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Lower Deer Creek Ecosystem Restoration and Flood Management: Existing Conditions Baseline 1-D and 2-D Hydraulic Models

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1. INTRODUCTION

A one dimensional (1-D) hydraulic model and two-dimensional (2-D) hydrodynamic model of Lower Deer Creek in Tehama County, CA were developed for use in the Lower Deer Creek Restoration and Flood Management Feasibility Study by Mussetter Engineering, Inc. (MEI) and Michael MacWilliams Environmental Consultant, respectively. The models will be used to assess existing channel and levee capacity and to evaluate specific alternatives for modifying the existing system to improve flood control and habitat on Lower Deer Creek. This technical memorandum presents the model development and calibration results for these models. In addition, analysis of results for both models is presented to provide an assessment of existing (i.e., baseline) conditions in Lower Deer Creek under a range of flow conditions.

The project reach extends from the confluence with the Sacramento River upstream approximately 10 miles (**Figure 1**). The work was performed as part of the Deer Creek Watershed Conservancy CALFED ERP Project.

2. HEC-RAS MODEL DEVELOPMENT

The 1-D hydraulic model was developed by Mussetter Engineering, Inc. (MEI) using the Corps of Engineers HEC-RAS v3.1.3 program. This program uses the step-backwater approach (USACE, 2005) to compute water-surface elevations and hydraulic parameters (e.g., velocity, depth and channel topwidth) based on the cross-sectional geometry of the channel. A portion of an existing HEC-RAS model for the Sacramento River developed by the California Department of Water Resources (DWR) was incorporated into the Deer Creek model to develop the downstream boundary conditions.

2.1. Model Geometry

The geometry for the HEC-RAS model is based on 111 cross sections with an average spacing of about 450 feet. The cross sections were developed using survey information and available digital terrain data for the channel and surrounding floodplain. Main channel cross sections were surveyed by CH2M HILL in the fall of 2004 and by MEI in October 2006. The main channel cross sections were extended to include the overbank areas that could convey flows up to the 100-year peak discharge by cutting cross sections from a digital terrain model (DTM)

produced by DWR from data collected in May 1999. Additionally, aerial photography that was flown in June 2006 and provided to MEI by McBain and Trush, Inc was used to adjust sections where the survey data did not align with the DTM topography. Both the survey data and the topographic mapping (DTM) are referenced in the California State Plane, Zone II horizontal datum (NAD 1927) and the National Geodetic Vertical Datum of 1929 (NGVD29).

2.2. Downstream Boundary Conditions

To account for the significant backwater effects from the Sacramento River, a portion of an existing HEC-RAS model for this section of the Sacramento River (RM214.834 to RM217.22) was obtained from DWR and incorporated into the Deer Creek model. This portion of the DWR model includes the USGS gage located at Woodson Bridge (Sacramento River at Vina Bridge near Vina, California, USGS Gage No. 11383730, CDEC identification VIN), about 1 mile downstream from the mouth of Deer Creek. Information from this gage was used to develop a stage-discharge relationship for the downstream boundary condition for the entire model.

2.3. Other Model Input

Other input to the HEC-RAS model included hydraulic roughness coefficients, hydraulic structures and split flow optimization procedures. The hydraulic roughness was incorporated into the model using Manning's n -values that were assigned based on field observations, bed material characteristics, aerial photography, previous experience with gravel-bed channels similar to Deer Creek and published values for similar streams (Barnes, 1967; Hicks and Mason, 1991; Arcement and Schneider, 1989). Manning's n -values selected for the model ranged from 0.035 to 0.038 in the main channel, based on the very coarse gravel to small cobble-bed material. Roughness values between 0.04 and 0.05 were used for mid-channel and alternate bars depending on the observed substrate, and size and density of vegetation. Manning's n -values for the channel banks ranged from 0.06 to 0.08 to account for vegetation encroachment and irregular bank geometries. Overbank n -values ranged from 0.1 to 0.12 for areas with moderate to heavy vegetation.

2.3.1. Levees

The project reach of Deer Creek has approximately 5.1 miles of project levees that were installed by the Corps of Engineers in 1948 (**Figure 2**). An additional 2.1 miles of non-project levees and berms were identified and surveyed. These levees and berms have been constructed by local landowners. Both sets of levees were incorporated into the model based on survey data, aerial imagery and information from the DTM.

2.3.2. Bridges

Three bridges cross Deer Creek within the modeled reach, including the Union Pacific Railroad (UPRR) Bridge (Sta 122+56), the Highway 99 Bridge (Sta 152+85) and the Leininger Road Bridge (Red Bridge) (Sta 304+49). Each bridge structure was entered into the model using the internal HEC-RAS Bridge/Culvert interface based on diagrams and measurements provided to MEI (CH2M HILL, undated), and modeled using the standard-step energy approach. The cross sectional geometries at the up- and downstream face of the UPRR Bridge and the Hwy 99 Bridge were based on survey data taken approximately 25 feet upstream from the upstream bridge face. Cross sections at the up- and downstream faces of Red Bridge were cut from the

DTM since no in-channel survey information was available at this location. Expansion and contraction coefficients of 0.5 and 0.3, respectively, were used in the vicinity of each of the bridges. A submerged inlet and outlet discharge coefficient (C_d) of 0.8 was used to model pressure flow conditions at Red Bridge.

2.3.3. Diversions

The Stanford-Vina Diversion (SVD) Dam (Sta 240+39) was coded into the model using the HEC-RAS Inline Structure editor based on diagrams and measurements provided to MEI (CH2M HILL, undated). The channel cross section at the upstream face of the dam was cut from the DTM and slightly adjusted to match observed aggradation upstream from the structure.

2.3.4. Split Flows

An island causes split flow in the upstream portion of the study reach between Sta 479+59 and Sta 386+75. The split flow was modeled using the internal HEC-RAS flow optimization procedure with the energy balance option to calculate the flow distribution in each channel for the full range of discharges. The channel geometry for the main channel was developed based on survey data and information from the DTM, and cross sections in the side channel were cut from the DTM. The split-flow optimization procedure was also used to evaluate flow loss at locations where the flow overtops the levees. The HEC-RAS lateral weir structure feature was used to define the elevations along the right levee upstream from Red Bridge and along both levees between Red Bridge and the SVD. The computed discharge that breaks out along the right levee (up- and downstream from Red Bridge) is removed from the model and re-enters the channel upstream from the diversion dam at Sta 241+50, and the flow that breaks out along the left (south) levee is captured by Delaney Slough that delivers the lost flow back to the main channel at Sta 171+00.

3. UNTRIM MODEL DEVELOPMENT

The 2-D hydrodynamic model was developed using the UnTRIM model (Casulli and Zanolli, 2002) following a similar approach to that employed by MacWilliams et al. (2004). The objective of the 2-D model is to provide more detailed information about specific conditions and locations which cannot be accurately represented in the 1-D model. Specifically, the 2-D model can provide increased understanding of existing conditions at multiple branching channels and junctions, such as the junction of Deer Creek with the Sacramento River and the associated backwater effects in Deer Creek, and above Red Bridge where multiple branching channels occur. The 2-D model will also be used to provide a more detailed assessment of flow conditions within the channel under large floods to evaluate the habitat benefits from different alternatives.

3.1. Model Domain and Bathymetry

The model domain for the 2-D UnTRIM model includes the reach of Deer Creek from the confluence with the Sacramento River upstream to the upstream extent of available photogrammetric data, portions of the Deer Creek floodplain relevant to the project, and the Sacramento River from Woodson Bridge upstream to approximately 3.5 miles beyond the mouth of Deer Creek, as shown in **Figure 3**.

While the UnTRIM model can be applied in a fully three-dimensional (3-D) mode, the current application uses only one vertical layer, resulting in a depth-averaged 2-D model. The primary advantage of applying the model in 2-D is that a much larger number of grid cells in the horizontal can be used to represent that bathymetry. Grid resolution within the low-flow channel is typically finer than 20 feet, with resolution increasing to about 50 feet along the edges of the project levees. Along the Sacramento River, grid resolution ranges from 80 feet within the main channel for the Sacramento River and increasing to about 150 feet along the edges of the Sacramento River floodplain. The current model uses approximately 200,000 grid cells in the main channel of lower Deer Creek and the Sacramento River areas of the grid. Additional grid areas will be added to the model as needed to model flood conditions in the Deer Creek floodplain following levee failure and flow conditions under specific project alternatives.

The sources of model bathymetry for the 2-D model are identical to those used in the development of the HEC-RAS model presented in Section 2.1. The primary bathymetry source is the photogrammetry which was completed as part of the Lower Deer Creek Flood Management Study, and includes detailed (2 feet) topography for the lower Deer Creek study area based on photography and ground-control data provided by the California Department of Water Resources (DWR). Sacramento River bathymetry developed for the Sacramento and San Joaquin River Basins Comprehensive Study was merged with this photogrammetry by DWR and is the principal source for the Sacramento River bathymetry used in the model. Because the DEM derived from the bathymetry does not include bathymetry below the water surface, low-flow channel bathymetry data was derived from cross-section and point survey data collected by CH2M HILL in 2004 and by MEI in 2006.

3.2. Boundary Conditions

The primary boundary conditions applied in the 2-D model include the discharge in Deer Creek, the discharge in the Sacramento River, the Sacramento River stage at Woodson Bridge at the downstream limit of the model, and the bed roughness values for each grid cell within the model. The locations of the flow and stage boundaries are shown on Figure 3. The hydraulic roughness was incorporated into the model using a roughness height, z_o . Values were assigned based on field observations, bed-material characteristics, and aerial photography. Bed roughness heights within the channel were assigned to be 0.01 meter, with values on the channel margins of either 0.04 or 0.10 meters, based on observed vegetation density. These values are typical of those applied in river channels and are approximately equivalent to the Manning's n -values applied in the 1-D model.

4. HEC-RAS AND UnTRIM MODEL CALIBRATION

4.1. Calibration Data

Four flow events on December 8, 2004, May 19, 2005, December 31, 2005, and May 3, 2006, provided model calibration criteria (**Table 1**). The 2004 and 2005 events had relatively high-flows from which maximum water-surface elevations were determined from field indicators and previously installed crest gages. Water-surface elevations for the 2006 event were surveyed directly in the field. Discharge levels for both Deer Creek and the Sacramento River were determined from the instantaneous peak discharge recorded at the relevant gages for each event (USGS Gage No. 11383500: Deer Creek near Vina, CA, and USGS Gage No. 11383730:

Table 1. Calibration data for four events in Deer Creek.

a

Date: December 8, 2004		
Deer Creek Discharge - 2,120 cfs		
Sacramento River Discharge - 13,570 cfs		
Mark	Station	Elevation
H1	48395.67	396.92
L5	31309.33	278.43
S4	30462.02	274.52
L4	27316.10	258.84
S1	24101.76	247.03
L3	21280.89	225.85
S3	15422.39	206.22
L2	14260.66	202.44
S2*	12310.26	195.85
L1*	6309.49	175.30

b

Date: May 19, 2005		
Deer Creek Discharge - 2,940 cfs		
Sacramento River Discharge - 67,405 cfs		
Mark	Station	Elevation
H1	48395.67	397.52
L5	31309.33	278.59
S4	30462.02	274.93
L4	27316.10	259.42
S1	24101.76	247.14
L3*	21280.89	225.06
S3	15422.39	206.58
L2	14260.66	202.97
S2*	12310.26	195.75
L1*	6309.49	178.34

c

Date: December 31, 2005		
Deer Creek Discharge - 10,300 cfs		
Sacramento River Discharge - 137,000 cfs		
Mark	Station	Elevation
HWM25	45151.47	376.02
HWM24	39772.69	337.74
HWM23	36716.98	319.52
HWM22	35341.59	308.12
HWM21	31113.03	283.93
HWM20	30664.30	281.61
HWM18	29596.72	275.64
HWM17	29433.52	274.23
HWM16	23599.63	241.75
HWM15	20040.01	225.85
HWM14*	19184.57	220.83
HWM13	16041.39	213.43
HWM11	15723.48	211.74
HWM10	14204.73	206.91
HWM9*	12719.54	200.79
HWM7	11638.27	200.29
HWM6	9726.76	196.76
HWM5	8907.00	194.04
HWM4	6787.37	192.92
HWM3	5874.76	190.57
HWM2	578.51	189.95
HWM1	242.86	189.87

d

Date: May 3, 2006		
Deer Creek Discharge - 1,210 cfs		
Sacramento River Discharge - 19,285 cfs		
Mark	Station	Elevation
HWM8	29595.67	270.46
HWM12	16043.82	206.68
HWM19	12124.33	194.82

*points that were disregarded during calibration process

Sacramento River at Woodson Bridge). The discharge for the 2006 event was estimated by averaging gage data over the period of hours during which the survey was conducted.

4.2. HEC-RAS Calibration

The HEC-RAS model was calibrated by adjusting the Manning's n -values and ineffective flow areas until the computed water-surface elevation reasonably matched the measured stage data (**Figures 4 through 7**). During the calibration process, several calibration points were disregarded because differences between measured and computed elevations were so great that the measured data appear to be unreasonable. Although the difference in measured and computed water-surface elevation is relatively large at some locations (up to 0.9 feet at 10,300 cfs), the difference is generally less than 0.5 feet, with average absolute differences ranging from 0.3 to 0.4 feet. In addition, the relatively even distribution of positive and negative differences indicates that the model does not consistently under- nor over-predict the water-surface elevations. Based on this comparison, the calibrated model appears to accurately predict hydraulic conditions within the project reach over a relatively wide range of flows.

4.3. UnTRIM Calibration

Each of the four calibration flows were modeled using a constant stage at Woodson Bridge, and constant discharges in the Sacramento River and Deer Creek. The model was run until the flow reached steady-state. These steady flows were modeled to provide an equivalent calibration condition to the HEC-RAS calibration simulations. The UnTRIM model was calibrated by adjusting the z_0 roughness and locations until the computed water-surface elevation reasonably matched the measured stage data (**Figures 8 through 11**). During this process, some refinement was made to the low-flow channel bathymetry to improve the representation of the channel to be as consistent as possible with the available data. This process was necessary due to the complexity of merging bathymetry from different data sets collected between 1999 and 2006 into a single representative model geometry. The model calibration entailed comparison to the same set of data used in the HEC-RAS calibration. Similar to the HEC-RAS calibration, several points were discarded because of indications that the data were unreliable. The resulting model calibration shows a similar level of agreement to the HEC-RAS model, with differences between observed and predicted water surface typically less than 0.5 feet, and average absolute differences for each of the four calibration flows ranging from 0.2 to 0.43 feet.

4.4. Summary of Model Calibration Results

The model calibration made use of the best available stage observation data for four flows that occurred between 2004 and 2006. However, some uncertainties should be considered when evaluating the agreement between observations and predictions. Of the four flow events for which stage data were collected, the data for the 1,210-cfs flow should be the most reliable, since the data were direct observations of stage. Both the HEC-RAS and UnTRIM models show very good agreement with observed stage at this discharge. Observations for the 2,120- and 2,940-cfs flow events were made using crest gauges deployed in 2004. However, some issues with specific measurements were reported when the measurements were made, resulting from damaged or missing gauges. As a result, several data values were disregarded (as noted in Table 1). Both models show very good agreement with the remaining calibration data. High-water marks for the 10,300-cfs flow on December 31, 2005, were staked in May 2006 based on debris lines, and the stakes that could be found were then surveyed in late-summer. Given the level of uncertainty associated with these data, deviations between the

model results and measured data should be expected. Both the 1-D and 2-D models show larger differences between predicted and observed stage for these measurements.

Comparisons between aerial photographs taken in 1999 when the photogrammetric data were collected and 2006 show that some reaches of Deer Creek have undergone significant changes during the period spanning the calibration flows. In particular, the 10,300-cfs flow on December 31, 2005, resulted in significant channel adjustments. In some reaches, survey data collected during 2004 may not exactly represent conditions in May 2005, or even the channel geometry at the high water for the 10,300-cfs flood. These ongoing channel changes may also account for some of the differences between observed and predicted high-water levels, though this cannot be directly quantified.

Because the two models were developed and calibrated independently, results from both models that differ significantly from the data in the same direction indicate that there may be potential issues with that data point or with the available bathymetry. This is particularly true for the 10,300-cfs flow. At this flow, HWM 23 is under-predicted by 0.92 and 0.85 feet in the 1-D and 2-D models, respectively, indicating that the two models agree within 0.07 feet. Similarly, at HWM24 water levels are over predicted by 0.64 and 0.59 feet, respectively, and agree within 0.05 feet between the two models. Although agreement between the two models does not definitively show that the data point is not valid, when the two models show cross-agreement but differ from the data, this then suggests that the difference may be due to data uncertainty or that the channel geometry incorporated into both models may not exactly represent the existing geometry at the time of a specific flow event.

Despite these uncertainties resulting from both observation data and composite bathymetry, the overall level of agreement achieved with both models is a strong indication that both models are accurately predicting water levels in Deer Creek for the range of flow conditions that are important to this study. For each of the four flow events simulated, the average absolute error between observed and predicted water levels ranged from 0.2 to 0.43 feet, indicating that both models are suitably calibrated for assessing potential modifications to Lower Deer Creek. The following two sections provide a review of existing conditions to provide a baseline for assessing project alternatives.

5. ANALYSIS OF HEC-RAS MODEL RESULTS

5.1. Hydrology

The HEC-RAS model was executed over a range of discharges from baseflows through the existing conditions 100-year peak discharge. The distribution of flows was established by performing flow-duration and flood-frequency analyses using the mean daily and annual peak flow data from the Deer Creek near Vina, CA gage (USGS Gage No. 11383500) (**Figures 12 and 13**). A comparison of the results of the flood-frequency analysis with results from previously published reports indicates relatively small differences, due primarily to the additional years of data used for the present analysis (**Table 2**). Due to the lack of concurrence between flows in Deer Creek and the Sacramento River, a unique boundary condition at the mouth of Deer Creek does not exist. To determine the range of downstream boundary conditions for the modeled flows in Deer Creek, a relationship was developed between measured concurrent mean daily flows at the Deer Creek near Vina and the Sacramento River at Woodson Bridge gages (**Figure 14**). The Deer Creek flows were sorted into seven bins and the concurrent

median, 10th and 90th percentile flows in the Sacramento River were identified for each bin. The representative Deer Creek flow for each bin was computed using the logarithmic mean of the upper and lower limits of the bin. For peak flows above the range of the mean daily flows, the discharges in the Sacramento River upstream from the confluence were maintained at constant values of 101,260, 122,200, and 141,260 cfs for the 10th percentile, median and 90th percentile flows, respectively, since measured peak flows in Deer Creek are not coincident with peak flows in the Sacramento River. The median value for the Sacramento River was utilized for evaluating the hydraulic conditions in the project reach, while the 10th and 90th percentiles were used to evaluate the backwater effects from the Sacramento River (**Table 3**).

Table 2. Comparison of flood frequency analyses for Deer Creek from three sources.			
Recurrence Intervals	Peak Discharge (cfs)		
	Tompkins, et al. (2005)	Tompkins and Kondolf, (2006)	Current Study
1.5-yr	3,876	3,960	3,900
2-yr	5,371	5,690	5,480
5-yr	9,937	10,100	9,950
10-yr	13,550	13,700	13,400
25-yr	18,700	20,600	18,000
50-yr	22,910	23,822	22,100
100-yr	27,420	27,420	26,300

Table 3. Range of flows for HEC-RAS model execution.				
Description	Deer Creek Discharge (cfs)	Median Sacramento River Discharge (cfs)	10th Percentile Sacramento River Discharge (cfs)	90th Percentile Sacramento River Discharge (cfs)
50% Exceedence	150	9,150	6,120	13,370
25% Exceedence	350	12,140	7,730	21,310
10% Exceedence	650	18,130	9,750	39,430
5% Exceedence	1,000	26,560	13,310	54,750
1% Exceedence	2,500	53,680	25,470	93,820
1.5yr Peak Flow	3,900	69,820	34,590	108,530
2-yr Peak Flow	5,480	87,930	49,110	122,720
5-yr Peak Flow*	9,950	132,210	111,210	151,210
10-yr Peak Flow*	13,400	135,660	114,660	154,660
15-yr Peak Flow*	16,000	138,260	117,260	157,260
25-yr Peak Flow*	18,000	140,260	119,260	159,260
50-yr Peak Flow*	22,100	144,360	123,360	163,360
100-yr Peak Flow*	26,300	148,560	127,560	167,560

*Using constant discharge in Sacramento upstream from confluence with Deer Creek of

122,260 cfs (median), 101,260 cfs (10th percentile), and 141,260 cfs (90th percentile).

Backwater conditions from the Sacramento River were evaluated over a range of flows in Deer Creek (10-percent exceedence flow to the 100-year peak flow) for the corresponding 10th percentile, median and 90th percentile flows in the Sacramento River (**Figure 15**). At the lower range of flows in Deer Creek (10-percent exceedence) with the 90th percentile Sacramento River flow, the backwater effects from the Sacramento River extend upstream for about 7,000 feet above the confluence. For the 2-year peak flow in Deer Creek, with the highest (90th percentile) Sacramento River flow, the backwater influence from the Sacramento River extends about 11,000 feet upstream of the confluence. At the 100-year peak flow in Deer Creek, the backwater influence of the Sacramento River only extends about 6,500 feet upstream of the confluence.

5.2. HEC-RAS Model Results

Model runs were made for a range of flows from the median (50-percent exceedence) mean daily discharge through the 100-year instantaneous flood peak in Deer Creek (Table 3). Water-surface profiles from these runs are shown in **Figures 16a and 16b**. As expected, the model results indicate that hydraulic conditions (main channel velocities, hydraulic depths and top widths) vary considerably through the project reach based on local slope, channel geometry hydraulic roughness conditions, and downstream hydraulic controls (**Figures 17 through 19**). The channel capacity remains relatively consistent throughout the entire reach, containing the 2-year peak discharge (5,480 cfs) (Figures 16a and 16b). Depending on the stage of the Sacramento River, a significant backwater effect is created that can extend upstream from the confluence with Deer Creek (Figure 15).

5.3. Subreach Designations

To facilitate more detailed study and sediment-transport analysis, the project reach of Deer Creek was subdivided into 12 subreaches based on similarity of channel characteristics and the hydraulic model results (**Table 4**). The three bridges and SVD provide natural boundaries because of the imposed hydraulic control and discontinuity between up- and downstream flow conditions. The up- and downstream limits of the split flow at Sta 479+72 and Sta 386+77 are obvious boundaries, as well. Additional subreach boundaries were selected at Sta 203+48, Sta 277+41, Sta 293+64 and Sta 442+02 based on trends in the hydraulic conditions. Reach-averaged hydraulic results are provided in **Figures 20 through 22**, and **Tables 5 through 7**.

5.4. Levee Capacity

The Corps of Engineers levee project (USACE, 1999; Figure 2) was designed to contain a peak discharge of 21,000 cfs with 3 feet of freeboard. Three elements were identified as project levees. PL1 runs along the left bank from Sta 385+00 to the SVD at Sta 240+38. PL2 runs along the right bank, beginning just above Red Bridge (Sta 314+00) and extending downstream to the SVD. PL3 runs along the left bank beginning above Highway 99 (Sta 175+00) and terminating at the UPRR Bridge (Sta 152+84). The modeled water-surface elevations at a 21,000-cfs flow indicates that the flow is primarily contained below the top-of-levee, but encroaches into the 3-foot freeboard over nearly the entire length of PL1 (**Figure 23**). Overtopping of the levee occurs at Sta 369+92, and below Red Bridge between Sta 280+08 and Sta 263+82. Similarly, PL2 contains the design flow while failing to maintain the 3-foot freeboard, with overtopping immediately below Red Bridge between Sta 297+67 and Sta 291+20 and between Sta 280+08 and Sta 263+82 (**Figure 24**). PL3 contains the design flow

with 3 feet or more of freeboard for nearly the entire length; however, there is only approximately 2 feet of freeboard between Highway 99 and UPRR Bridge (Sta 141+18 to Sta 133+69) (**Figure 25**).

Table 4. Subreach designations.				
Subreach	Downstream Station (ft)	Upstream Station (ft)	Description	Average Slope (ft/ft)
12	0	12255.86	RR to Sacramento River	0.0023
11	12255.86	15284.7	Hwy 99 to Railroad Trestle	0.0016
10	15284.7	20348.37	Channel to Hwy 99	0.0029
9	20348.37	24038.89	Constriction below Stanford Vina	0.0066
8	24038.89	27741.8	Channel to Stanford Vina	0.0054
7	27741.8	29364.66	Expansion below Red Bridge	-0.2162
6	29364.66	30448.94	Constriction below Red Bridge	2.2744
5	30448.94	33403.98	Monitor Reach	0.0074
4	33403.98	38677.37	Downstream of Junction	0.0066
3	38677.37	44202.49	Main Channel to Junction	0.007
2	44202.49	47972.33	Multi-thread Channel	0.0081
1	47972.33	50000	Head to Split Flow	0.0072

Table 5. Subreach-averaged main channel velocity under existing conditions.												
Discharge	Subreach-averaged Velocity (ft/s)											
	1	2	3	4	5	6	7	8	9	10	11	12
50% Exceedence	2.1	3.4	3.0	2.8	3.5	2.2	2.4	2.7	2.2	2.5	2.3	2.5
25% Exceedence	2.9	4.4	3.6	3.6	4.1	2.6	3.2	3.2	2.8	3.1	2.8	2.8
10% Exceedence	3.6	4.9	4.1	4.2	4.5	2.7	3.9	3.3	3.3	3.0	3.1	3.0
5% Exceedence	4.1	5.5	4.0	4.7	5.0	3.1	4.5	3.8	3.8	3.9	3.4	3.2
1% Exceedence	5.5	6.1	5.0	5.4	6.0	4.7	6.0	4.7	5.1	4.7	4.1	3.2
1.5-yr Peak Flow	6.2	6.7	5.6	5.9	6.8	5.6	6.9	5.4	5.8	5.5	4.7	3.4
2-yr Peak flow	6.9	7.1	6.1	6.5	7.4	6.6	7.5	5.9	6.5	6.1	5.2	3.6
5-yr Peak Flow	7.8	7.0	7.1	7.7	8.5	8.7	8.9	7.2	8.1	7.1	6.1	4.1
10-yr Peak flow	8.1	7.7	7.7	8.4	8.7	10.0	9.5	7.9	8.9	7.7	6.5	4.7
15-yr Peak Flow	10.2	7.9	8.4	9.0	9.1	10.8	10.1	8.3	9.3	8.0	6.7	5.2
25-yr Peak Flow	10.1	8.1	8.7	9.3	9.2	11.4	10.5	8.5	9.5	8.1	6.8	5.5
50-yr Peak Flow	10.7	8.5	9.3	10.0	9.8	12.2	11.1	8.5	9.9	8.4	6.9	6.0
100-yr Peak flow	12.3	9.0	9.8	10.6	10.4	12.9	11.4	8.4	10.2	8.7	7.0	6.4

Table 6. Subreach-averaged main channel hydraulic depth under existing conditions.												
Discharge	Subreach-averaged Hydraulic Depth (ft)											
	1	2	3	4	5	6	7	8	9	10	11	12
50% Exceedence	0.85	0.73	0.74	0.66	0.79	1.01	0.75	0.61	0.91	0.7	0.9	1.0
25% Exceedence	1.3	1.1	1.1	1.0	1.1	1.7	1.2	0.9	1.4	1.1	1.4	1.6
10% Exceedence	1.7	1.5	1.3	1.3	1.4	2.3	1.6	1.4	1.8	1.6	1.9	2.4
5% Exceedence	2.1	1.8	1.4	1.6	1.8	2.9	2.0	1.7	2.2	1.8	2.2	2.9
1% Exceedence	3.3	2.1	2.1	2.5	3.1	4.3	3.3	3.0	3.5	3.1	3.7	4.7
1.5-yr Peak Flow	4.1	2.5	2.6	3.3	4.1	5.3	4.2	3.8	4.5	3.9	4.6	6.2
2-yr Peak flow	4.6	2.9	2.9	4.1	5.0	6.3	4.8	4.6	5.3	4.7	5.3	7.7
5-yr Peak Flow	6.3	4.1	4.0	5.8	7.3	8.2	6.4	6.4	7.0	6.6	7.4	11.3
10-yr Peak flow	7.4	4.5	4.6	6.9	9.0	9.4	7.2	7.5	7.9	8.0	8.9	12.6
15-yr Peak Flow	7.1	5.0	5.4	7.6	10.0	10.2	7.8	8.1	8.5	8.8	10.0	13.5
25-yr Peak Flow	7.7	5.1	5.8	8.2	10.8	10.7	8.3	8.3	8.8	9.3	10.7	14.1
50-yr Peak Flow	7.1	5.3	6.6	9.2	12.0	11.8	8.9	8.6	9.5	10.2	12.2	15.4
100-yr Peak flow	7.2	5.7	7.2	10.1	12.8	12.3	9.1	8.8	10.2	11.2	13.5	16.4

Table 7. Subreach-averaged main channel top with under existing conditions.												
Discharge	Subreach-averaged Top Width (ft)											
	1	2	3	4	5	6	7	8	9	10	11	12
50% Exceedence	83.38	60.76	68.66	80.62	54.34	68.61	81.60	92.21	75.11	85.1	69.2	61.3
25% Exceedence	90.8	70.7	85.4	99.1	80.1	80.7	95.5	118.7	93.0	107.4	88.1	76.0
10% Exceedence	104.3	90.7	122.2	117.8	101.9	105.2	103.6	146.7	109.9	133.6	110.5	90.4
5% Exceedence	118.5	101.5	170.2	129.7	113.8	111.6	109.3	158.8	117.9	145.5	135.4	108.2
1% Exceedence	136.5	181.9	220.2	182.5	134.4	125.5	124.7	180.8	138.5	168.9	166.6	166.9
1.5-yr Peak Flow	154.5	214.9	249.2	197.2	141.6	130.5	132.7	191.5	150.2	178.9	180.0	186.6
2-yr Peak flow	174.3	234.9	275.5	205.7	146.0	133.0	148.6	201.7	160.3	187.9	195.2	195.9
5-yr Peak Flow	196.0	254.9	301.3	218.0	150.9	138.5	155.4	209.9	175.8	206.2	213.7	210.0
10-yr Peak flow	199.4	257.3	311.0	222.6	153.0	139.8	155.4	210.0	185.9	207.5	219.4	214.7
15-yr Peak Flow	201.7	258.9	316.8	224.8	153.5	139.8	155.4	210.0	187.6	208.1	220.1	215.9
25-yr Peak Flow	202.8	259.2	317.4	225.8	153.6	139.8	155.4	210.0	188.3	208.5	220.1	216.1
50-yr Peak Flow	216.9	259.2	317.9	227.2	153.7	139.8	155.4	210.0	189.0	208.9	220.1	216.5
100-yr Peak flow	217.0	259.3	317.9	228.2	153.7	139.8	155.4	210.0	189.9	209.2	220.2	216.8

Several other features were identified as non-project levees (Figure 2). In the absence of design criteria, the conveyance capacity of each feature was estimated as the level at which the lowest point is overtopped. NPL1 is a series of three berms that extend upstream from the most upstream project levee (PL1). The berms [NPL1 (a), (b) and (c) from upstream to downstream] are discontinuous and raise the elevations of locally low areas between areas of naturally higher ground. NPL1(a) fully contains all flows up to the 100-year discharge. NPL1(b) is high enough to contain the 100-year discharge, though the discontinuity between (a) and (b) allows flow to break-out behind (b) at flows above 17,000 cfs. NPL1(c) fully contains the 100-year discharge, though the discontinuity between (b) and (c) allows flow to pool behind (b) at flows above the 50-percent exceedence level.

NPL2 is located on the south bank downstream of the SVD (Sta 191+00 to Sta 214+50). This feature fully contains the 100-year discharge, with freeboard ranging from 3 to 11 feet. A discontinuity occurs in the middle of the levee; however, that allows water to pond behind the structure. NPL3 begins just upstream of the Highway 99 Bridge on the north bank and terminates at the bridge. This levee is overtopped by flows above 20,000 cfs. NPL4, located on the south bank inside of PL3 just downstream of the Highway 99 Bridge, consists of two discontinuous berms. The upstream berm is overtopped by flows above 18,000 cfs. However, the discontinuity allows flows above 10,000 cfs spill into the lower ground behind the berm. The downstream berm is significantly lower in elevation than the upstream berm, and is overtopped by flows above 6,000cfs. NPL5 is located immediately downstream of the UPRR Bridge on the north bank. This levee is overtopped at flows above 24,000 cfs.

5.5. Backwater Effects

In addition to the Sacramento River (Figure 15), several structures within the project reach also create significant backwater effects. At both the UPRR Bridge and SVD, upstream backwater occurs at the lowest flows evaluated in the model. Red Bridge creates a noticeable backwater effect at flows greater than 11,000 cfs. Highway 99 has no noticeable effect at any flow level. Neither the project levees nor non-project levees create a noticeable backwater effect at any flow level.

6. ANALYSIS OF UNTRIM MODEL RESULTS

The four sets of flow conditions used in the model calibration demonstrate some of the important hydraulic characteristics of lower Deer Creek. The discharges on the Sacramento River for the four calibration flows range from 13,570 to 137,000 cfs, while the discharges simulated on Lower Deer Creek range from 1,210 to 10,300 cfs. The water surface predicted by the 2-D UnTRIM model for each of the four calibration flow conditions are provided in **Figures 26 through 29**. The significant features of these flow conditions are discussed in the following sections.

6.1. Sacramento River

Water levels in the lower reach of Deer Creek are strongly influenced by the discharge in the Sacramento River. For the lowest two Discharges in Deer Creek that were simulated, the concurrent discharge in the Sacramento River is also relatively low (Figure 26 and 27). For both the 13,570 cfs (Figure 27) and the 19,285 cfs (Figure 26) flows, Sacramento River flow is contained within the primary channel of the Sacramento River, and mid-channel bars are visible for both flows. Water-surface profiles along lower Deer Creek for these flows show relatively little backwater effects in Deer Creek (Figures 10 and 11). At a discharge of 67,405 cfs significant additional areas of the Sacramento River floodplain are flooded relative to the lower flows (Figure 28). In addition, significant backwater occurs in the lower reach of Deer Creek as seen in Figure 9. When comparing the water surface in the lower reach of Deer Creek for 2,120 cfs (Figure 10) and 2,940 cfs (Figure 9) a significantly different water-surface profile is evident in approximately the lower two miles of Deer Creek due to backwater effects from the Sacramento River. At a discharge of 137,000 cfs vast portions of the Sacramento River floodplain are flooded (Figure 29) and backwater develops for more than two miles up the lower reach of Deer Creek (Figure 8). As a result of these backwater effects, it appears that flooding on the lower

two to three miles of lower Deer Creek is dominated by the effect of flows in the Sacramento River.

6.2. Sacramento River to Red Bridge

The reach of lower Deer Creek from the Monastery to approximately two miles above Red Bridge is primarily contained by project levees. Flow within this reach occurs dominantly within the main channel, with some small channels around in-channel bars becoming active with increasing discharge. A detailed discussion of channel and levee capacity within this reach is provided in Section 5.

6.3. Deer Creek Above Red Bridge

Above the project levees, the upper reach of lower Deer Creek included in the model demonstrates significantly different properties with increasing discharge than the levee reach. At a discharge of 1,210 cfs (Figure 26), flow in this reach is mostly within the principal channel, though some small side channels are active. With increasing discharge to 2,120 (Figure 27) and 2,940 cfs (Figure 28) additional side channels become active and the reach typifies characteristics of a braided stream. At 10,300 cfs (Figure 29), the flow spreads out into a large network of braided channels through the upper reach.

7. SUMMARY AND CONCLUSIONS

A 1-D HEC-RAS hydraulic model was developed for the project reach of Lower Deer Creek using the Corps of Engineers HEC-RAS computer software (USACE, 2005) and a 2-D hydrodynamic model was developed using UnTRIM (Casulli and Zanolli, 2002). The geometry for the two models was based on surveyed data collected by CH2M HILL in 2004 and MEI in 2006, and information from a DTM developed by DWR. Hydrologic input to the model was based on discharge records from USGS Gage No. 11383500 at Deer Creek near Vina, CA, and a study of historical concurrent flows in the Sacramento River (USGS Gage No. 11383730 at Sacramento River at Woodson Bridge). The downstream boundary condition was set by the stage of the Sacramento River at Woodson Bridge. Other model inputs (hydraulic roughness, non-conveyance flow areas) were based on field observations and engineering judgment. The HEC-RAS and UnTRIM models were calibrated using stage observations for four discharges in lower Deer Creek ranging from 1,210 to 10,300 cfs.

The model results indicate the main channel is able to contain flows up to the 2-year peak discharge (5,480 cfs). The project reach was divided into 12 subreaches based on the locations of control structures and trends in hydraulic parameters. Model results indicate levees constructed by the Corps of Engineers in 1948 do not contain the design discharge of 21,000 cfs with the prescribed 3 feet of freeboard. Significant backwater effects are created by the Sacramento River, the UPRR Bridge and Red Bridge at flows above 11,000 cfs.

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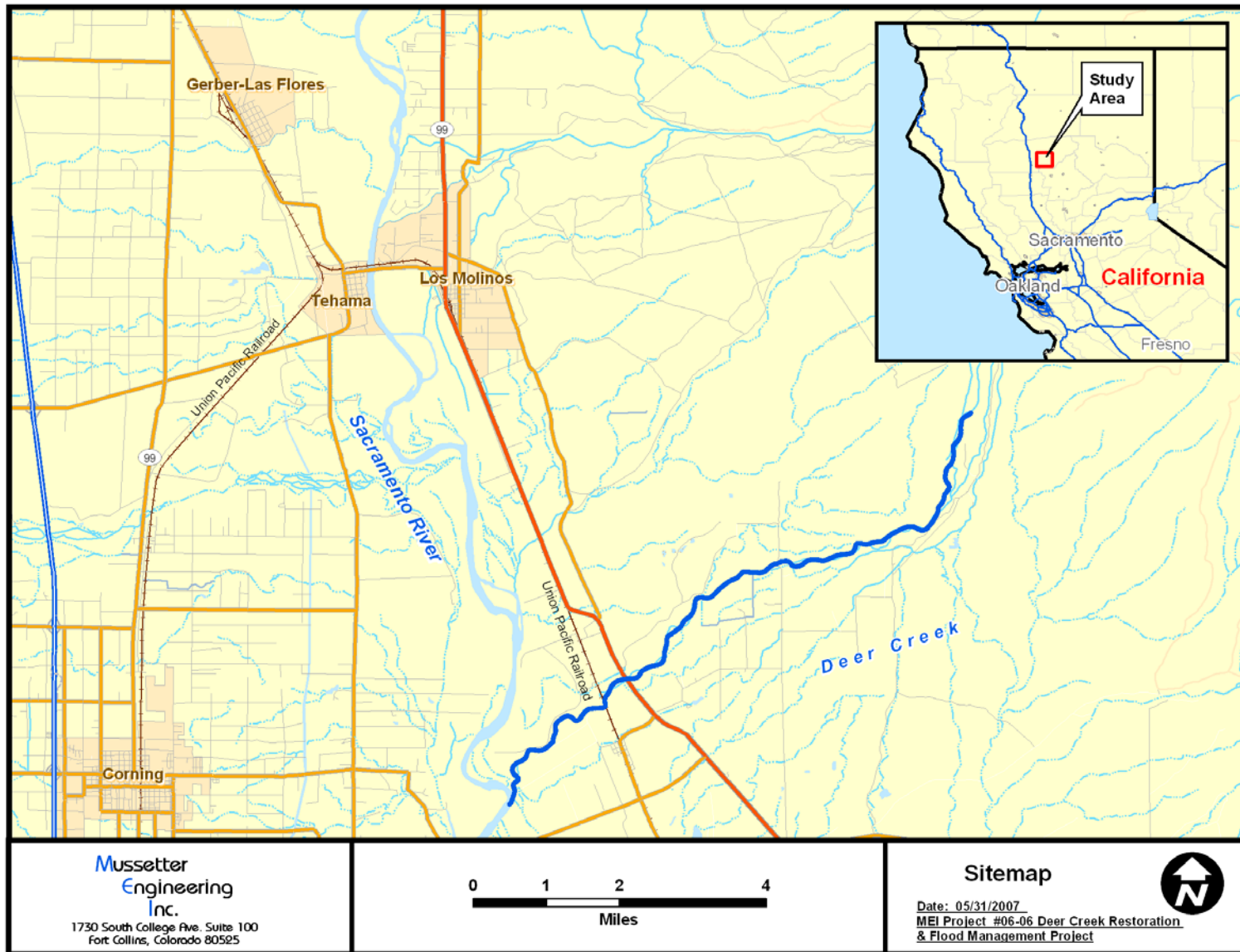


Figure 1. Vicinity map of Deer Creek project reach.

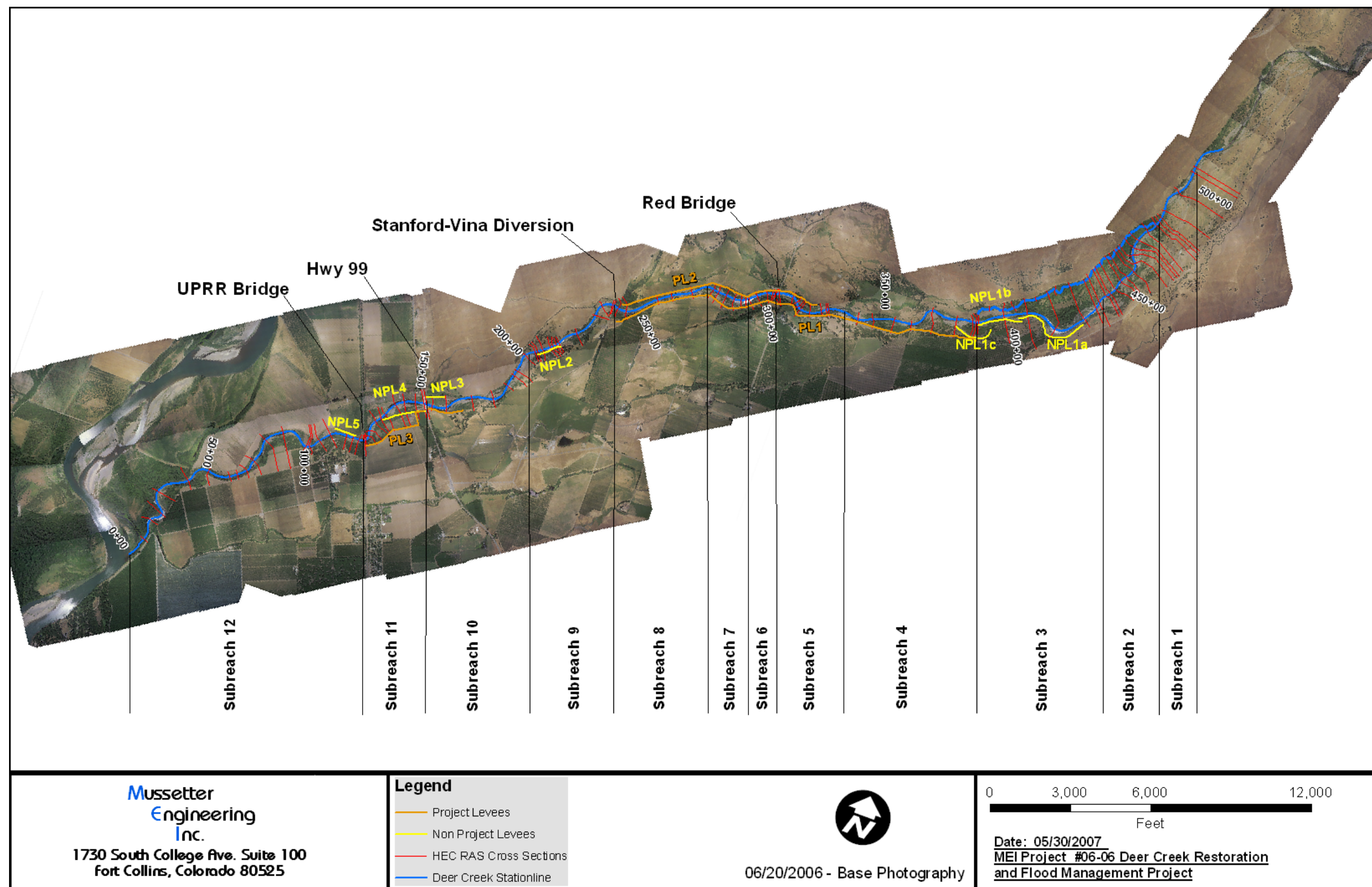


Figure 2. Aerial photograph with identified landmarks and levees of Deer Creek project reach.

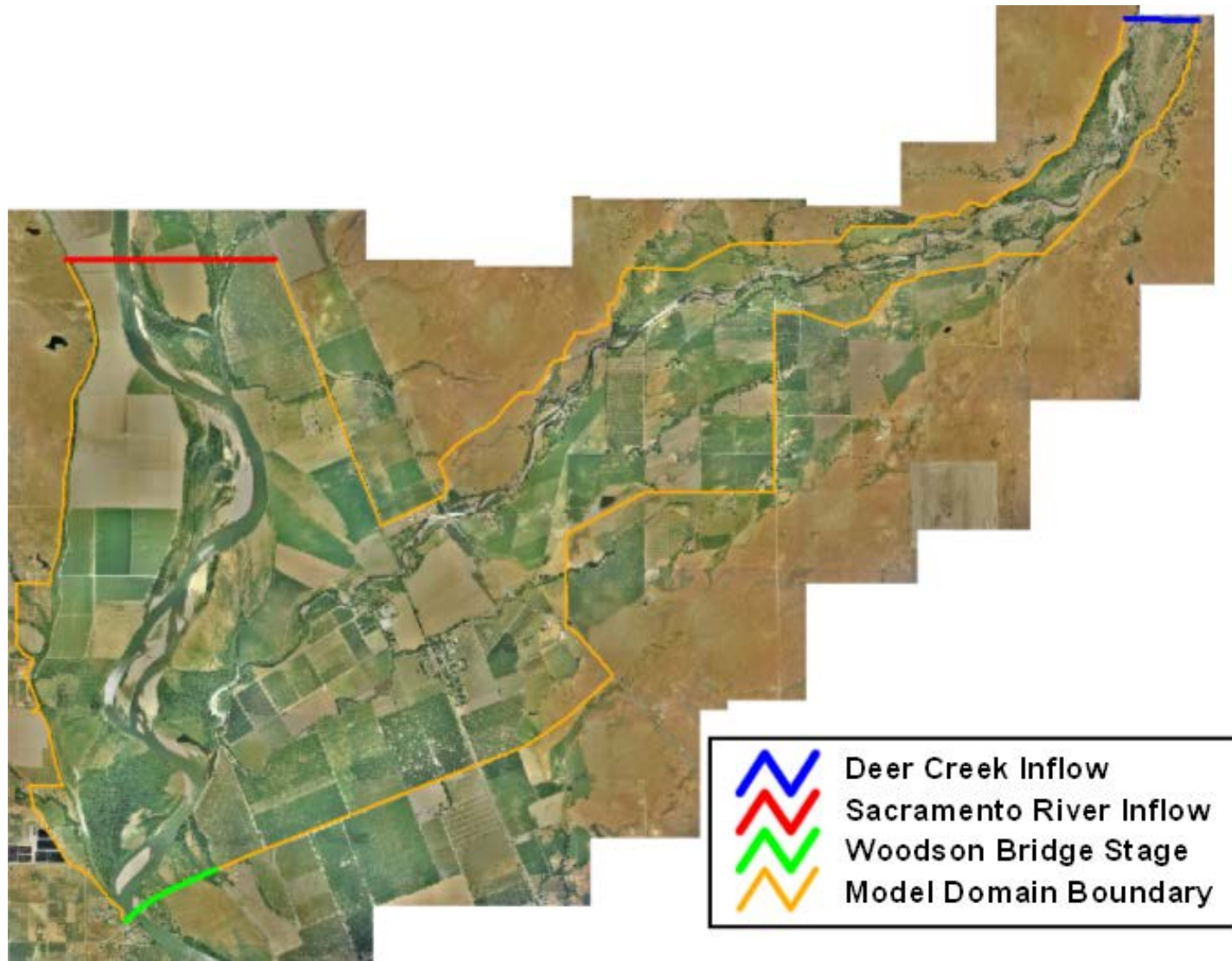


Figure 3. 2-D UnTRIM Model Domain and Boundaries for Lower Deer Creek and Sacramento River.

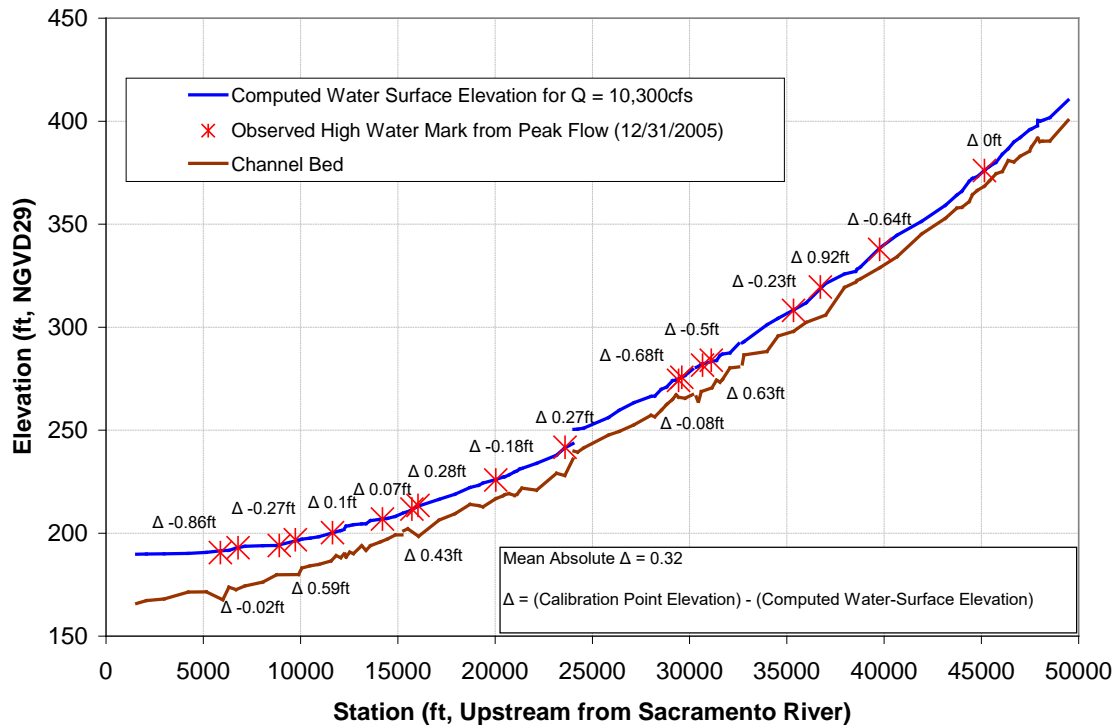


Figure 4. Computed HEC-RAS water-surface elevation at $Q = 10,300 \text{ cfs}$ and observed high-water marks from flow even on December 31, 2005.

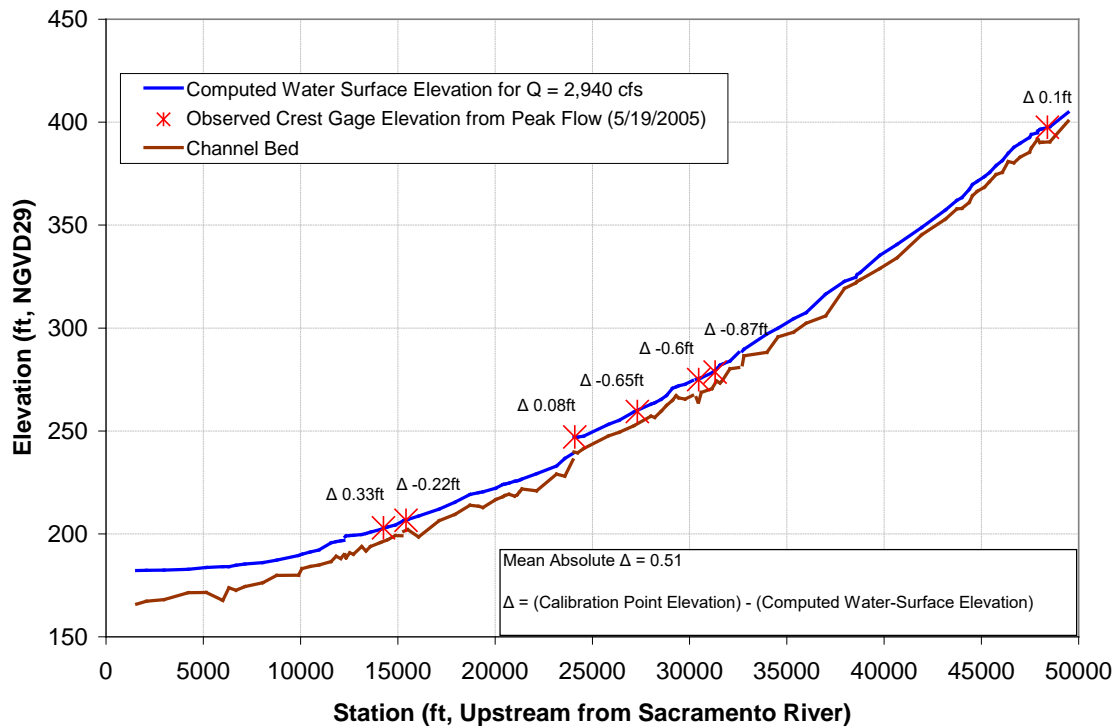


Figure 5. Computed HEC-RAS water-surface elevation at $Q = 2,940 \text{ cfs}$ and crest gage elevations from flow event on May 19, 2005.

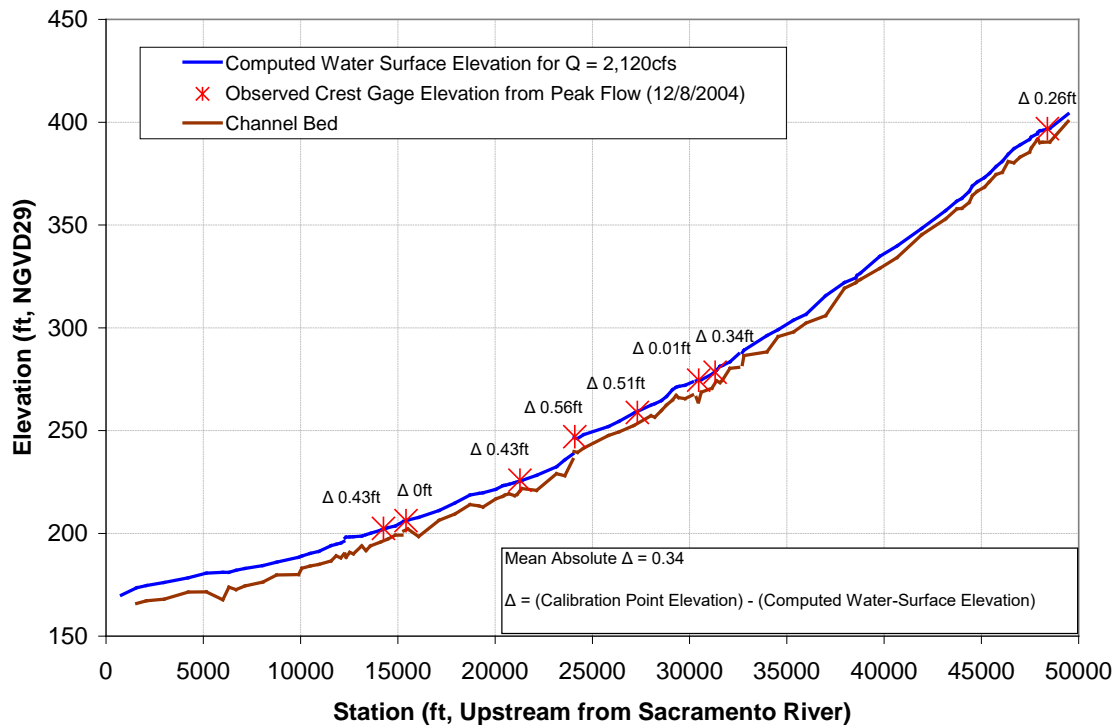


Figure 6. Computed HEC-RAS water-surface elevation at $Q = 2,120$ cfs and crest gage elevation from flow event on December 8, 2004.

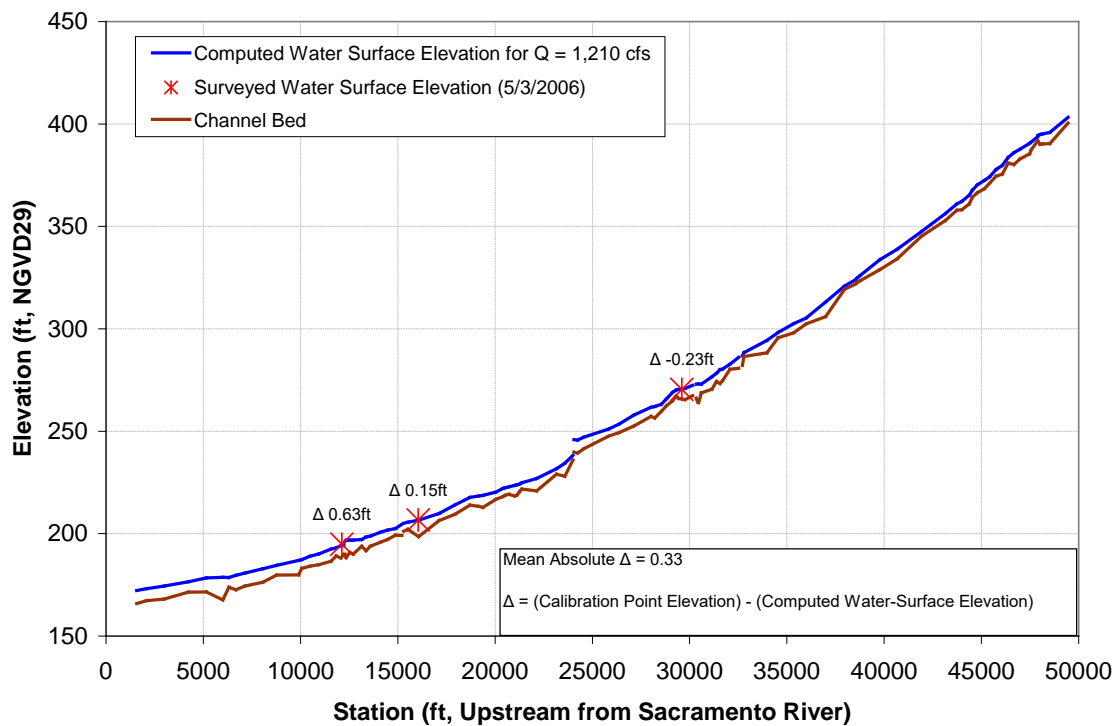


Figure 7. Computed HEC-RAS water-surface elevation at $Q = 1,210$ cfs and surveyed water-surface elevations on May 3, 2006.

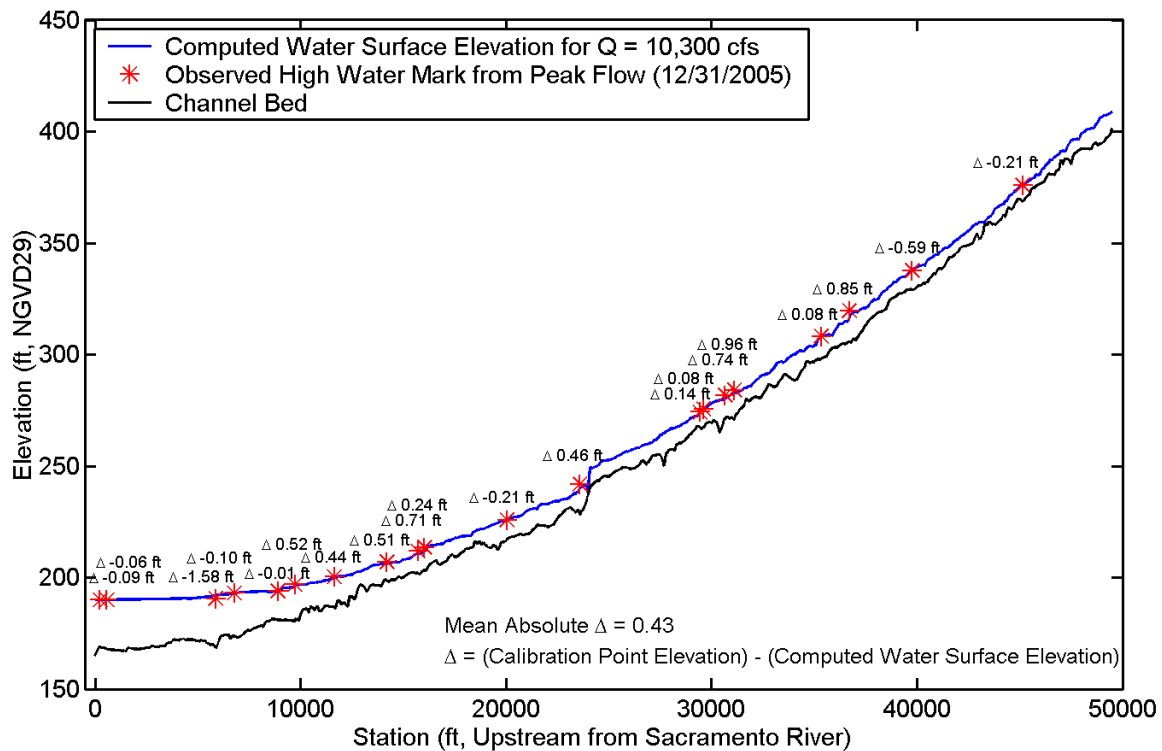


Figure 8. Computed UnTRIM water-surface elevation at $Q = 10,300$ cfs and observed high-water marks from flow even on December 31, 2005.

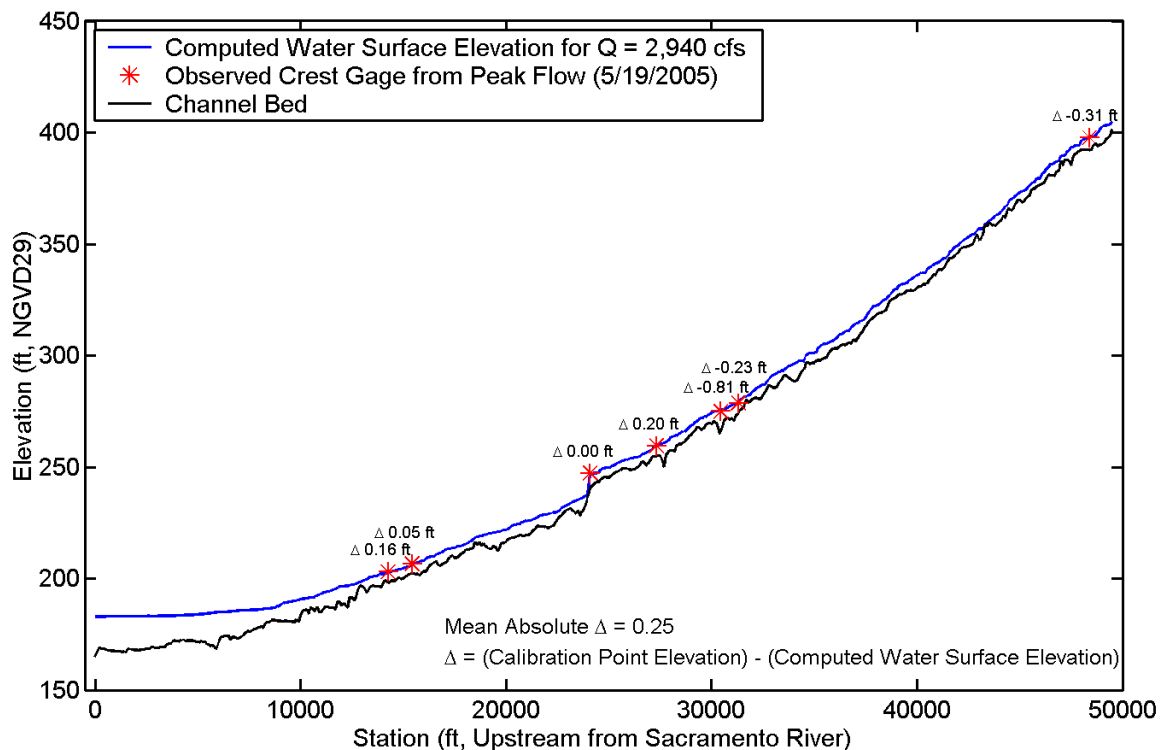


Figure 9. Computed UnTRIM water-surface elevation at $Q = 2,940$ cfs and crest gage elevations from flow event on May 19, 2005.

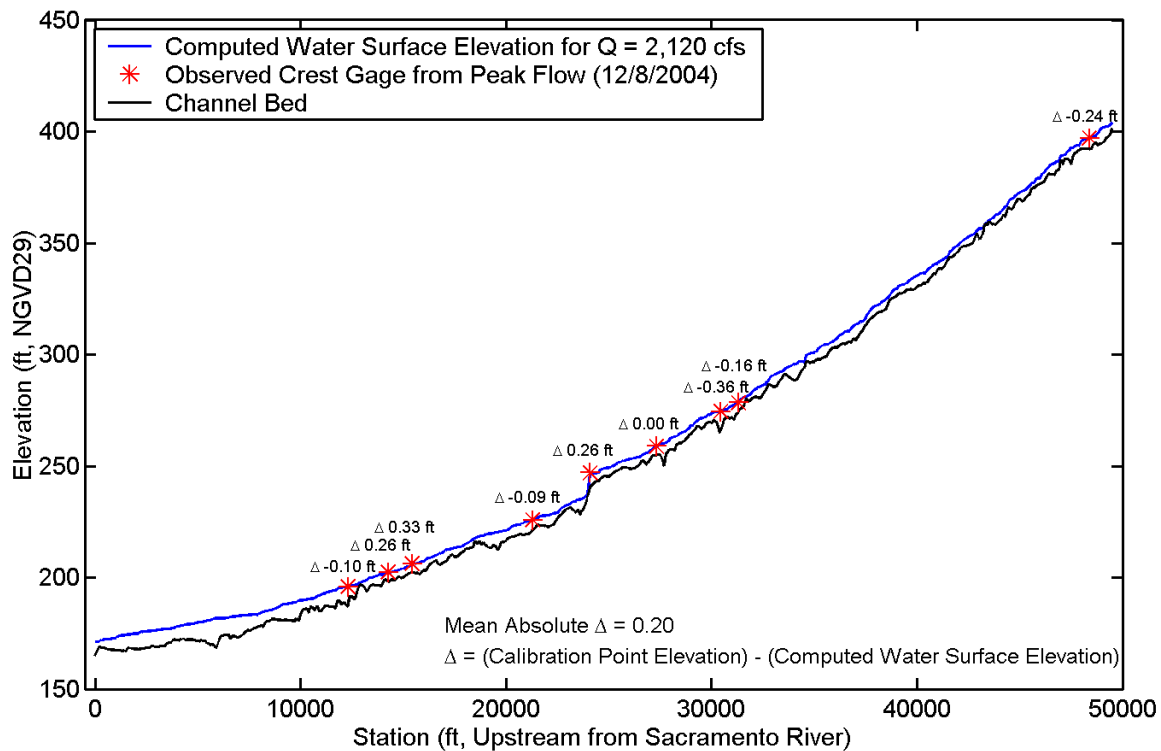


Figure 10. Computed UnTRIM water-surface elevation at $Q = 2,120$ cfs and crest gage elevation from flow event on December 8, 2004.

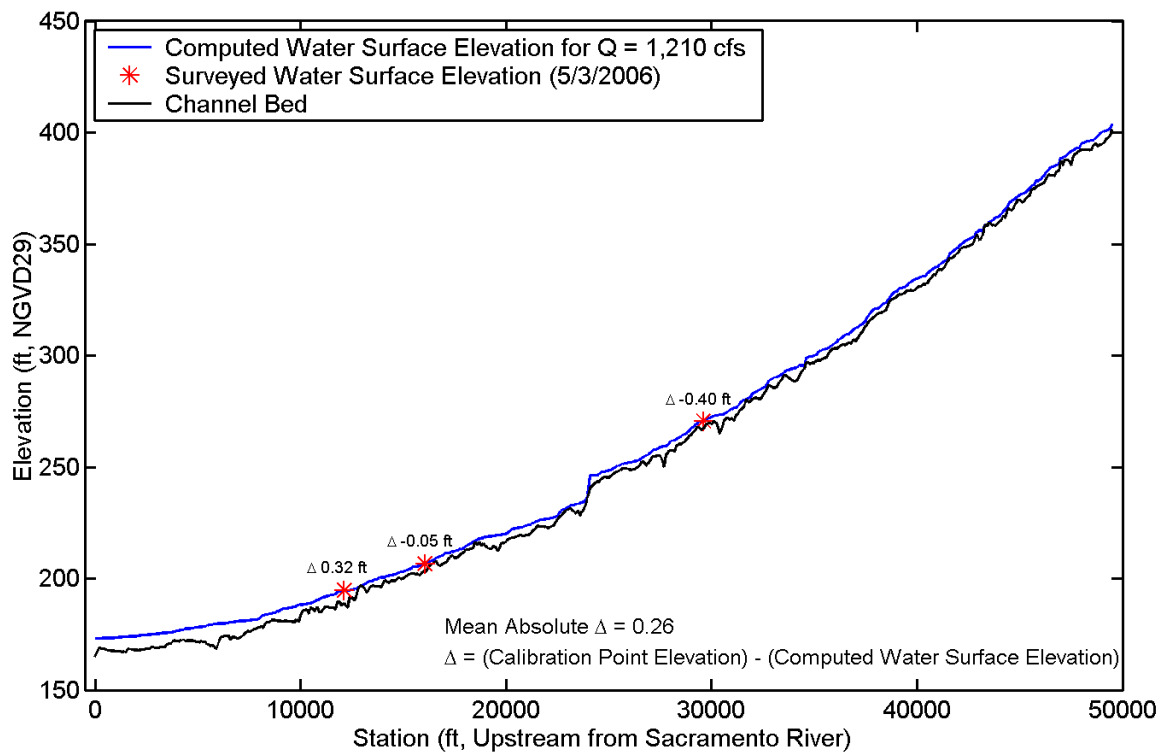


Figure 11. Computed UnTRIM water-surface elevation at $Q = 1,210$ cfs and surveyed water-surface elevations on May 3, 2006.

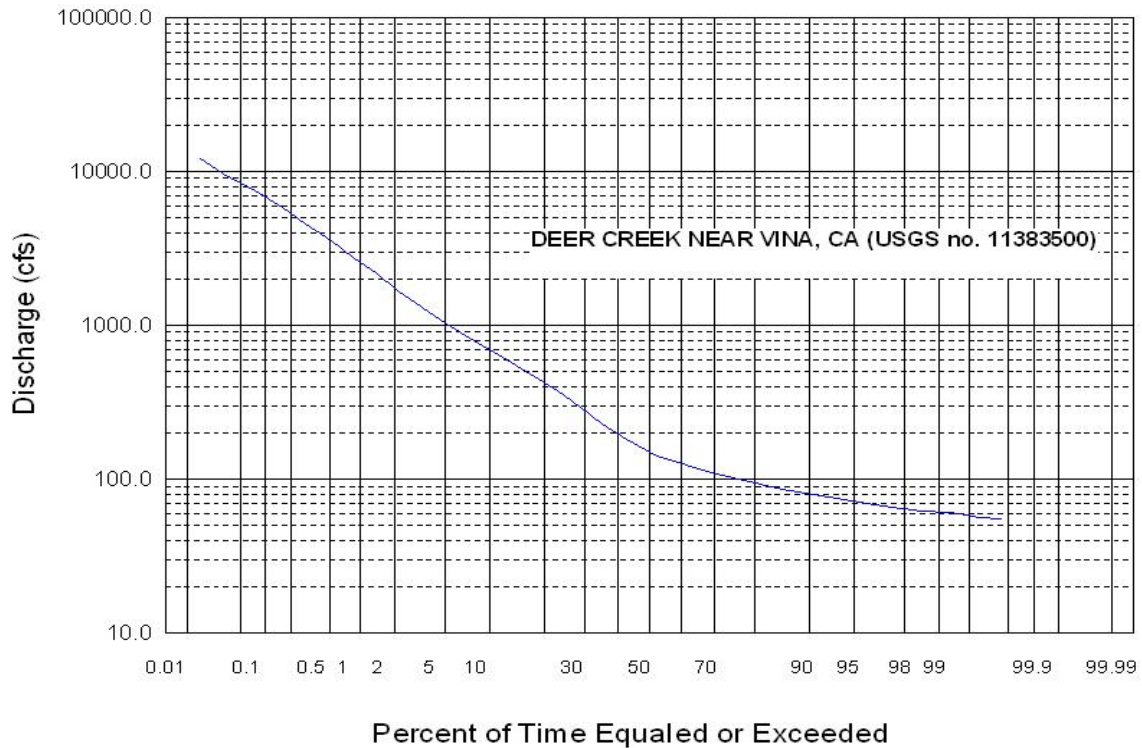


Figure 12. Flow-duration analysis of mean daily average flows from USGS Gage No. 11383500 (Deer Creek near Vina, CA), October 1, 1911 through November 6, 2006.

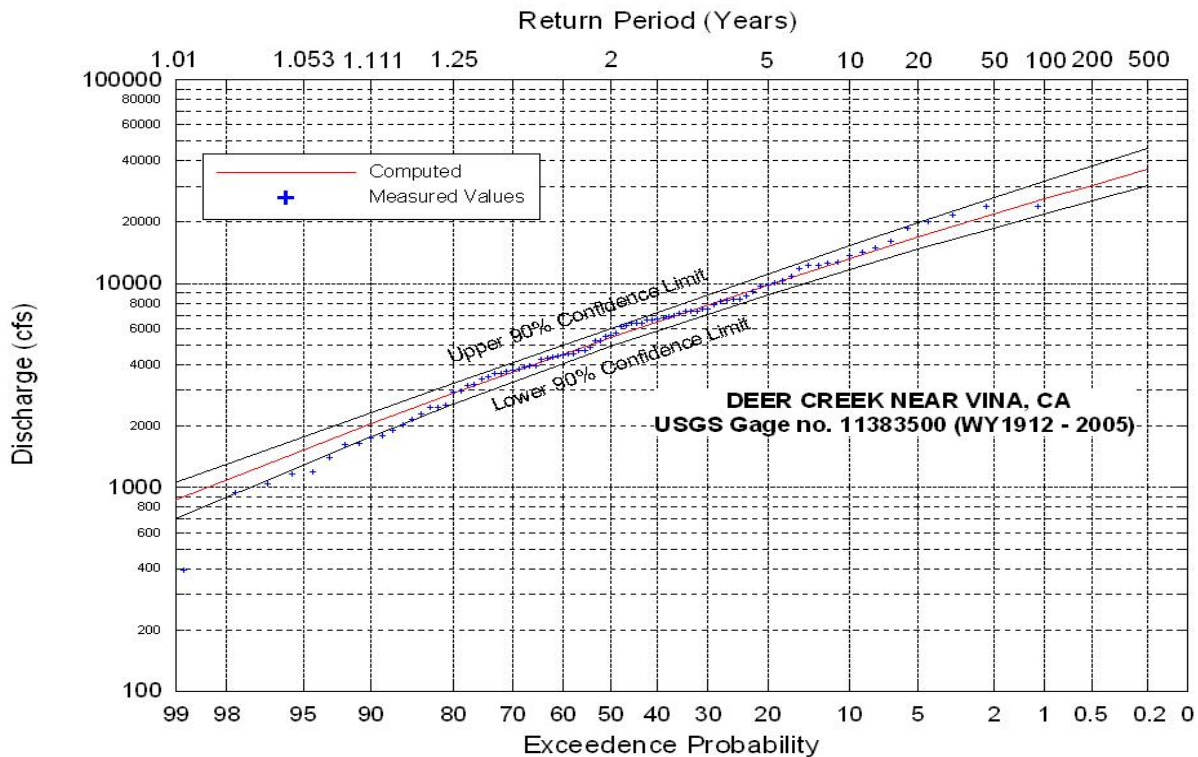


Figure 13. Flood-frequency analysis of annual peak flows from USGS Gage No. 1138350 (Deer Creek at Vina, CA), WY1912 through WY2005.

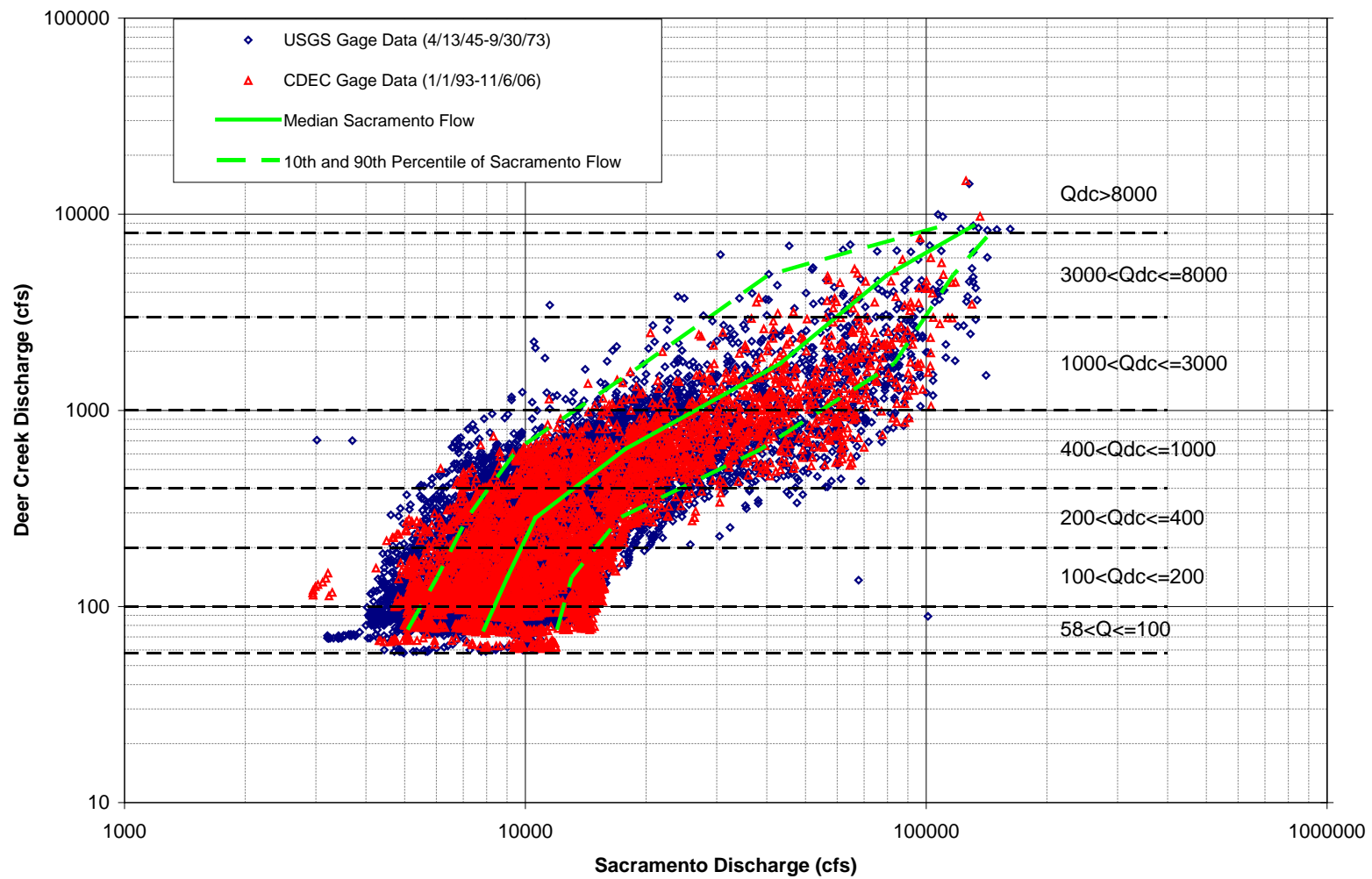


Figure 14. Relationships (median, 10th, and 90th percentile) of measured coincident mean daily flows in Deer Creek and the Sacramento River at Woodson Bridge.

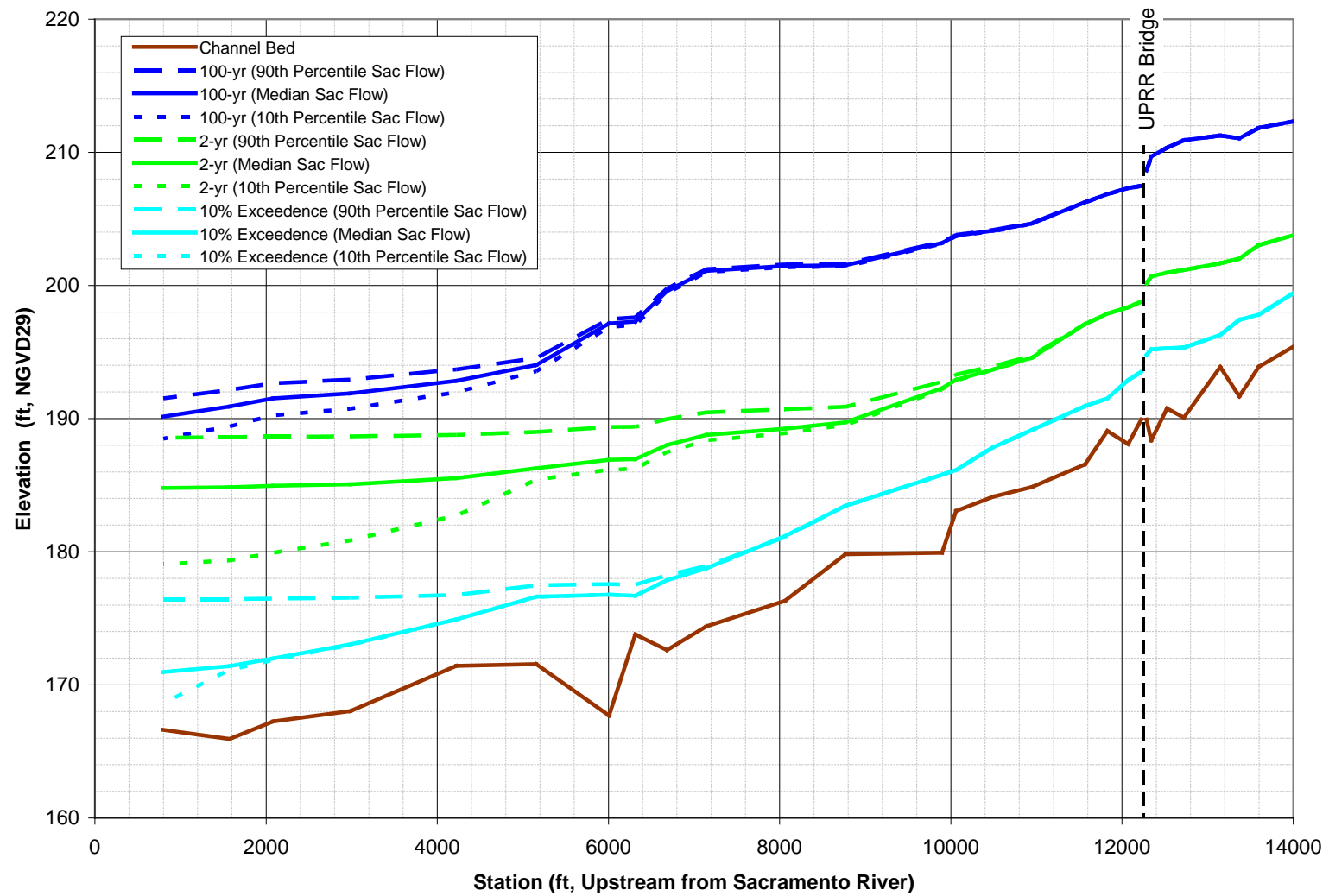


Figure 15. Water-surface profiles in Subreach 12 of Deer Creek for a range of flows in Deer Creek and three boundary conditions in the Sacramento River.

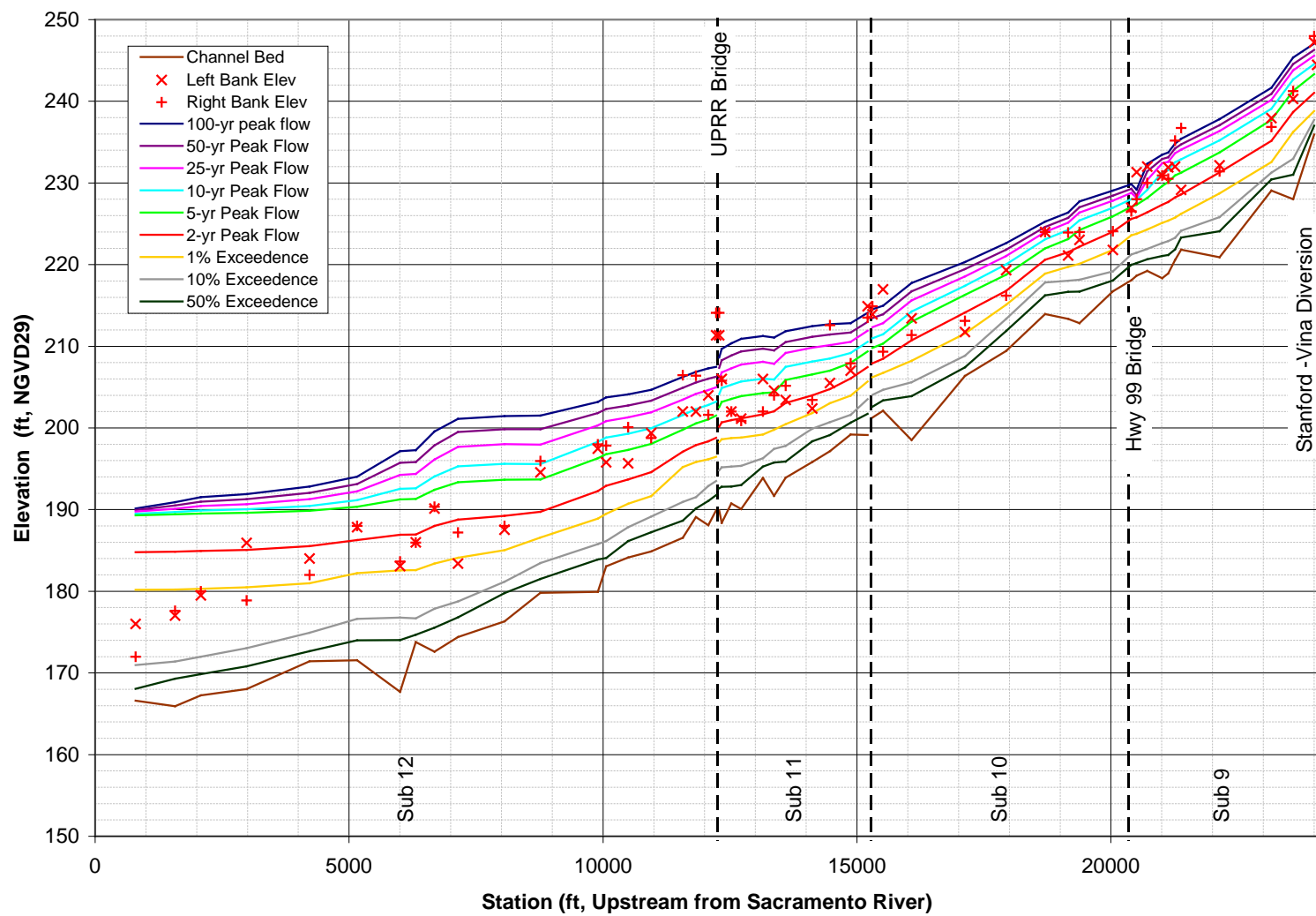


Figure 16a. Computed water-surface profiles under existing conditions below Stanford-Vina Diversion Dam.

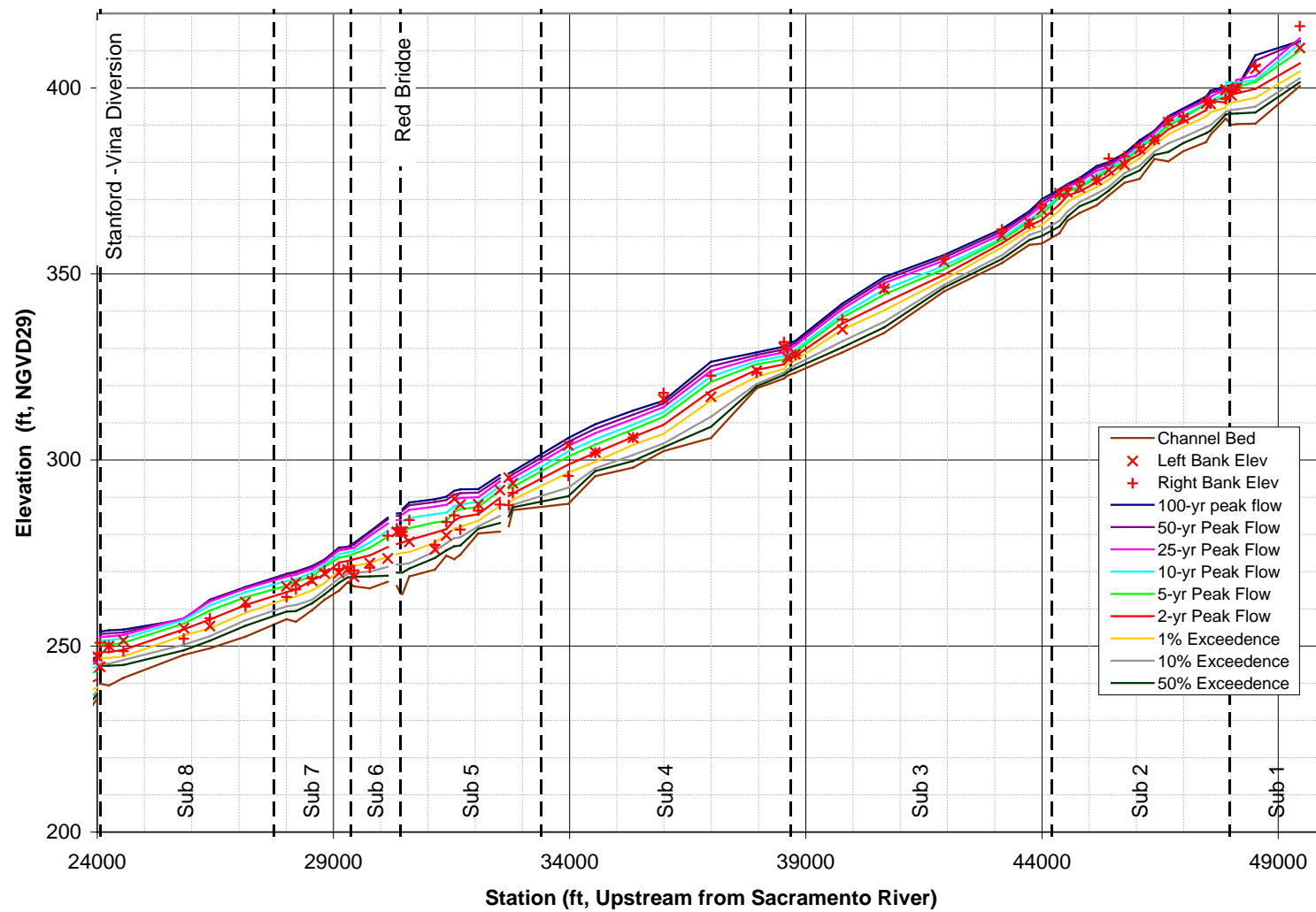


Figure 16b. Computed water-surface profiles under existing conditions above Stanford-Vina Diversion Dam.

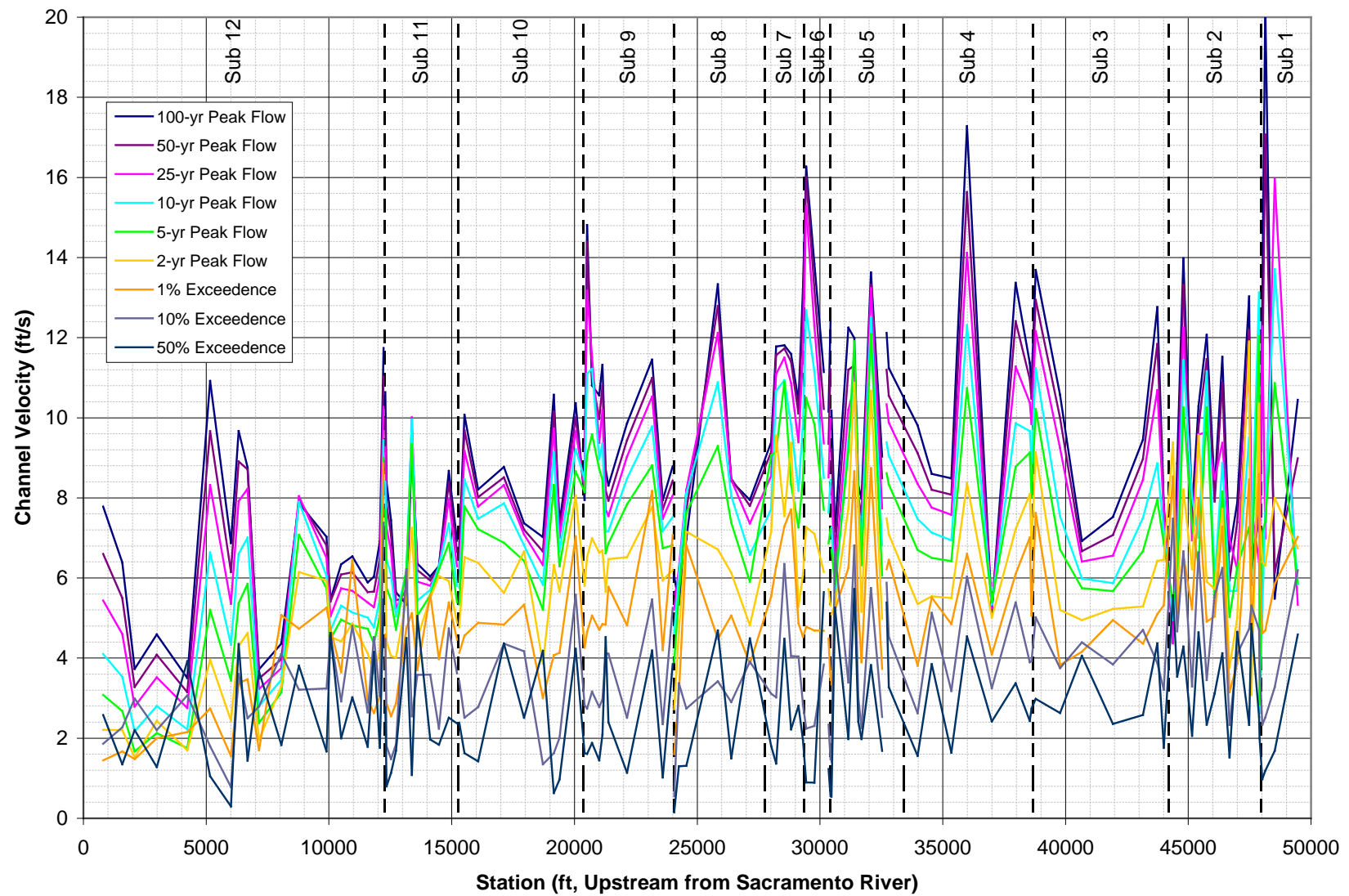


Figure 17. Computed channel velocity profiles under existing conditions.

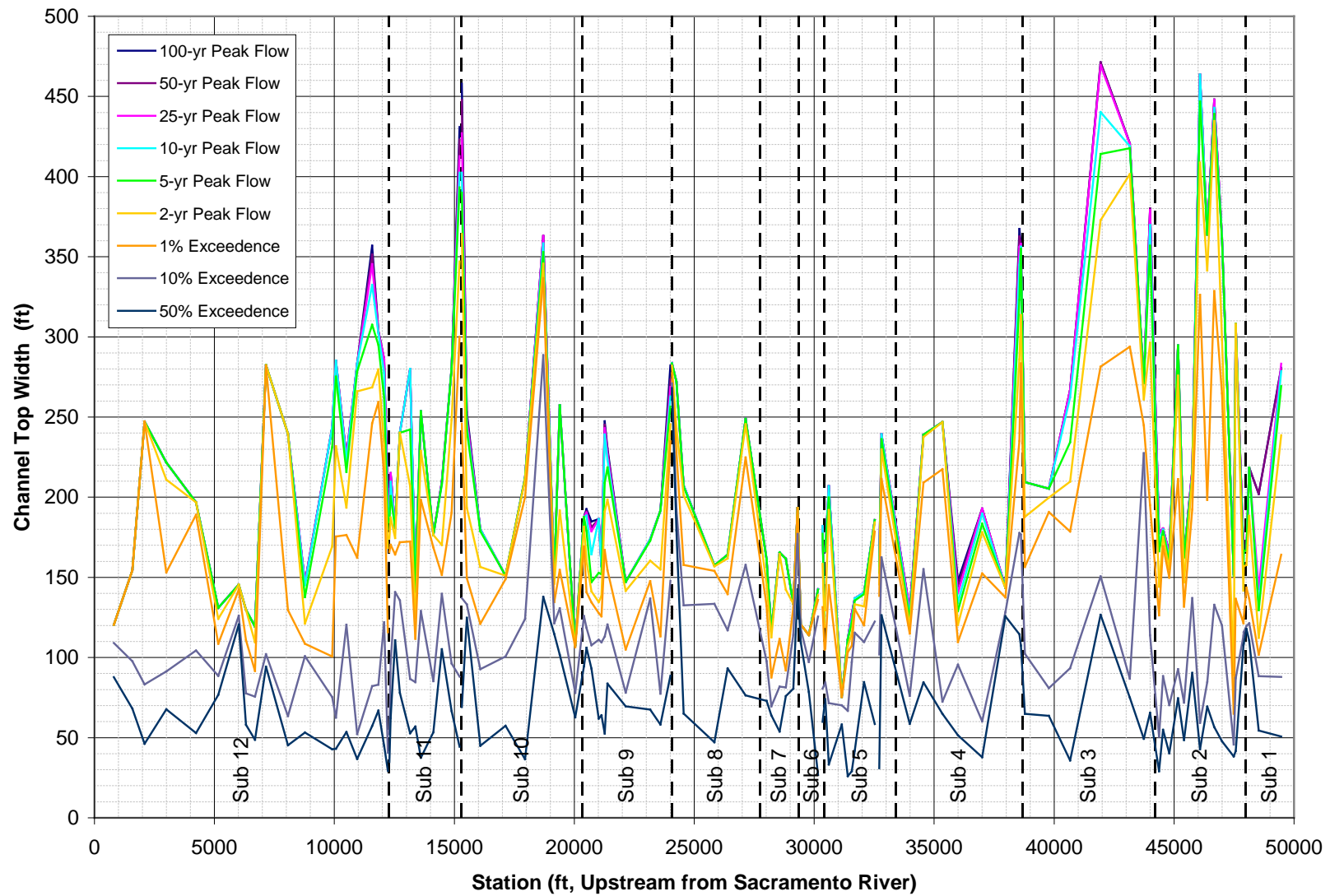


Figure 18. Computed channel top width profiles under existing conditions.

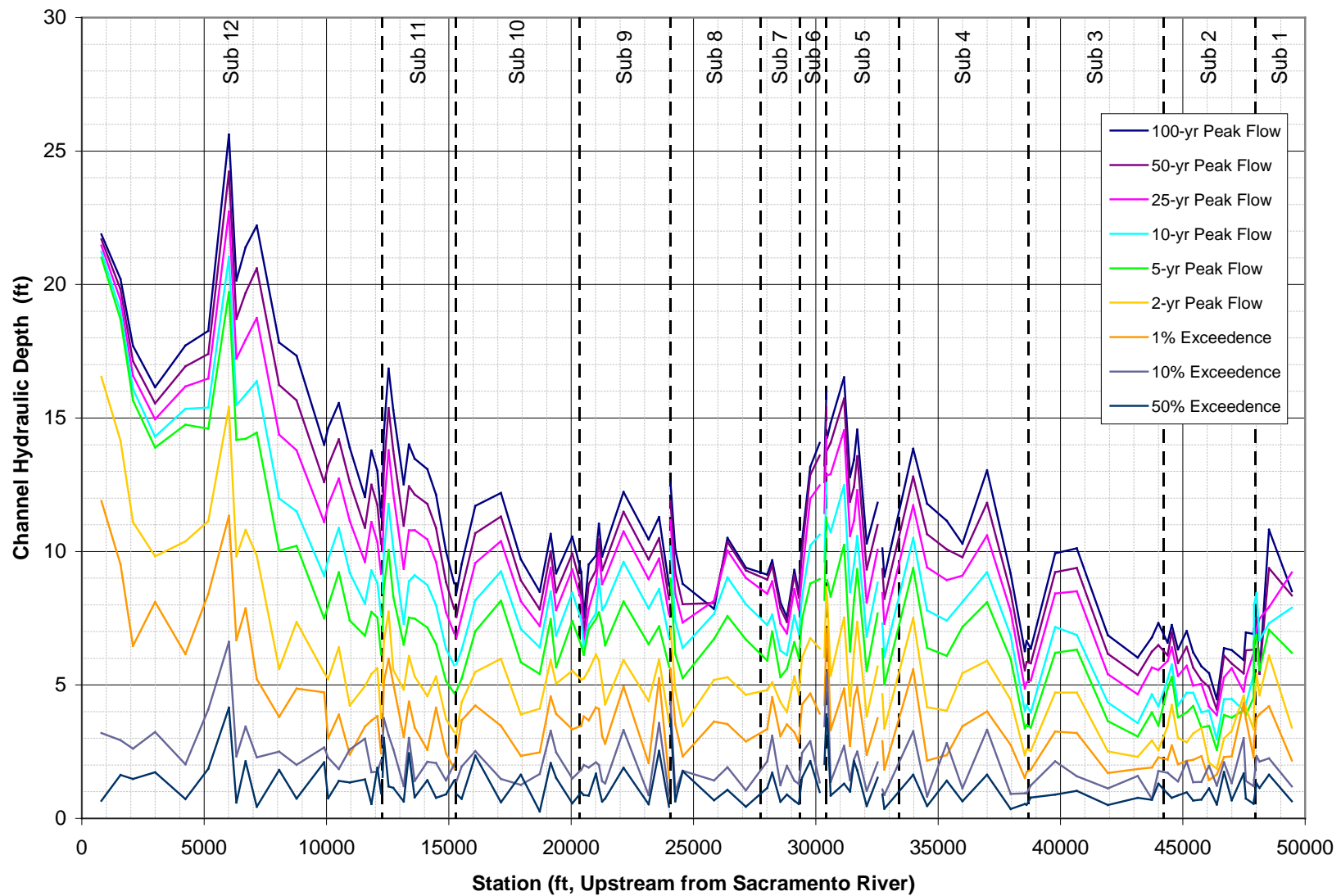


Figure 19. Computed channel hydraulic depth profiles under existing conditions.

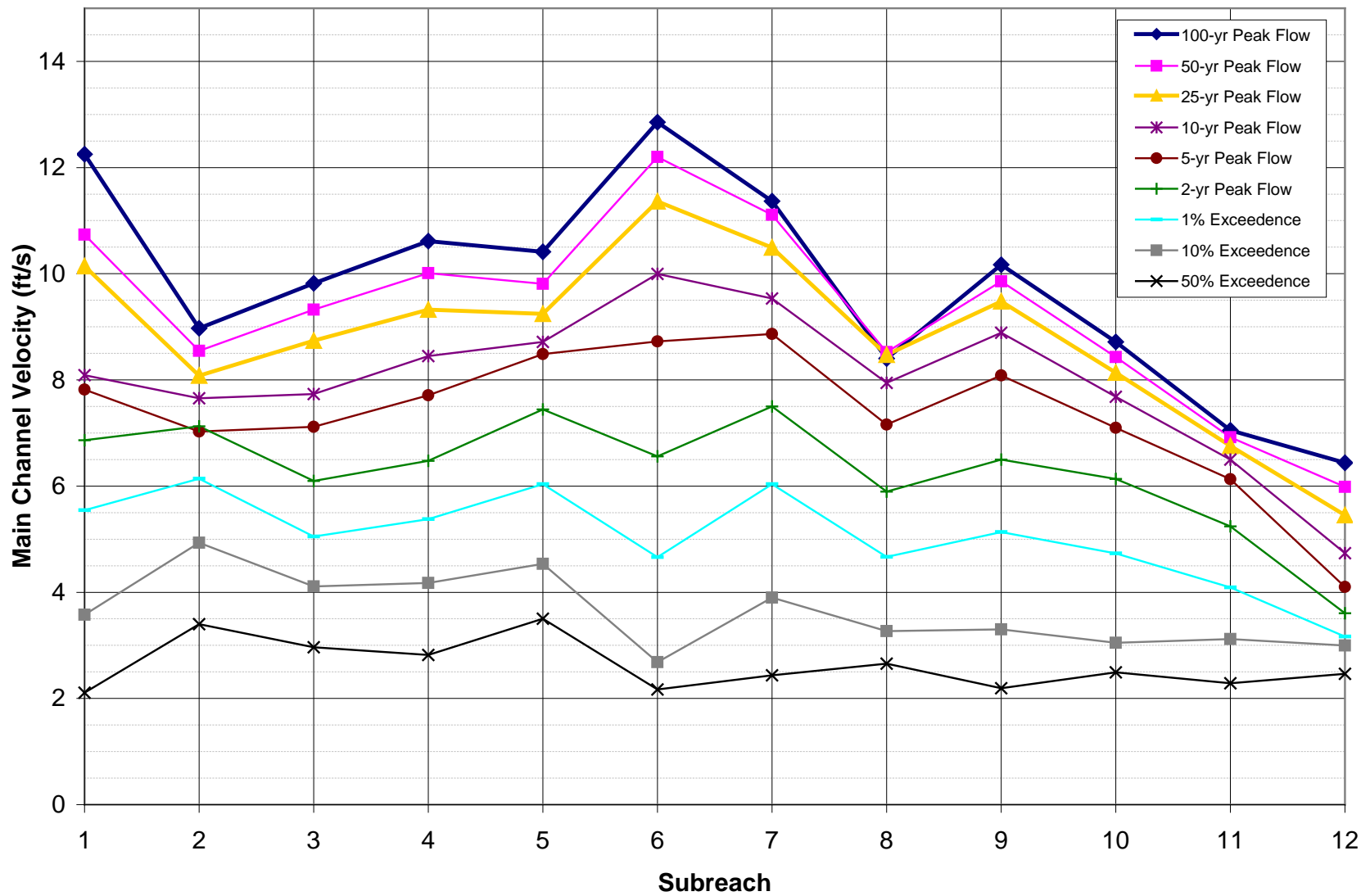


Figure 20. Subreach-averaged main channel velocity under existing conditions.

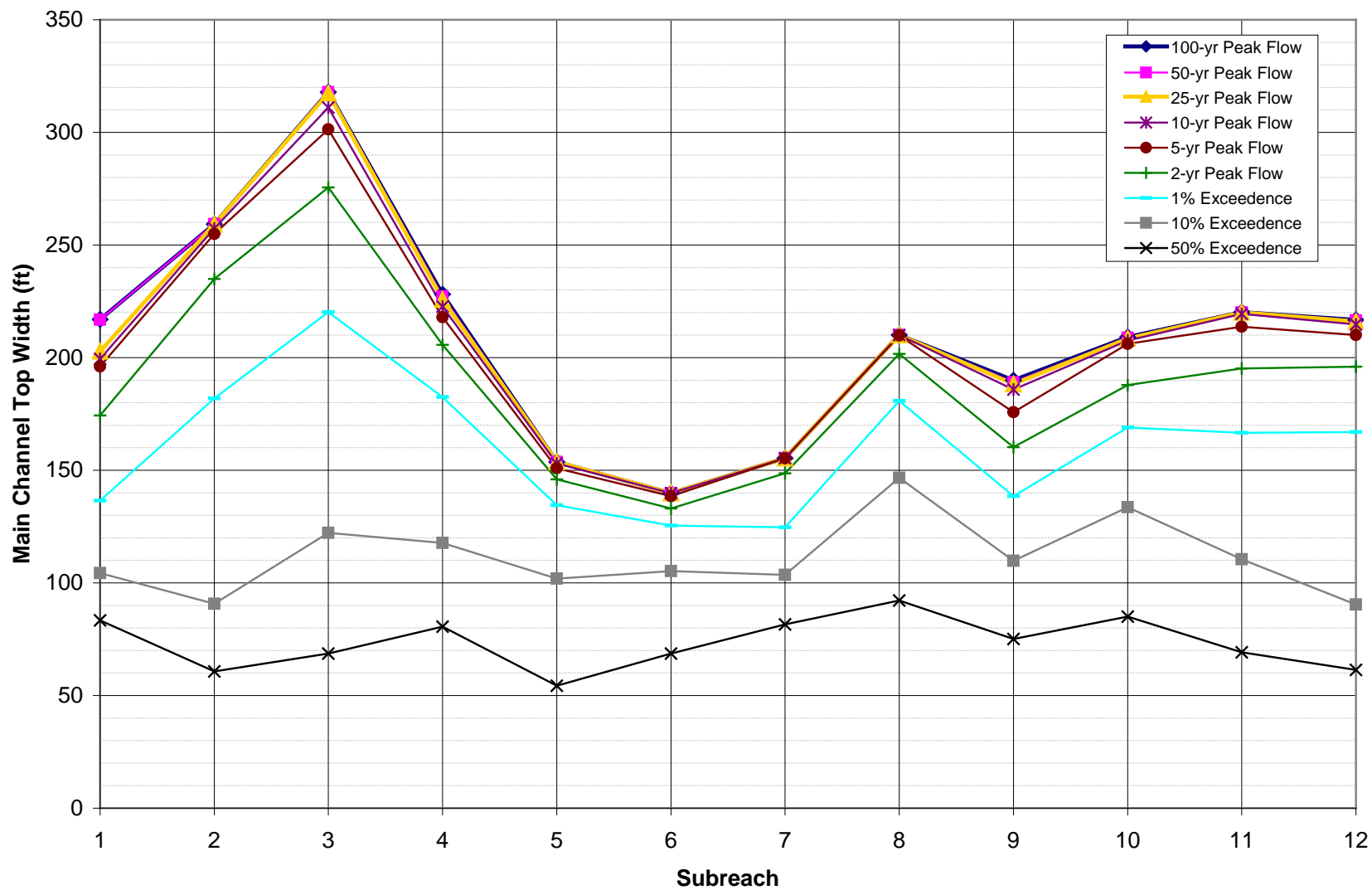


Figure 21. Subreach-averaged main channel top width under existing conditions.

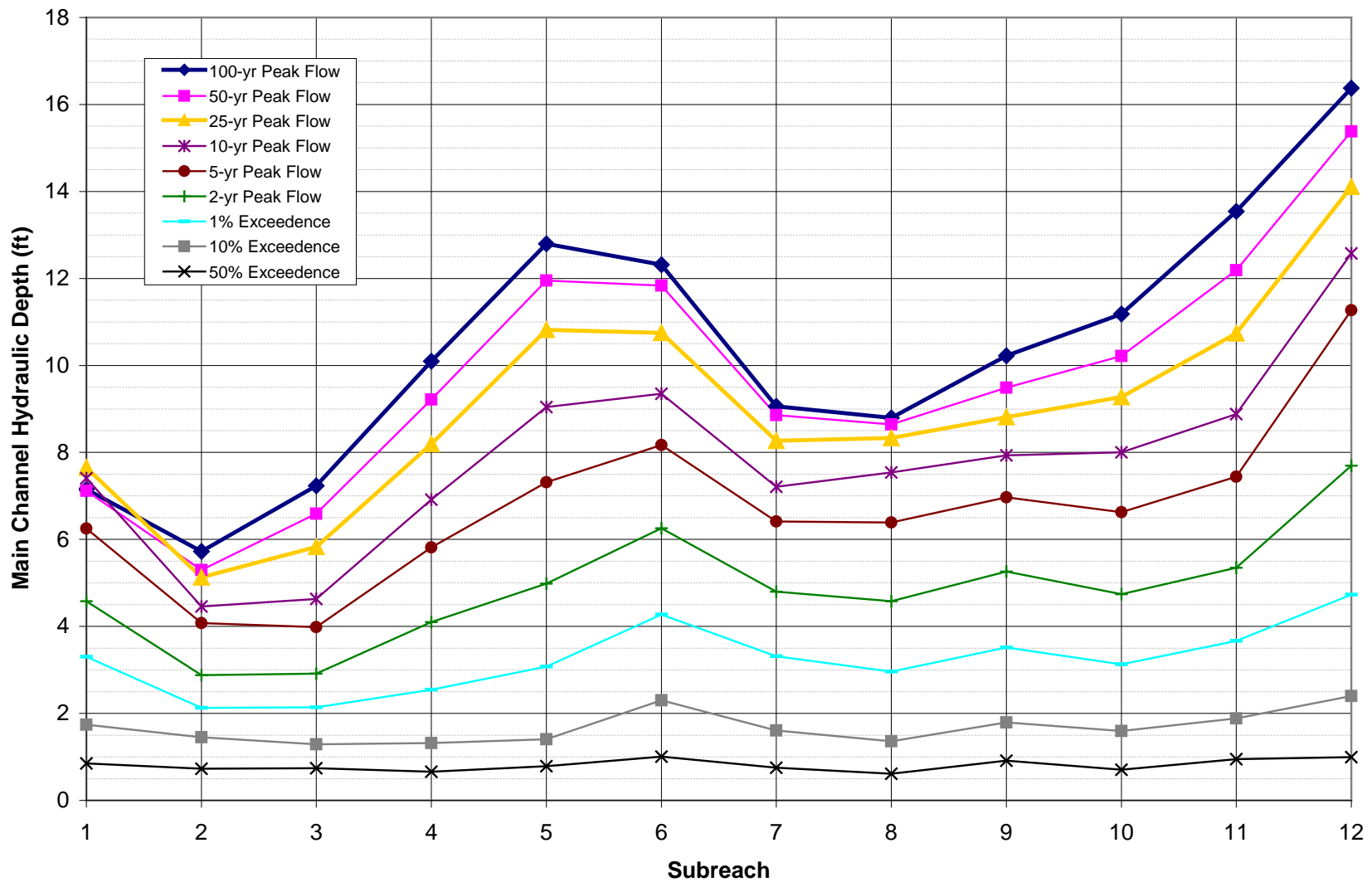


Figure 22. Subreach-averaged main channel hydraulic depth under existing conditions.

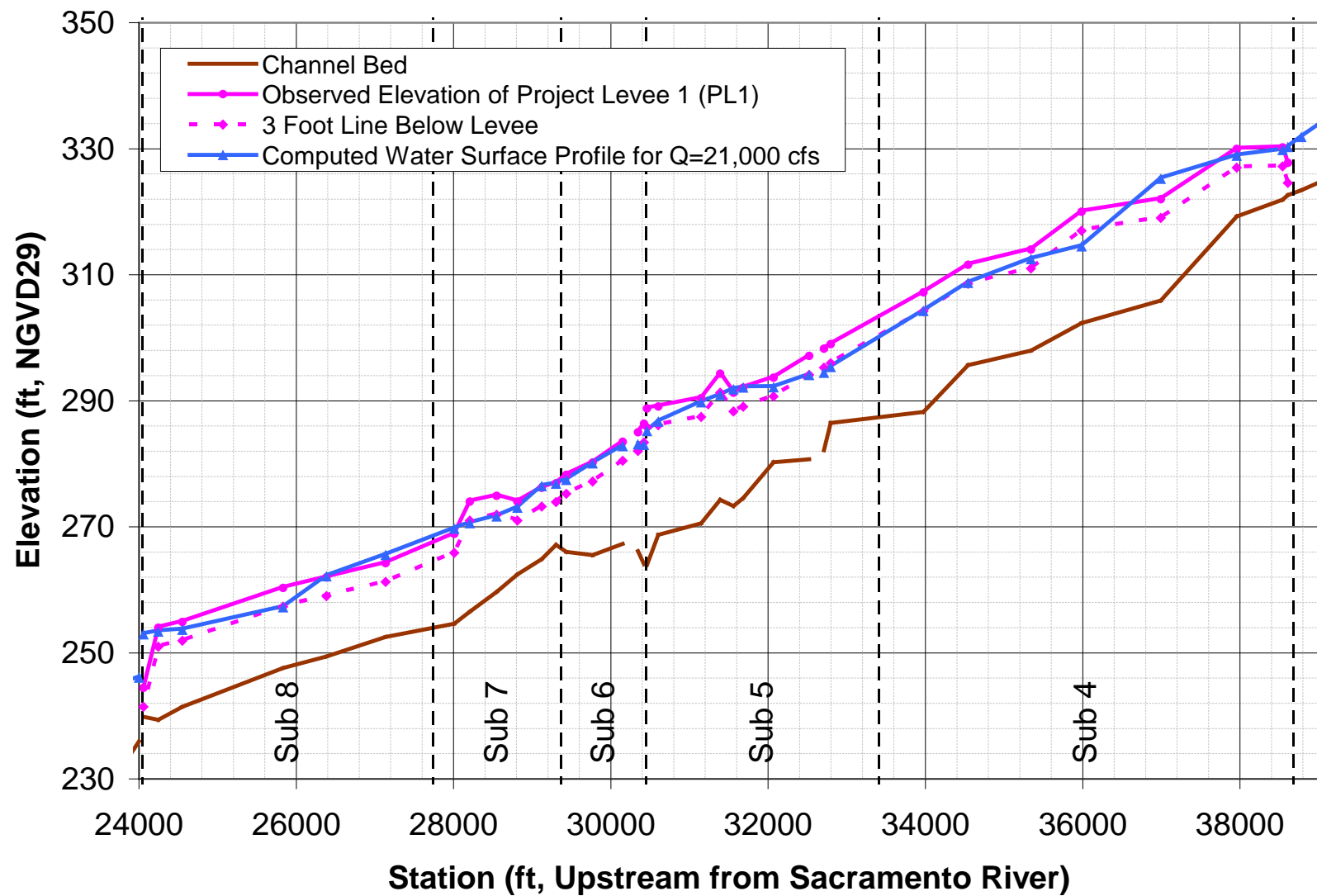


Figure 23. Computed water-surface profile for $Q = 21,000$ cfs, Project Levee 1 (PL1) elevation and 3-foot freeboard line.

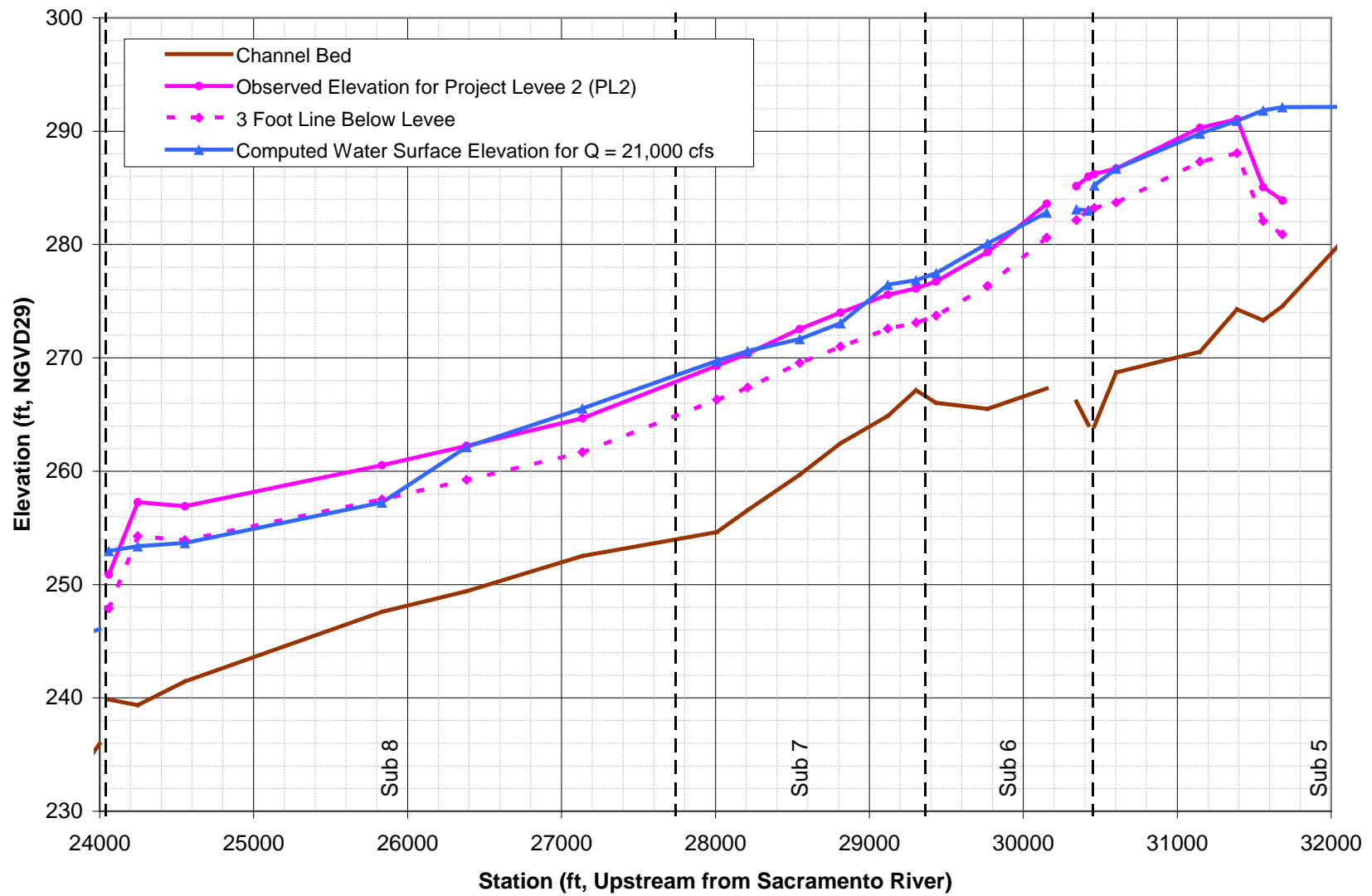


Figure 24. Computed water-surface profile for $Q = 21,000$ cfs, Project Levee 2 (PL2) elevation and 3-foot freeboard line.

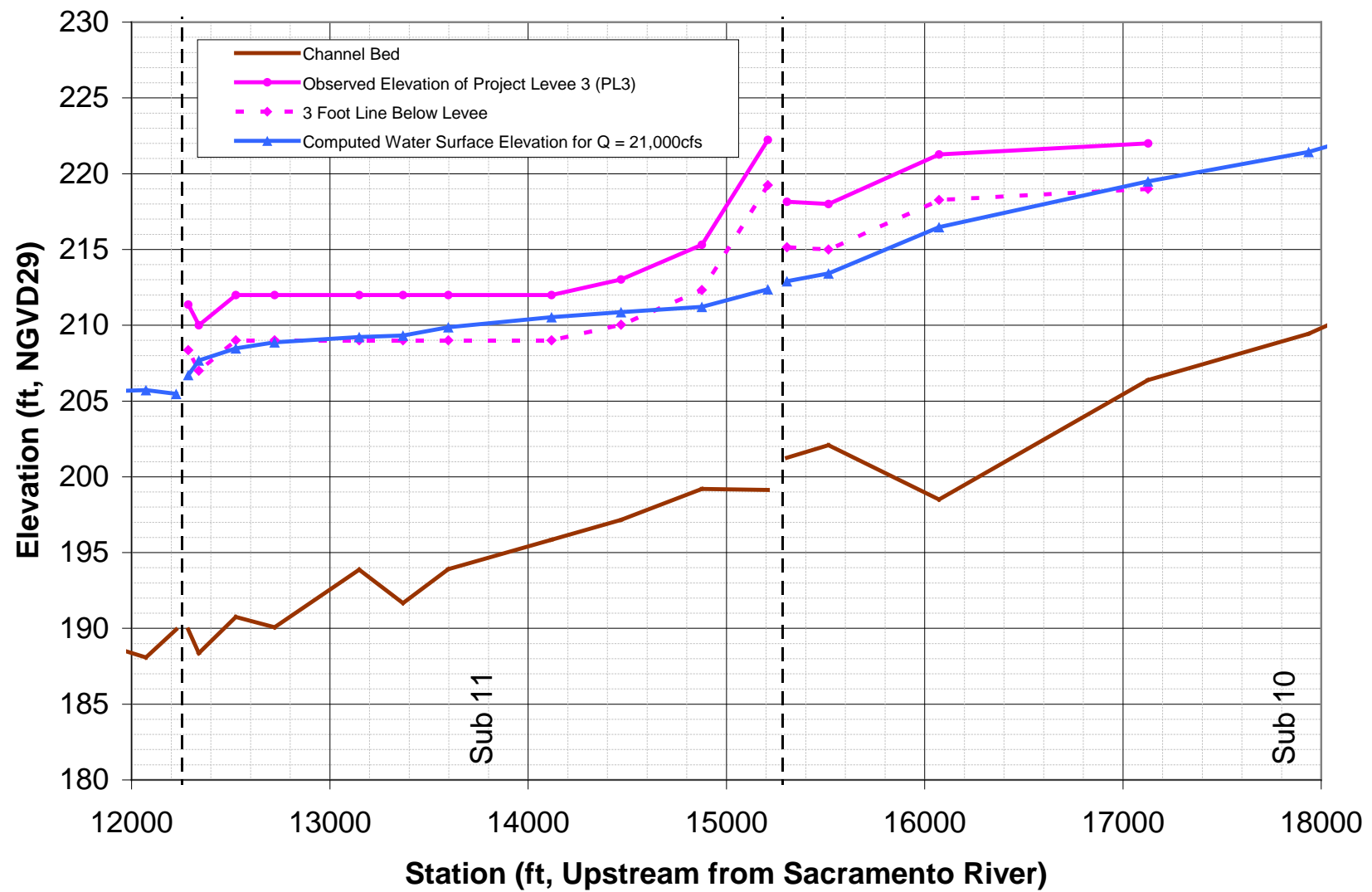


Figure 25. Computed water-surface profile for $Q = 21,000$ cfs, Project Levee 3 (PL3) elevation and 3-foot freeboard line.

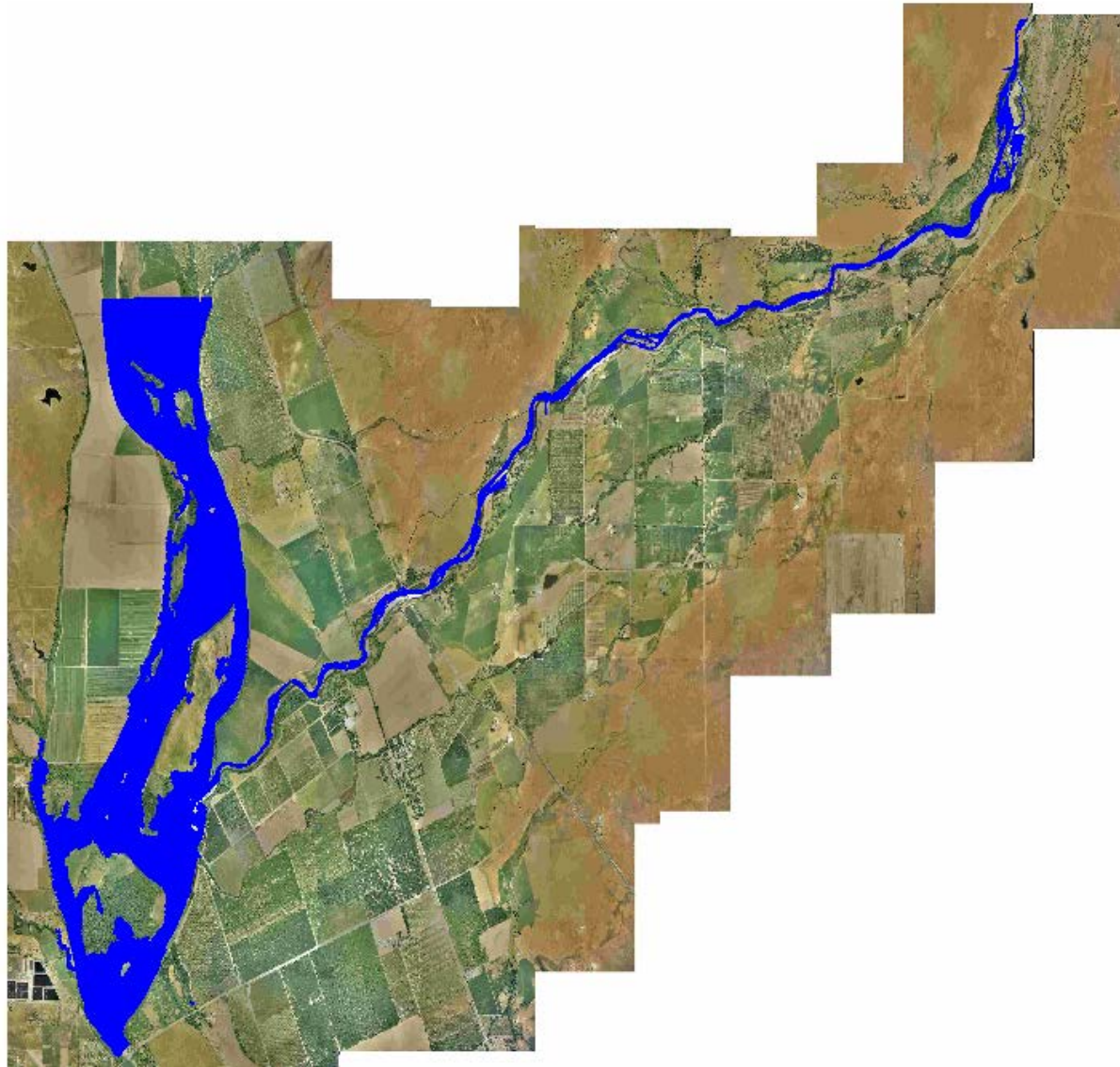


Figure 26. Computed 2-D water-surface from UnTRIM model for Sacramento River flow of 19,285 cfs and Deer Creek flow of 1,210 cfs.

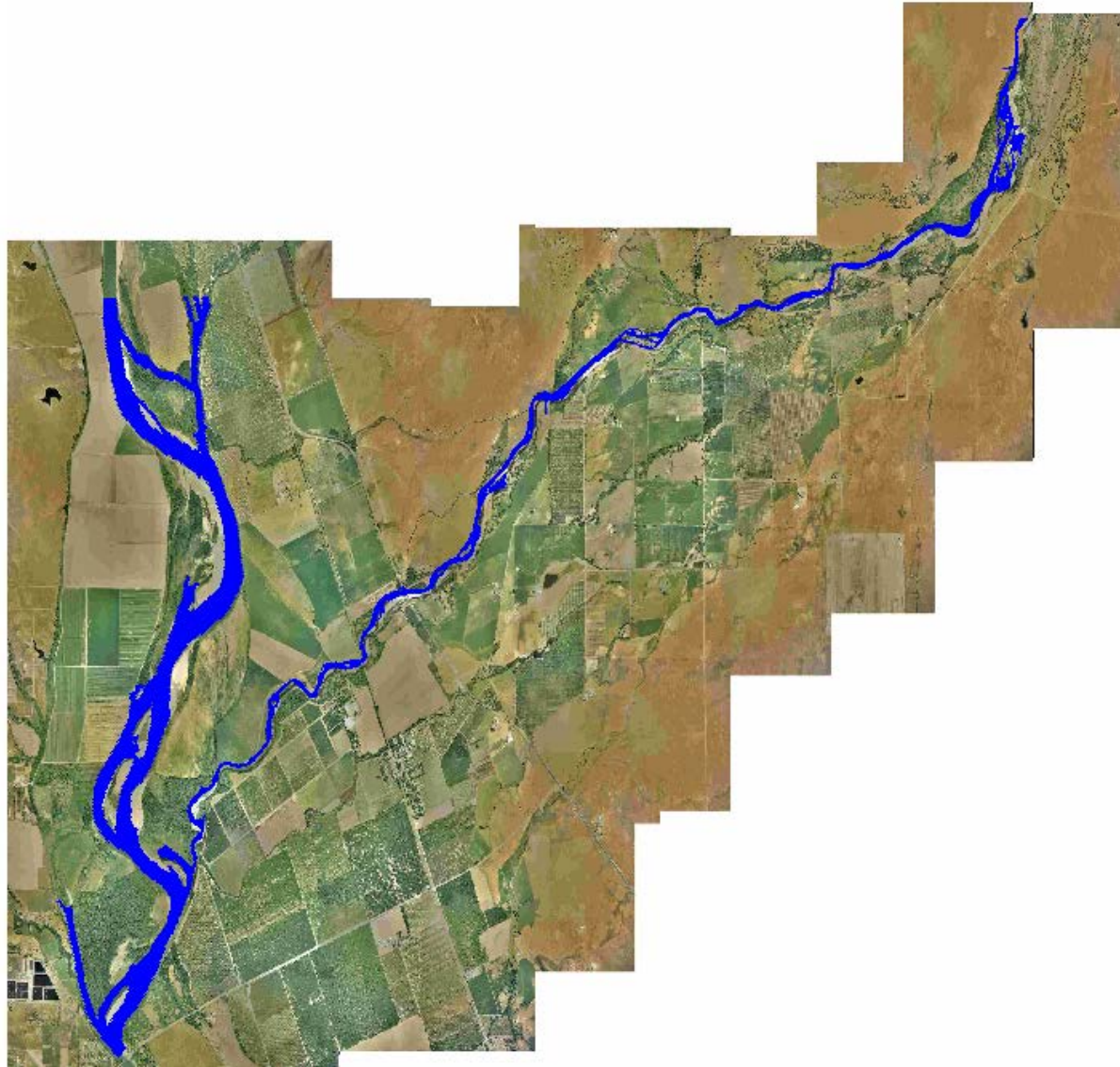


Figure 27. Computed 2-D water-surface from UnTRIM model for Sacramento River flow of 13,570 cfs and Deer Creek flow of 2,120 cfs.

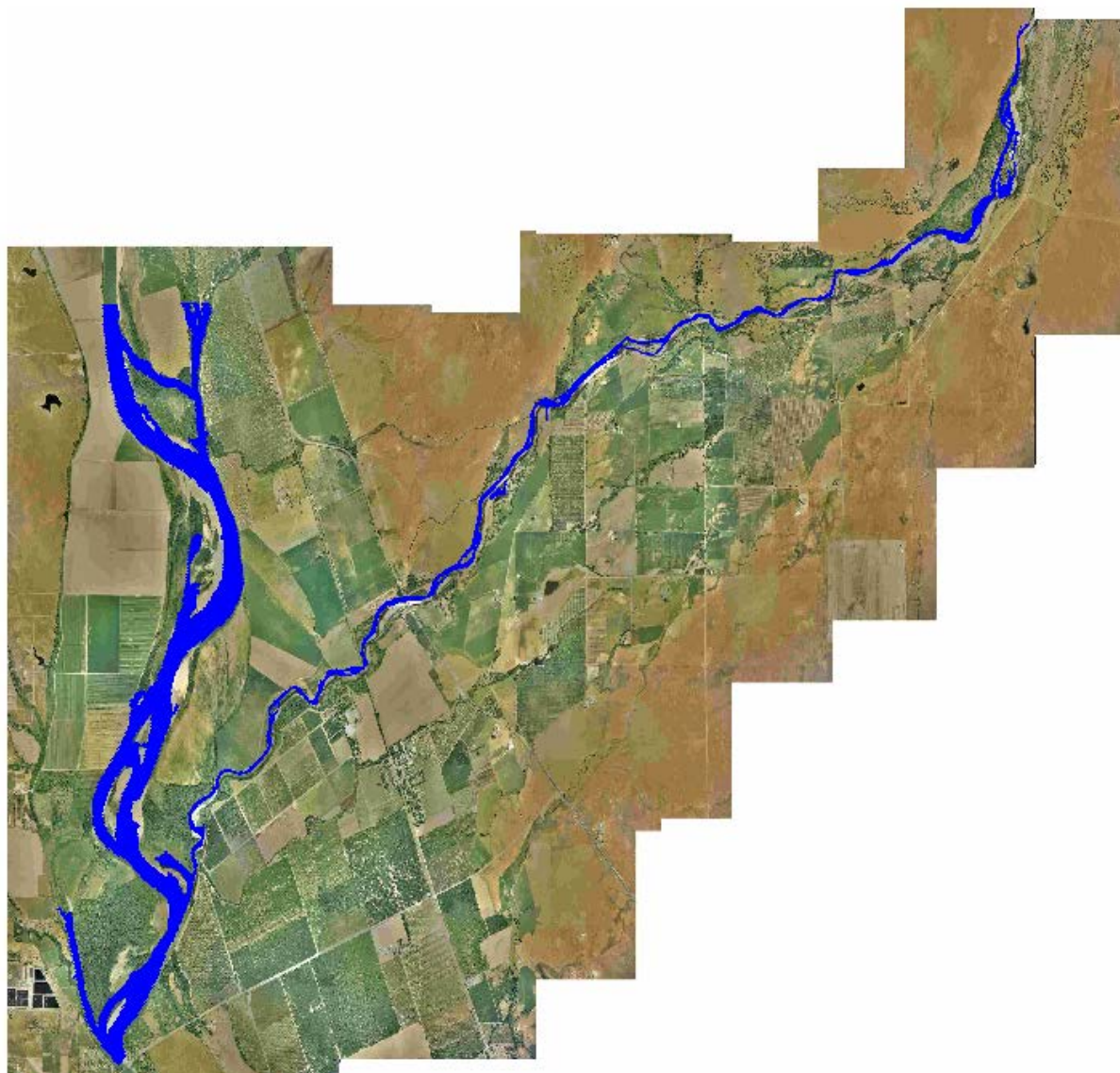


Figure 28. Computed 2-D water-surface from UnTRIM model for Sacramento River flow of 67,405 cfs and Deer Creek flow of 2,940 cfs.

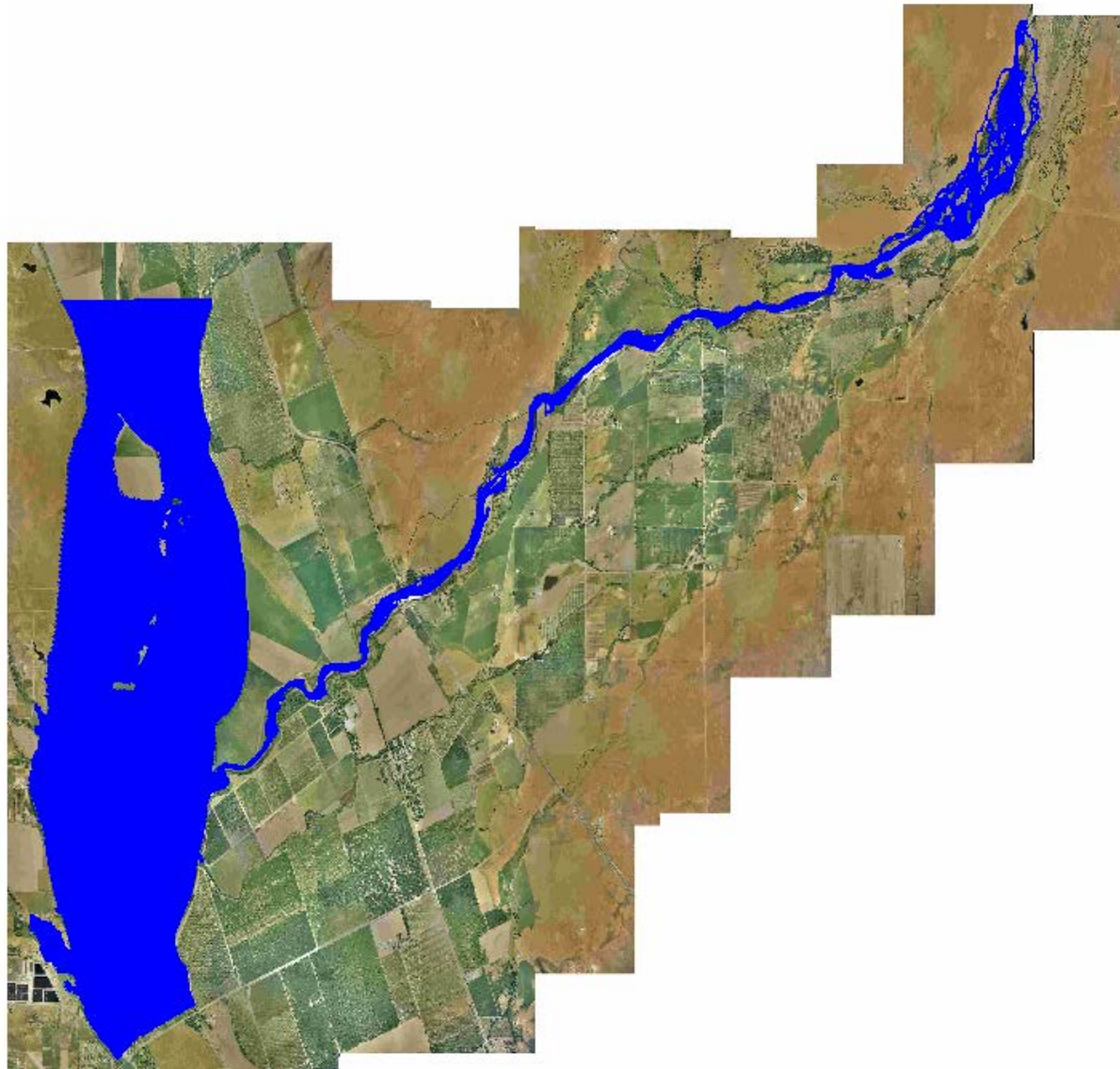


Figure 29. Computed 2-D water-surface from UnTRIM model for Sacramento River flow of 137,000 cfs and Deer Creek flow of 10,300 cfs.