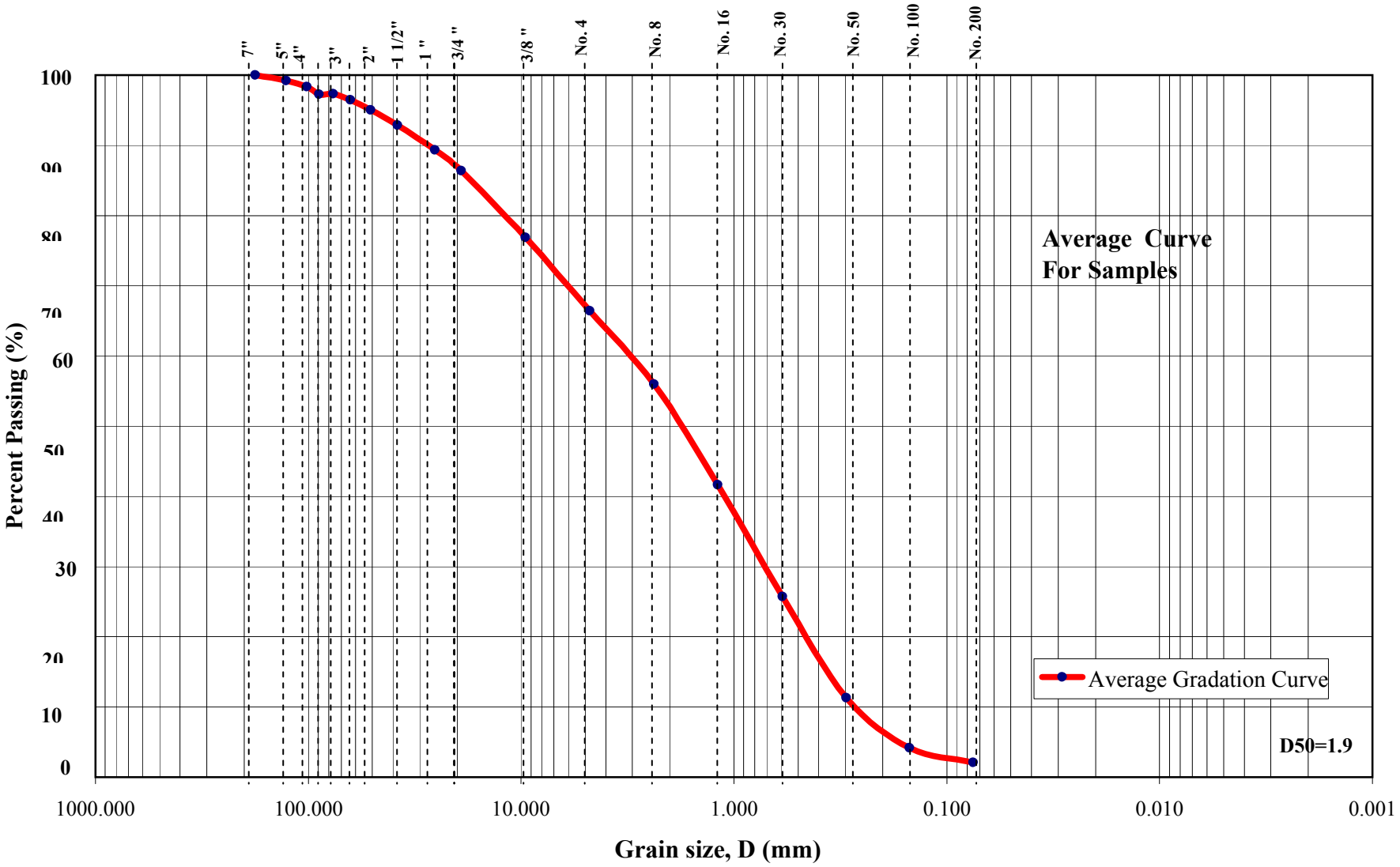

Figure 3.2A Castaic Creek Bed Average Sediment Gradation Curve Reach A

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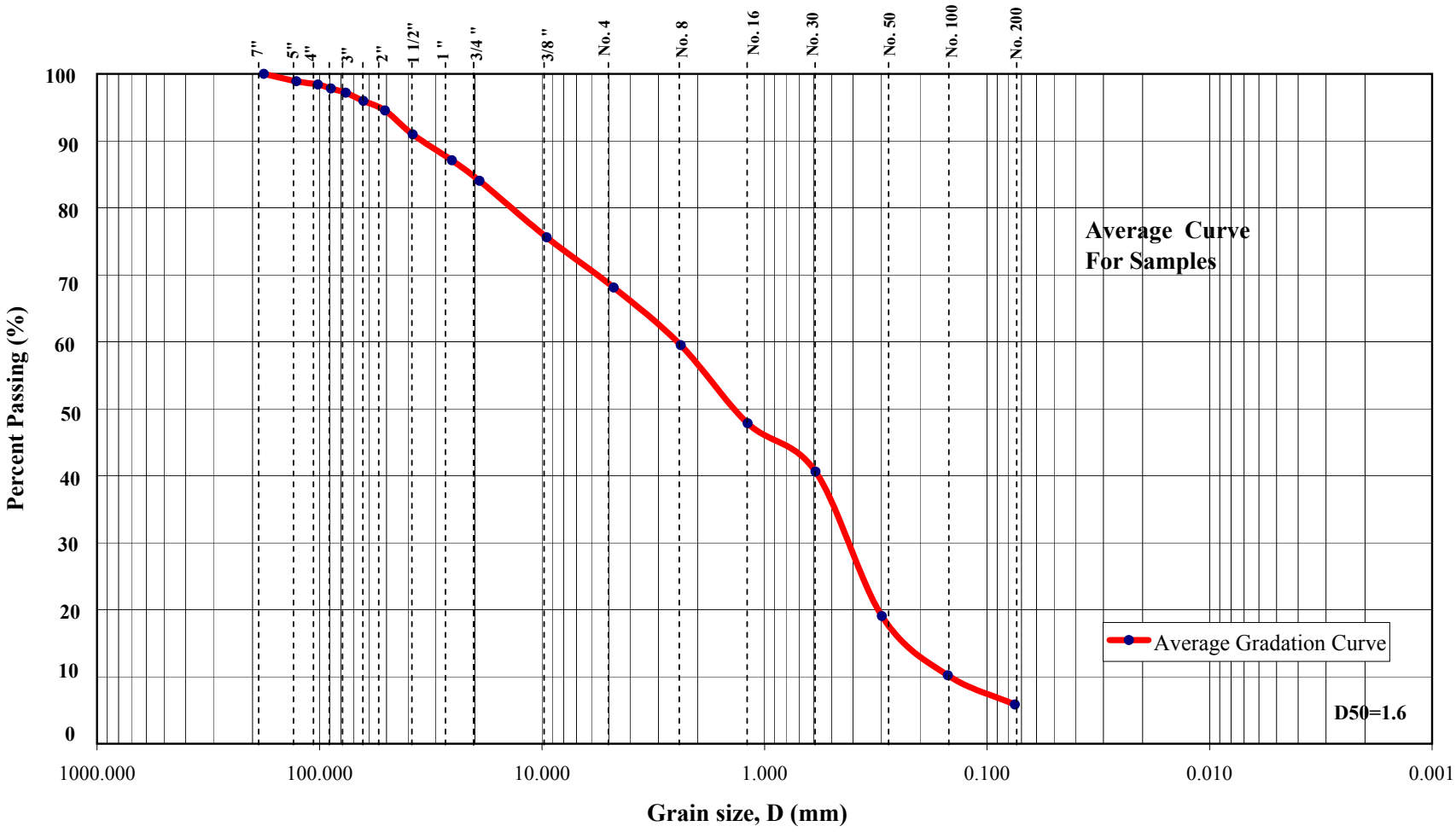

Figure 3.2B Castaic Creek Bed Average Sediment Gradation Curve Reach B

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of data for a given station is accomplished by taking the mean of the three samples from that station. Averaging for a reach is the average of all samples in that reach. Averaging provides a single representative sediment grain size distribution for a given station that can be used for numerical modeling or other analysis. A plot of grain size distribution for each sampling station is presented in Figure 3.1A-B. All sampling stations are averaged in a given reach to calculate the sediment data for that reach. Raw grain size distribution plots for the study are presented in the September 15, 2005 Allan E. Seward Engineering Geology report, reproduced in the Appendix Chapter 3.

3.3 Sediment Characterization

A review of the raw gradation curves for the samples indicates that most samples are comprised of poorly graded sands and gravels with cobbles. The D_{50} values for all samples ranged from 0.46 to 4.24 mm. A comparison of Figure 3.2A-B with Figure 3.1A-B indicates that averaging retains the essential character of the sampled soil.

4 General Adjustment

4.1 SAM Model

General adjustment was estimated in this study using the US Army Corps of Engineers (ACOE) SAM steady state and HEC-6 one-dimensional numerical models. Here, SAM was employed to provide a first approximation of sediment transport potential for multiple subreaches within Castaic Creek. The SAM Sediment Hydraulic Package is an integrated system of programs developed through the Flood Damage Reduction and Stream Restoration Research Program to aid in the analyses associated with designing, operating and maintaining flood control channels and stream restoration projects. SAM combines the hydraulic information and the bed material gradation information to compute the sediment transport capacity for a given channel or floodplain hydraulic cross-section for a given discharge at a single point in time. A number of sediment transport functions are available for this analysis and SAM has the ability to assist in selecting the most appropriate sediment transport equation.

The three primary fluvial components of SAM are SAM.HYD, SAM.SED and SAM.AID. SAM.HYD provides a steady state, normal-depth, one-dimensional representation of channel hydraulics. The SAM.SED module combines the hydraulic parameters with the bed material gradation curve to compute bed material discharge rating curves by size classification. The SAM.AID module provides the user with recommended sediment transport equations based on the best matches between hydraulic parameters and grain size distribution of the study reach with parameters from widely accepted and published research.

4.1.1.a SAM Model Theory and Limitations

The SAM numerical model is built upon hydraulic and fluvial components. The hydraulic components include representations of river bed characteristics that are input into an analytical procedure. The fluvial component includes representation of bed gradation as percent finer statistics and a selection of up to twenty sediment transport equations. SAM's hydraulic component will accept either average reach parameters or cross-section data imported from HEC-2/HEC-6 models. Hydraulic modeling is based on a uniform flow equation where discharge is the dependent variable such that,

$$Q = f(D, n, W, z, S)$$

where Q is discharge in cfs, D is flow depth in feet, n is the Manning's number, W is bottom width in feet, z is the channel side slope, and S is the energy slope. The bottom width is representative of the total moveable bed width of the channel and Manning's number is a composite value. Normal depth is calculated using Manning's equation, and effective values of width and depth are calculated following normal depth calculations. In cases where HEC-2/HEC-6 cross-sections are used for modeling, as in this

study, the effective depth and width are calculated from the cross-section data based on the channel hydraulics.

The fluvial component is based on sediment transport functions to calculate the bed material portion of the sediment discharge rating curve.

The sediment transport equations are of the form,

$$GS_i=f(V, D, S_e, B_e, d_e, \rho_s, G_{sf}, d_s, i_b, \rho_f, T)$$

where GS_i is the transport rate for sediment size class i; the hydraulic terms V, D, S_e, and B_e, are the average velocity, effective flow depth, energy slope, and effective flow width, respectively; the sediment particle parameters d_e, ρ_s, and G_{sf} are the effective particle size, particle density, and grain size shape factor, respectively; the sediment mixture properties, d_s and i_b are the geometric mean particle size of sediment class i and fraction of class i in the bed, respectively; and the fluid properties ρ_f, and T, the water density and temperature, respectively. Twenty well known, published, peer-reviewed transport equations are available including Ackers-White, Colby, Laursen-Copeland, Laursen –Madden, MPM, Toffaleti, Yang, Van Rijn, and others. Once the data assembly is complete, the SAM.SED module can be used to create a sediment discharge-rating curve based on grain size distribution. The reader is referred to the SAM user's documentation for further reference.

It is important to note that the SAM model is a zero-dimensional computational package that is only based on a single averaged cross-section at a particular point in time. As such, SAM simulations can only represent a reach average during a steady state discharge. Because SAM applies sediment transport to a point, no variability in size distribution in either space or time is calculated. With these limitations in mind, in this study SAM is intended to provide a first calculation of sediment transport to which other calculations can be compared.

4.1.1.b SAM Model Assembly

In this study, hydraulic representation of the river bed is accomplished in several distinct steps. First, the HEC-RAS numerical model is thinned to no more than 100 stations per cross-section using HEC-RAS's cross-section points filter. Second, the HEC-RAS geometry is converted to HEC-2 format using the Army Corps RAS2UNET software. Next, the HEC-2 model deck is arranged to run in sub critical mode and all features, such as ineffective flow areas and levees, are added. Once the HEC-2 model is complete it is re-imported into HEC-RAS and compared to the original model. Any differences between the RAS and 2 models are resolved. Once the original and re-imported models match, the

HEC-2 model is run to produce the Army Corps' T95 binary hydraulic simulation output file. Next, the T95 file is then read directly into SAM using the SAM model's M95 subroutine using the reach length option. Sections are chosen based on the reach characterization described below. A comparison of output and discussion is presented in Appendix Chapter 4.4

This conversion methodology is powerful because it ensures that data created for, and analyzed using; HEC-RAS and HEC-2 hydraulic software is fully compatible with, and implemented in, SAM fluvial analyses. Copies of the HEC-RAS and HEC-2 models for both the existing and proposed (ultimate) conditions are included in Appendix Chapter 4, as well as a table comparing the velocity and depth of the different models used in each phase of the project described above.

Both the existing and proposed conditions HEC-RAS model were originally developed by Sikand Engineering in *Newhall Ranch Santa Clara River HEC-RAS Study* (June, 2000) and submitted as part of the Newhall Ranch Specific Plan. The Sikand model was further modified by PACE to create a model that was reviewed and accepted by LACDPW. Slight differences exist between the PACE and Sikand models in terms of discharge and stationing and changes have been made to the PACE model to meet LACDPW criteria, otherwise the models are the same. The only other changes made to the HEC-RAS model occur in the proposed condition where the alignment of soil cement bank protection was revised to reflect the most current proposed condition, and the afore mentioned Q_{CAP} discharge approved by LACDPW for this study in 2005. The current position of the bank protection avoids River jurisdictional areas and is further outside the River than the bank protection originally proposed in the Sikand model. Finally, subreaches within the SAM model are specified and average hydraulic parameters are calculated for those subreaches. Subreaches are determined by examining the hydraulic parameters of the individual HEC-RAS cross-sections and identifying correlations between those hydraulic parameters and the longitudinal position in the channel of the individual cross-section. This process is described in detail below.

4.1.2 Reach-by-Reach Channel Hydraulic Characterization

Reaches are defined by changes in discharge within Castaic Creek. These changes occur at HEC-RAS sections STA 12406 (Reach A) and STA 5340 (Reach B) in the existing and proposed conditions, as shown in Figure 1.1. In the case of Castaic Creek, all subreaches have been defined based on a sensitivity analysis of model reach length and are approximately 1,750 feet long. The variation of subreach length in both the existing and proposed condition resulted in large variations in calculated toe-down depth for both reaches in the Creek. To determine the SAM model's sensitivity to reach length for the Creek, a reach

sensitivity analysis was conducted to determine the subreach length where the SAM model converged to a single value of adjustment. The sensitivity analysis was conducted by running the SAM model for various increasing subreach lengths of approximately 1400, 1750, and 2300 feet. Where the value of adjustment converges for the given subreaches, the convergence is said to be the limit of sensitivity of the model to reach length and the best approximation of the study parameter. Model simulations presented in Appendix Chapter 4.5 show that as reach length increases, calculated general adjustment converges to a single value of adjustment for each of the subreaches for the existing and proposed conditions ranging from approximately +3 to -4 feet. This value is approximately the same as that found by Tetra Tech, 2003 and determined by both SLA studies. It is expected that the HEC-6 numerical modeling will predict lower general adjustment than these converged values.

Subreaches are defined in Table 4.1A-B and shown in Figure 1.1. Both the existing and proposed conditions models employ similar subreach lengths and locations. The use of similar subreaches was employed for two reasons: to ensure consistency between existing and proposed conditions analysis, and because the sensitivity analysis for both the existing and proposed conditions is similar.

Table 4.1A: Castaic Creek Existing Conditions Sub-Reach Stationing

Subreach	US Section	DS Section	Transport Equation
SRA1	12407	10983	MPM
SRA2	10738	9299	MPM
SRA3	9043	7689	MPM
SRA4	7326	5435	MPM
SRB1	5340	4221	MPM
SRB2	3865	2627	MPM
SRB3	2308	1173	MPM

Table 4.1B: Castaic Creek Proposed Conditions Sub-Reach Stationing

Subreach	US Section	DS Section	Transport Equation
SRA1	12407	10983	MPM
SRA2	10738	9299	MPM
SRA3	9043	7689	MPM
SRA4	7326	5435	MPM
SRB1	5340	4221	MPM
SRB2	3865	2627	MPM
SRB3	2308	1173	MPM

4.1.3 Input Data and Selection of Transport Functions

Representation of sediment grain size distribution in SAM takes the form of percent finer data obtained from sieve analysis of channel

sediment grab samples. At each sample location multiple samples are collected and analyzed, and the average data is input into the model. All sampling and sieve analysis was conducted by Allan E. Seward Engineering Geology, and sample locations were chosen based on either the presence of recently active alluvium or the presence of adjacent/underlying older alluvium commonly incorporated into stream sediment load during major events, and based on hydraulics. Environmental constraints on subsurface investigations in active drainages limited sampling locations in some instances. In these cases the most representative and obtainable data is used as described previously in Chapter 3.

4.1.3.a SAM.AID Application and Theory

Sediment transport equations used in all SAM modeling were chosen with the assistance of the Army Corps' SAM.AID subroutine. The SAM.AID subroutine determines the most representative transport function based on the hydraulic parameters and percent finer data for each subreach by comparing model data with the results of 20 peer-reviewed and widely acknowledged sediment transport studies. This case-by-case transport equation selection is more likely to provide a robust representation of channel sediment transport than choosing an individual transport equation for all reaches.

Application of different transport functions to an individual channel reach may provide significantly differing model output. This is because the parameters of a given study from which the function is derived vary greatly. To accomplish the task of guiding the user in selecting an appropriate transport function, SAM.AID assumes that the function which best represents sediment transport in a gaged stream would also best represent transport in an ungaged stream with similar sediment and hydraulic characteristics. SAM.AID begins by comparing study parameters (V, D, S_e, B_e, D_{50}) with parameters in the transport function database. Comparison begins by determining if D_{50} falls within one of the ranges identified in the database. Once the database initial matches have been made, the three best matched sediment transport functions for the study reach are listed along with the parameters that matched the data set.

Once the best transport equation matches have been determined by SAM.AID, the most representative equations are run in SAM.SED for each subreach. Yang and MPM equations are added to all simulations where they are not explicitly matched by SAM.AID so that there is a continuity of comparison between subreaches. Yang was chosen because it was derived for gravely sediments (Chang, 1988) and MPM was chosen because it has been noted by Alonso (1980) that it compares favorably with bed load calculations. Both bed load and gravel estimates are important to the specific aims of this study. Following SAM.SED computations sediment transport potential for each subreach can then be estimated by reviewing the

calculations from each equation and analyzing the results. In some simulations and with some transport functions, SAM analysis provides transport potential calculations three or more orders of magnitude higher or lower than the median. This is because the transport function employed may have been based on sediment and hydraulic parameters very different than those examined in the present study despite being selected by SAM.AID. These values have been rejected and generally the median value of transport is employed.

The MPM equation was found to be the representative transport equation for all subreaches for the existing and proposed conditions because it was the only equation which produced results consistent with physical viability. The full summary of transport potential by subreach for the various transport equations for both existing and proposed conditions is provided in Appendix Chapter 4.6. The data show that existing and proposed conditions best matched with MPM estimates of transport, and that the other equations estimate unphysical volumes of discharge. Since the D_{50} for a given reach is the same for both the existing and proposed conditions models, the differences in transport function applicability is related to hydraulic differences between the two conditions.

4.1.4 SAM Bed Stability

Bed stability can be examined based on the change in potential transport between channel subreaches. Subreaches are readily determined from changes in hydraulic parameters, and frequently the most significant hydraulic parameter in terms of impact on stream stability is discharge (volume per unit time). If a channel subreach has equal potential transport both entering and exiting the reach then the subreach is said to be in, "equilibrium." Frequently, however, channel subreaches are either in an aggrading or degrading condition. For the purposes of this study, aggrading reaches are those whereby the potential transport entering the reach (the potential transport of the subreach upstream of that under immediate consideration) is higher than the potential transport leaving the subreach (the potential transport of the subreach under immediate consideration). In degrading subreaches the opposite is true and potential transport entering the reach is lower than that leaving the subreach. While it would appear that downstream subreaches would be degrading constantly because discharge generally increases in downstream subreaches and in turn increases the transport potential as one moves downstream, other factors such as hydraulic depth, mean subreach velocity, hydraulic top width, and bed slope contribute significantly to potential transport.

Bed stability was determined by calculating the difference between subreach upstream and downstream sediment potential transport for the Q_{CAP} discharge. Transport potential for each reach is shown in Table 4.2A-B. The table shows no clear trend in transport potential as a function of subreach. The difference in transport potential, ΔTP

(ton/day), was converted to bed adjustment, *GA* (feet), in the form of a chain calculation as:

$$GA = \frac{\Delta TP}{\rho bRL} day$$

where ρ is density in tons per cubic feet, *b* is channel width in feet, *day* denotes one day's time, and *RL* is reach length in feet. Density has been taken as 165.36 lb/ft^3. A summary of the adjustment for each reach is shown in Table 4.3.

General adjustment is based on SAM modeling presented in Figure 4.1A-B and Table 4.4 for the existing and proposed conditions, respectively. It is important to note that no apparent pattern of aggradation/degradation is apparent between cross-sections in the figure. General adjustment calculated using the equation presented in the Los Angeles County Hydrology and Sedimentation Manual (LACH&SM) is also shown in the figures. This latter calculation methodology is only based on flow mean velocity at a given channel section as computed by the HEC-2 conversion from the HEC-RAS model of the system. Scour predicted by the LACH&SM is less variable than that predicted by SAM for both the existing and proposed conditions. It should be noted that scour predicted by the ACOE HEC-6 numerical model is expected to be lower than SAM predictions in terms of depth and some longitudinal deviation from SAM results are expected. Additionally, less variability is expected from the HEC-6 model because of the time-dependent nature of the model.

The tables and figures indicate the general adjustment ranges from -2.3 to +3.1 feet for the existing condition, and -3.3 to +2.4 for the proposed condition, respectively. Subreaches SRA1, SRA2 and SRA3 do not change much between the existing and proposed condition, with the first two degrading slightly more and the third aggrading slightly more, according to SAM predictions. Subreaches SRA4 and SRB1 changed 50% or more with the former increasing aggradation by approximately 50% and the latter increasing in degradation by more than 50%. Of the remaining sub-reaches, SRB2 increases degradation slightly and SRB3 decreases aggradation slightly in the proposed condition, as predicted by SAM. It should be noted that general adjustment is not expected to exceed (decrease) approximately -2.5 feet because historical trends indicate that the Creek bed is approaching the armoring layer, as discussed in Chapter 5, below. These calculations indicate, however, that given a bed unlimited by armoring the bed could adjust by these amounts. Finally, the general trend in general adjustment for the study reach as indicated by SAM modeling is not apparent for either the existing condition or proposed condition.

Table 4.2A: Castaic Creek Existing Conditions Bed Stability, QCAP*								
Subreach	US Section	DS Section	Trans Eq	Transport (ton)	Top Width (ft)	Depth (ft)	A/D	Grade Change (ft)
SRA1	12407	10983	MPM	76,463	512.1	2.3		-2.3
SRA2	10738	9299	MPM	199,997	815.1	1.3	DEGRADE	-1.3
SRA3	9043	7689	MPM	187,431	1172.0	0.1	AGGRADE	0.1
SRA4	7326	5435	MPM	81,678	669.5	1.0	AGGRADE	1.0
SRB1	5340	4221	MPM	208,746	1070.2	1.3	DEGRADE	-1.3
SRB2	3865	2627	MPM	264,963	786.7	0.7	DEGRADE	-0.7
SRB3	2308	1173	MPM	112,041	529.1	3.1	AGGRADE	3.1

* Calculations for aggradation/degradation at the A/B reach interface are made at the request ot LACDPW. The discharge increases significantly at this location. It is expected that the HEC-6 numerical modeling will show significantly different values.

Table 4.2B: Castaic Creek Proposed Conditions Bed Stability, QCAP*								
Subreach	US Section	DS Section	Trans Eq	Transport (ton)	Top Width (ft)	Depth (ft)	A/D	Grade Change (ft)
SRA1	12407	10983	MPM	76,742	507.4	2.4		-2.4
SRA2	10738	9299	MPM	216,910	781.0	1.5	DEGRADE	-1.5
SRA3	9043	7689	MPM	174,631	1150.8	0.3	AGGRADE	0.3
SRA4	7326	5435	MPM	46,272	426.0	1.9	AGGRADE	1.9
SRB1	5340	4221	MPM	179,197	436.6	3.3	DEGRADE	-3.3
SRB2	3865	2627	MPM	233,083	527.8	1.0	DEGRADE	-1.0
SRB3	2308	1173	MPM	120,980	505.7	2.4	AGGRADE	2.4

* Calculations for aggradation/degradation at the A/B reach interface are made at the request ot LACDPW. The discharge increases significantly at this location. It is expected that the HEC-6 numerical modeling will show significatnly different values.

Table 4.3: Castaic Creek Existing vs Proposed Conditions Bed Stability*					
Subreach	US Section	Existing Conditions Grade Change (ft)	Proposed Conditions Grade Change (ft)	**Delta (ft)**	**Result**
SRA1	12407	-2.3	-2.4	**0.1**	**INCREASE DEG**
SRA2	10738	-1.3	-1.5	**0.2**	**INCREASE DEG**
SRA3	9043	0.1	0.3	**-0.2**	**INCREASE AGG**
SRA4	7326	1.0	1.9	**-0.9**	**INCREASE AGG**
SRB1	5340	-1.3	-3.3	**2.0**	**INCREASE DEG**
SRB2	3865	-0.7	-1.0	**0.3**	**INCREASE DEG**
SRB3	2308	3.1	2.4	**0.7**	**DECREASE AGG**

* Calculations for aggradation/degradation at the A/B reach interface are made at the request ot LACDPW. The discharge increases significantly at this location. It is expected that the HEC-6 numerical modeling will show significatnly different values.

Table 4.4: Castaic Creek SAM & LACH&SM General Adjustment Comparison					
Subreach	US Section	Existing Conditions		Proposed Conditions	
		SAM[1] (ft)	LACH&SM[2] (ft)	SAM[1] (ft)	LACH&SM[2] (ft)
SRA1	12407	-2.3	-0.5	-2.4	-0.5
SRA2	10738	-1.3	-4.2	-1.5	-4.2
SRA3	9043	0.1	-2.7	0.3	-2.7
SRA4	7326	1.0	-1.7	1.9	-1.1
SRB1	5340	-1.3	-2.8	-3.3	-3.6
SRB2	3865	-0.7	-4.2	-1.0	-5.1
SRB3	2308	3.1	-1.7	2.4	-1.8

1: SAM calculations represent an entire subreach

2: LACH&SM calculations represent one cross-section. Data for sections as presented in Appendix Ch. 6

4.2 HEC-6

The ACOE HEC-6 model is a one-dimensional moveable bed open channel hydraulic and sediment model. The model was designed to simulate change in

Figure 4.1A: Castaic Creek Existing Conditions SAM & LACH&SM General Adjustment Comparison

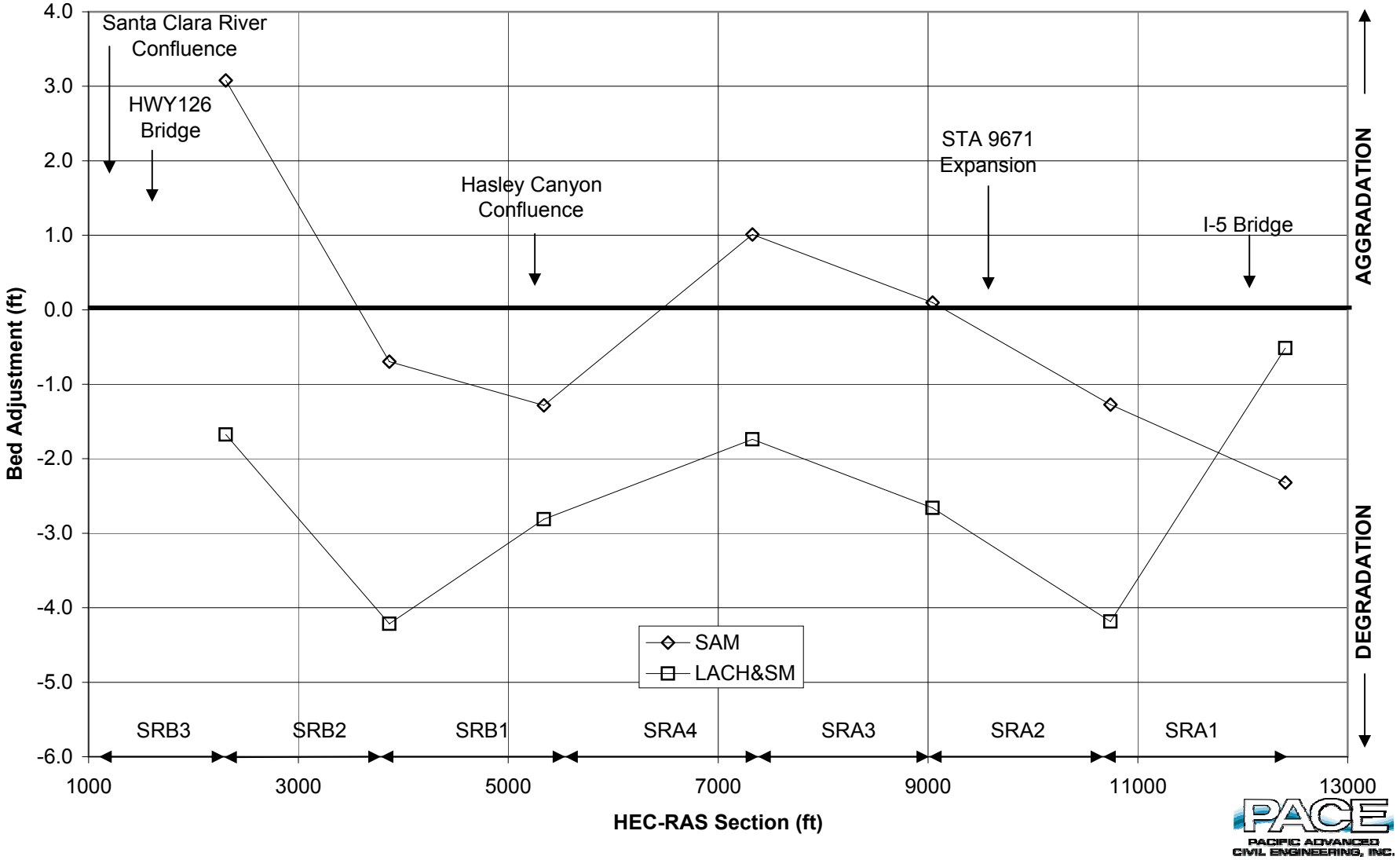
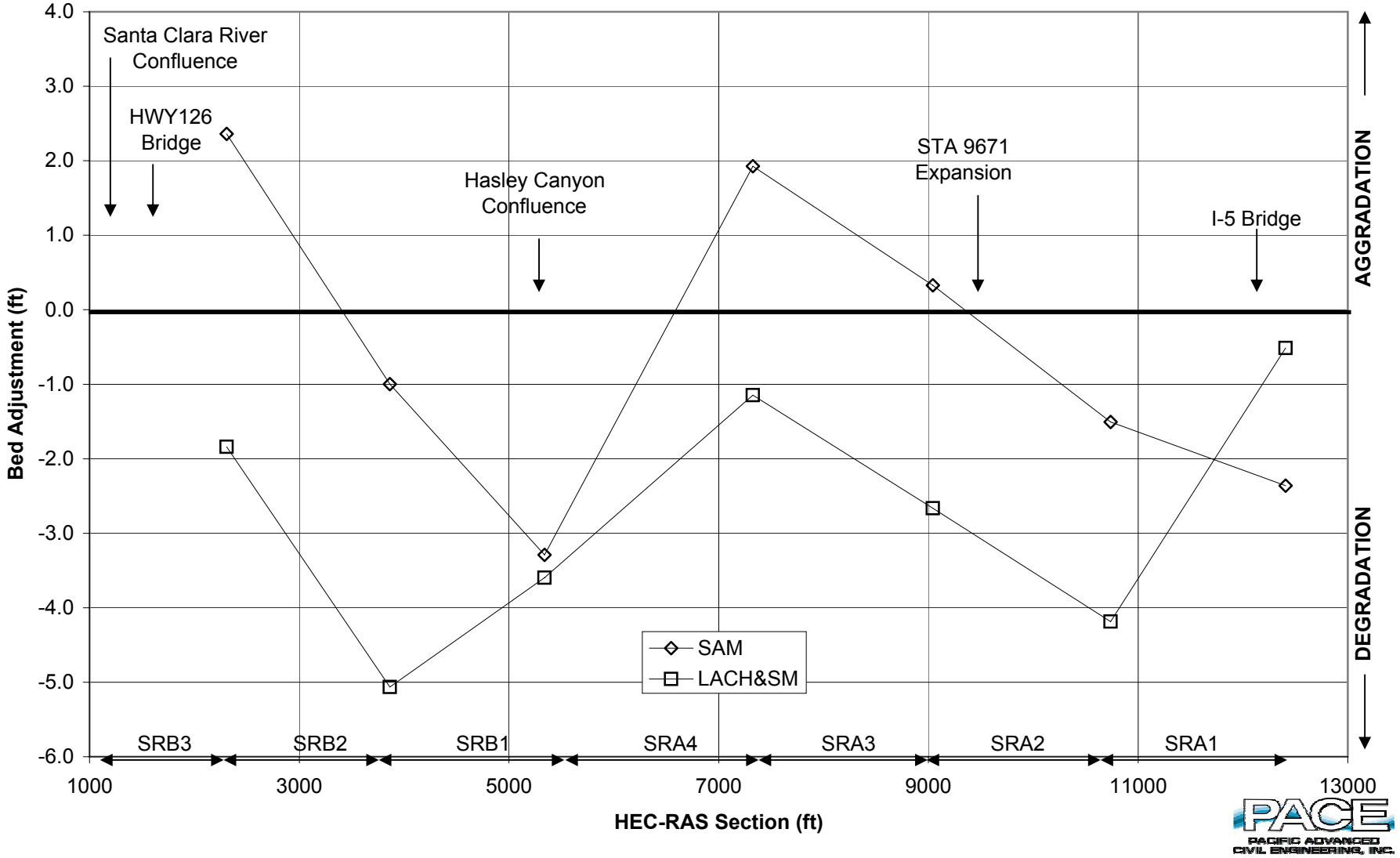


Figure 4.1B: Castaic Creek Proposed Conditions SAM & LACH&SM General Adjustment Comparison



river bed profiles resulting from sediment scour and deposition over long periods of time. The model segments hydrograph data into a progression of steady flow events with varied discharge and duration. Every segment of flow is used to calculate a water surface profile and associated hydraulic parameters (e.g. velocity, depth, etc.). From the hydraulic parameters potential sediment transport rates are estimated for each model reach and scour or deposition is next estimated so that cross-section shape can be updated. Sediment calculations are based on grain size distribution so that sorting and armoring can be considered.

A one dimensional energy approximation to the equations of motion is used for hydraulic calculations in HEC-6. Manning's equation is utilized to incorporate bed friction. The model also uses both an up- and down-stream boundary condition with internal conditions optional. Krone's method is for deposition of fines, and the method of Ariathurai and Krone is used for scour. Sediment transport functions are user selectable and 13 different equations are possible. Colby's method is used to adjust transport potential and armoring is simulated using Gessler's method. Sediment boundary conditions operate such that inflowing sediment load is a function of inflow discharge.

At this date the "T" enhancement of the HEC-6 program, created by Mobile Boundary Hydraulics, is expected to be used.

5 Long-Term Adjustment

Long-term adjustment was calculated based on historical records in the form of topographic data. First, topographic data, provided by Allan E. Seward Engineering Geology dating from 1930, 1947, 1963, 1999, 2004 and 2005, was digitized (See Appendix Chapter 5). This was accomplished by determining a common coordinate system and creating lines of equal ground surface elevation within the study area. Topographic data was available in several formats including digital elevation maps (1999, 2004, 2005), topographic surveys (1963) and quad maps (1930, 1947). Digital elevation maps were only adjusted for horizontal location. Cross-sections were next cut at the locations of select HEC-RAS sections (show in Figure 5.1) for each historical topography. At least one cross-section was chosen for each subreach established in Section 4 based on engineering interest such as gravel mining or agricultural infill, as described below. The HEC-RAS sections chosen are 1531, 1645, 1982, 2627, 2975, 4572, 6624, 8050, and 9299. Historical topographic data is not available above approximately section 9299. Areas of the 1999, 2004 and 2005 sections are calculated and these areas are used to calculate the average change in bed elevation over time. The other topographies (1930, 1947 and 1963) are not used to calculate average change in bed because the trends in bed change that occurred during these years occurred prior to the construction of the Dam upstream of the project site, as discussed below. To calculate the area of a given cross-section the lowest historical point on the section is determined and the area of each vertical foot of the section in one foot intervals is calculated, as shown in Figure 5.2A-I, for each topography. The area of a section is the sum of the one foot area intervals, also shown in the figure. All areas for a given section have a common toe and top from which the area is calculated. The relative average change in depth for a given section and topography is calculated as the area divided by the top width, where the top width is taken as the width of the upper most one foot area. The top width in this sense is not a hydraulic characteristic but a physical one, which along with the sectional area determines the maximum capacity of the section. Moreover, the calculated depth is a relative physical value based on the section area and represents an average physical characteristic of the section as a whole.

Several events within the available historical record (1930 to present) have had an impact on the Creek bed and fluvial mechanics. These events include construction of the upstream dam, construction of bridges spanning the Creek, agricultural infill along the Creek banks, gravel mining of the Creek bed and banks, and periodic burning of surrounding vegetation during forest fires. These different historical events can be placed into two categories: events that occurred prior to completion of construction of the Dam in 1971, and those that occurred both prior to and following Dam completion. Of the events listed here only agricultural infill, bridge construction (Commerce Center Bridge only) and periodic fires are in the latter category. The most significant historical event in the formation of the present bed condition was the aforementioned completion of the Creek's Dam and subsequent formation of Castaic Lake. The consequence of this event is tied to the Dam's ability to prohibit sediment from traveling from the upper watershed to portions of the Creek below the Dam. Within the project reach the construction of the Dam appears to have resulted in the continual degradation of the bed, detailed below. Gravel mining, as seen in the 1963 topography, removed large volumes of sediment from the bed within the project reach upstream of Commerce Center Drive. In its current state, the Creek bed appears to have recovered from