TETRA TECH

MEMORANDUM

TO: Jim Well, Ducks Unlimited
FROM: Dai Thomas, PE, Bob Mussetter, PhD, PE, Mike Harvey, PhD, PG
SUBJECT: **FINAL-Evaluation of the Proposed M&T Pump Intake on the West Bank of the Sacramento River. Project No. US-CA-62-5**

DATE: October 14, 2013

1 INTRODUCTION

As part of an effort to reduce the risk of mortality to native anadromous salmonids, including special-status species within the Sacramento River Basin, the M&T Chico Ranch/Llano Seco Rancho fish screen and pumping facility was redesigned, upgraded, and relocated during 1997 from Big Chico Creek to the Sacramento River. Since its construction, local geomorphic changes, including erosion and lateral migration of the west bank of the Sacramento River and related sediment deposition at the mouth of Big Chico Creek and in the vicinity of the fish-screened intakes have posed a threat to the normal operation and fish protection function of the facility (**Figure 1**).

An up-river gravel bar adjacent to the Bidwell-Sacramento River State Park is growing in size and migrating toward the fish-screened diversion, putting pressure on the right river bank and causing sedimentation problems at the diversion intake. As a result of continued sediment deposition and increased lateral migration, the intake screens are progressively becoming inundated by encroaching sediment, which could cause a reduction in sweeping velocities across the screens (parallel to screen) that would render the screens out of compliance with the National Oceanographic and Atmospheric Administration's National Marine Fisheries Service (NMFS) and the California Department of Fish and Wildlife (CDFW) fish-screen criteria. Periodic maintenance has been required to reduce the size of the gravel bar and prevent interference with the facility. In 2001, 200,000 tons of material was excavated from the gravel bar as a short-term solution to limit sedimentation impacts. An additional approximately 100,000 tons of material was removed in 2007, and 1,500 feet of short-term, rock toe and brush bank protection was installed on the west side of the river adjacent to the U.S. Fish and Wildlife Service's (USFWS) Sacramento River National Wildlife Refuge Capay Unit to prevent further channel migration upstream from the pump inlets (Figure 1).

An Expert Panel composed of multi-disciplinary experts in fluvial geomorphology, sediment transport and hydraulic modeling was formed by CALFED in 2001 to guide investigations, and to ultimately provide recommendations for both short- and long-term solutions to the identified problems. A Steering Committee comprised of stakeholders, CALFED representatives, and engineers with expertise in civil engineering, fish screening and pumping plant technology was also formed to support the investigations.

The Steering Committee and Expert Panel have investigated a wide range of solutions, including continued dredging, construction of a series of spur dikes along the west bank of the river (Steering Committee, 2006) and relocation of the pump intake farther downstream along the east (left) bank (Figure 1).

An additional alternative that involved construction of a T-Screen intake about 500 feet west of the existing intake was considered and rejected early in the evaluation process. The proposed T-screen intake required 8 feet of depth at the minimum pumping discharge of 5,000 cfs (Note: the 8-foot depth was set specifically for the M&T fish screens and may not be appropriate for other studies or other fish screen designs). This alternative was rejected for a variety of reasons that included uncertainty about the longevity of the intake due to continued lateral migration of the river and the ability to maintain the facilities. In December, 2012, a meeting was held that included the project stakeholders, steering committee members and Technical Advisory group (TAG). During the meeting, Mr. Kelly Moroney (USFWS), a member of the TAG, requested that the alternative to re-locate the pumps to the west bank be re-reconsidered because some of the significant issues that led to its initial rejection have been resolved. In particular, the interim toe protection (aka, the rock toe) that has been installed along the west bank adjacent to the USFWS Capay Unit, if formalized into a permanent structure, would eliminate continued westward migration of the river.

Following the request by the USFWS, Ducks Unlimited (DU) directed Tetra Tech to evaluate the viability of this alternative from a river-process perspective. This memo presents the results of that evaluation, including the following specific issues:

- Is there a location for a new intake along the existing interim toe protection that will provide adequate flow depths and appropriate sweeping velocities for the fish screens and limit sedimentation issues at the intakes over the range of flows over which the pump facility must operate that could be as low as 5,000 cfs?

- If so, will the interim toe protection, in its current condition, prevent additional westward migration of the river that could jeopardize the proposed intake over the projected 40-year life of the facility?

1.1 Scope of Work

Tetra Tech performed the following tasks to meet the objectives of this investigation:

1. Previous hydrographic surveys, 2-dimensional (2-D) sediment transport analyses and physical modeling analyses were re-evaluated with a particular focus on the bed dynamics in the vicinity of the interim toe protection to develop a more in-depth understanding of how the area changes with discharge and whether areas with sufficient depth and size persist to accommodate the new intake.

2. The interim rock toe inspections were re-evaluated to determine if the toe protection is adequate to provide bank protection over the next 40 years.

3. The meander modeling analyses conducted by Dr. Eric Larsen (Larsen 2006, 2008) were re-evaluated to determine the potential for the river to migrate and form a channel to the west of the toe rock.

4. The mesh geometry of the existing 2011 Baseline 2-D model (Tetra Tech, 2011) was updated with the 2012 hydrographic survey data and the mesh was refined in the vicinity of the rock toe. The updated 2012 baseline model was validated to measured water-surface elevations collected during the 2012 survey. The model was run at a series of steady-state

discharges from 5,000 to 90,000 cfs, and the model output was used to assess the following specific issues:

  a. velocity patterns and flow depths along the rock toe, and in particular, whether an area of sufficient size exists with depths greater than 8 feet at 5,000 cfs to accommodate the pump intake and fish screens,
  b. determine if the required depths and velocities in this area persist over the duration of the hydrographs, and
  c. evaluate the velocity magnitudes and patterns in this area over the range of flows to assess whether they meet fish screen criteria.

5. The 2012 Baseline model was run in mobile boundary mode using the previously calibrated inflowing sediment load rating curve over the 2008 (low-flow), 2010 (medium-flow), and 2011 (high-flow) hydrographs (Tetra Tech, 2012a). The low, medium and high-flow hydrographs were previously developed by Tetra Tech (2012a) to evaluate the sediment transport patterns in the reach; the hydrograph characteristics are summarized in the hydrology section of this Technical Memo. The model output was used to evaluate aggradation/degradation patterns in the vicinity of the rock toe.

6. The 2012 Baseline model was modified to represent realignment of the west bank by smoothing the bankline through the apex of the existing bend (i.e., the relatively low-radius area that is sometimes referred to as the "belly" of the bank). The realignment was accomplished by filling in a portion of the channel along the west bank, displacing the bankline to the east. The bank was filled to the height of the interim toe protection. This model, referred to as the "Bank Fill" model, was run at steady-state discharges of 5,000 and 90,000 cfs. The model was also run in the mobile-boundary mode over the low, medium and high-flow hydrographs to assess hydraulic conditions and bed dynamics along the realigned revetment.

7. The 2012 Baseline model was modified to represent the geometry of the 1996 mid-channel bar geometry to evaluate the impacts of discontinuing dredging of the primary gravel bar. For purposes of the analysis, the geometry of the bar, as measured during the 1996 survey, was used. This model, referred to as the "1996 Bar" conditions model, was run at steady-state discharges of 5,000 and 90,000 cfs, and in mobile-boundary mode over the low, medium and high-flow hydrographs.

8. The 1996 Bar and Bank Fill conditions models were combined to evaluate the long-term impacts of allowing the gravel bar to build back to 1996 conditions on the re-aligned bank. This geometry, referred to as the "Bank Fill + 1996 Bar" conditions model, was run at steady state discharges of 5,000 and 90,000 cfs, and in mobile-boundary mode over the low, medium and high-flow hydrographs.

2 HYDROLOGY

Hydrologic input to the model consists of both steady flows and unsteady time-series flows developed using recorded discharges at the Sacramento River near Hamilton City gage (California Data Exchange Center Station HMC). The analysis of the discharge regime in the study reach that was performed by Tetra Tech (2011b) was updated for this study by adding the

WY2012 data to the previously analyzed flow records at the Sacramento River near Hamilton City gage (USGS Gage No. 11383800), which is located at about RM 200, approximately seven miles upstream from the M&T Pump station. This included updating the flood-frequency curve for the post-Shasta Dam (1946-2012) period at the Hamilton City gage with the WY2012 peak flow of 44,080 cfs that occurred on March 21, 2012 (**Figure 2, Table 1**).

Table 1. Peak discharges and associated recurrence intervals derived from the flood-frequency curve (Figure 2) at the Hamilton City gage.

Peak Discharge (cfs)	Return Period (years)
59,100	1.2
70,900	1.5
87,600	2
126,900	5
145,800	10
152,300	20
157,800	50
237,800[a]	50
275,900[a]	100

[a] USACE (2008)

As noted in previous analyses (MEI, 2005; Tetra Tech, 2011b), the flood-frequency curve was developed using the Weibull plotting positions because flow regulation by Shasta Dam causes the curve to deviate significantly from the log-Pearson Type III (LPIII) frequency distribution that is typically used for flood-frequency analyses. The curve in Figure 2 indicates that the 1.5- and 2-year recurrence interval peak discharges are about 70,900 and 87,600 cfs, respectively. (The previous analysis indicated peak discharges for these recurrence intervals (RI) of 70,900 and 90,000 cfs, respectively.) The bankfull discharge in the reach is about 90,000 cfs and has a recurrence interval of 2.1 years. The most recent significant flows occurred in December 2005 (Qpeak = 134,600 cfs, R.I.=6 years) and March 2011 (Qpeak = 102,500 cfs, R.I.=3.1 years) (**Figure 3**). Based on the Hamilton City J-Levee analysis (USACE, 2008), the 50- and 100-year peak discharges are 237,800 cfs and 275,900 cfs, respectively.

The mean daily flow-duration curve for the Hamilton City gage was also updated with the WY2012 data. Flow records are available from the USGS for the period from WY1946 through WY1980 and from the CDEC for the period from WY1995 through WY2012 (**Figure 4**). The Hamilton City gage was not operating from 1981 to 1994, and therefore, there are no flow data during this period. The resulting curve indicates that the median flow (flow that is equaled or exceeded 50 percent of the time) at the gage is about 8,530 cfs, and the 10- and 90-percent exceedence flows are 22,200 and 5,460 cfs, respectively. The minimum pumping discharge of 5,000 cfs is equaled or exceeded approximately 95-percent of the time (Figure 3).

Three peak flow hydrographs were previously selected to represent the high, medium and low peak flow events in the models (Tetra Tech, 2012a; **Figure 5**).

The representative high peak flow hydrograph was developed using the March, 2011 event that had a reported peak discharge of 102,530 cfs. This hydrograph had an initial discharge of 11,270 cfs, increased to the peak flow of 102,530 cfs after about 14 days, and then receded to 12,850 cfs after 37 days (Figure 5). The total flow volume of the hydrograph was approximately 2.9 million ac-ft.

The medium peak flow hydrograph was developed using measured flows in January and February 2010. This hydrograph had two peaks that occurred approximately five days apart; the first peak of 76,710 cfs occurred on January 21, 2010 and the second peak of 73,700 cfs occurred on January 26, 2010. The discharge at the start of the hydrograph was 11,600 cfs, increasing to the first peak flow of 76,710 cfs after 7.6 days, decreasing to 13,780 cfs then increasing to the second peak 73,700 cfs after 13 days, and then receding to 9,778 cfs after 20 days. This hydrograph had a total flow volume of approximately 758,000 ac-ft.

The low peak flow hydrograph was developed using measured flows in January and February, 2008. This hydrograph had a peak flow of 66,186 cfs that occurred on January 26, 2008. The hydrograph had an initial discharge of 11,669 cfs , increased to the peak of 66,186 cfs after 20 hours and then receded to 11,130 cfs after approximately 20 days. The total volume of this hydrograph was approximately 287,000 ac-ft.

3 REVIEW OF PREVIOUS STUDIES

Previous studies conducted for the M&T Pumping Plant project, which include hydrographic surveys and numerical and physical modeling-based analyses, were re-evaluated with a particular focus on the bed dynamics in the vicinity of the interim toe protection to develop a more in-depth understanding of how the area changes with discharge and whether areas with sufficient depth and size persist to accommodate the new intake. In addition, the previous interim toe-rock surveys were re-evaluated to assess whether the toe protection is sufficient to provide bank protection over the next 40 years.

The previously developed station line that represents the distance along the approximate centroid of the flow was used to facilitate interpretation of the model results. The downstream end of this station line (Sta 0+00) is located at the downstream boundary of the USACE Butte Basin 2-D model (**Table 2**). The up- and downstream ends of the rock toe are located at Sta 1125+08, and Sta 1111+03 respectively, and the up- and downstream limits of the 2-D model are located at Sta 1183+00 and Sta 1028+00, respectively (**Figure 6**). The existing M&T pumping plant intake is located at Sta 1101+18 on the left (east) side of the river.

3.1 Hydrographic Surveys

Several hydrographic and topographic surveys have been conducted in the reach of the Sacramento River between River Mile (RM) 192 and RM 193.5 to monitor geomorphic changes in the reach, including bed aggradation, bank erosion and lateral migration of the river.

Table 2.	Stationing of points of interest along the project reach.
Station	Description
1028+00	Downstream end of 2-D model
1068+85	Alternative Pump Site 2 (3,500-foot site)
1084+20	Alternative Pump Site 1 (2,200-foot site)
1087+38	Relocated City of Chico Outfall
1101+18	M&T Pumps
1109+51	Downstream end of bank-attached bar
1111+03	Downstream end of rock toe
1114+53	Cross-Section 1
1116+34	Cross-Section 2
1118+88	Cross-Section 3
1121+99	Cross-Section 4
1125+08	Upstream end of rock toe
1130+01	Upstream end of bank-attached bar
1147+86	River Road
1183+00	Upstream end of 2-D model

The initial survey was conducted in December 2005. A peak flow of 134,600 cfs at the Hamilton City gage occurred in January 2006 that caused both lateral erosion of the west bank and significant vertical changes in the bed along the reach. To quantify these changes, the reach was re-surveyed in May 2006. The data from this survey has been used as the baseline condition for much of the modeling that has been conducted, to date. Additional surveys were conducted in January 2010, June 2011 and June 2012. The hydrographic surveys were typically conducted by surveying transects across the channel perpendicular to the main flow alignment at approximately 150-foot intervals along the survey reach. In addition, longitudinal profiles were surveyed to provide additional resolution in the vicinity of the fish screens.

The horizontal datum for all of the surveys was referenced to the State Plane Coordinate System, North American Datum of 1983 (NAD83) (California, Zone 2) and the vertical datum is the North American Vertical Datum of 1988 (NAVD88).

To evaluate the channel conditions along the rock toe and brush bank protection that was installed along west bank in 2007, comparisons of the minimum bed elevations adjacent to the rock toe and cross-section geometry comparisons of the 2006, 2010, 2011 and 2012 survey data were conducted.

Minimum bed elevations along the right bank in the vicinity of the toe protection were selected from the raw survey data to facilitate the analysis (**Figure 6**). Along the length of toe rock, the minimum bed elevation and thalweg (i.e., minium bed elevation across the entire river are the same; however, up- and downstream of the toe-rock, the thalweg is located along the left side of the channel).

The data indicate that minimum channel elevations moved toward the right bank between 2006 and 2010, most likely because the rock toe prevented further westward migration of the river. The largest changes occurred near Sta 1119+00, where the location of the minimum bed elevation moved up to 130 feet closer to the right bank. The minimum channel elevations remained in about the same place between 2010 and 2012.

Based on modeled water-surface elevations (discussed in more detail in Section 4.1), at least 8 feet of flow depth has consistently been present at two specific locations along the west bank since 2006 (Cross section XS1 and XS4; **Figure 7**). Based on the profiles shown in Figure 7, the length of the channel bed with depth greater than 8 feet is approximately 260 feet at XS1 and 550 feet at XS4.

On the basis of the surveyed cross section profiles, the largest changes in the bed geometry at XS1 occur between Sta 200 and Sta 400 due to dredging of the mid-channel bar in 2007 (**Figure 8a**). Up to one foot of aggradation occurred in the deepest part of the channel along the right (west) side between 2006 and 2010. The 2011 and 2012 cross sections are very similar, and most of the differences between the profiles are probably due to transposition of the survey data onto the cross-section lines rather than actual changes in the bed elevation. The peak flow between the 2011 and 2012 surveys was only approximately 44,000 cfs. The channel thalweg was located in a trough near the right bank during all three surveys. The location of the trough moved towards the right bank between 2006 and 2011 as the right bank eroded slightly and the left side of the trough filled with sediment. The trough was approximately 80 feet wide during all of these surveys.

At XS2, there was slight aggradation near the right bank and the minimum channel elevation moved closer to the right bank between 2006 and 2010 (**Figure 8b**). The bar dedging occurred between Sta 350 and Sta 560, and there was slight aggradation in this area between 2010 and 2012. The minimum channel elevation lowered 1.3 feet between 2010 and 2011, and increased by 2.8 feet between 2011 and 2012. Since the 2010, 2011 and 2012 data were collected very close to the cross-section line, the profiles should accurately represent the actual bed profile at this location.

At XS3, the location of the minimum channel elevation shifted towards the right bank between 2006 and 2010, but the actual elevation remained essentially the same from 2006 to 2012 (**Figure 8c**). The 2011 data appear higher than the 2010 and 2012 surveys, but the 2011 survey line was shifted a short distance off the actual cross section line; thus, this difference is likely due to transposition of the survey data onto the cross-section lines, and does not represent actual changes in the cross section. Between 2006 and 2010, the channel lowered between Sta 240 and Sta 560 due to the bar dredging, and approximately one foot of aggradation occurred between Sta 560 and Sta 800.

At XS4, the channel bed lowered by approximately 5 feet between Sta 240 and Sta 550 due to the dredging (**Figure 8d**). The cross section is not shown for 2010 because survey data were not collected in close proximity to the cross-section line. From 2006 to 2012, the channel aggraded by approximately 2 feet between Sta 550 and Sta 650. Near the right bank, the minimum channel elevation (thalweg) and the width of the trough (approximately 70 feet) remained relatively constant from 2006 to 2012.

Comparison of bathymetric surfaces of the overall survey reach from the 2006 and 2010 data indicates that more than 4 feet of aggradation occurred along the right bank between XS1 and XS2 (**Figure 9**). Although not reflected at the scale of the figure, the actual amount of aggradation in this area was between 8 feet and 10 feet. The comparison also indicates that 1 foot to 2 feet of degradation occurred between XS2 and XS3, and 2 feet to more than 4 feet of aggradation occurred between XS3 and XS4. The location of the 2007 gravel removal is clearly visible (>-4 feet) along the left (east) bank of the river adjacent to Bidwell State Park upstream from the pump intake.

A similar comparison of surfaces created from the 2010 and 2011 survey data indicate up to 5 feet of aggradation from the edge of the rock toe to approximately 60 feet into the channel (**Figure 10**). Between XS2 and XS3, the channel bed degraded slightly, as also shown in the cross-section comparisons. Very little change in the bed elevation occurred at XS4. The 2011 to 2012 data indicate a small amount of degradation (0.2 feet to 1 foot) along the right bank between XS1 and XS4. A small amount of aggradation (0.2 feet to 1-foot) occurred on the left side of the channel between XS1 and XS4, including the bar dredging area.

In summary, evaluation of the 2006 to 2012 survey data indicates that XS1 and XS4 have consistently had flow depths greater than the design depth of 8 feet at flows as low as 5,000 cfs. The length of the channel bed with depth meeting the 8-foot criteria is approximately 260 feet in the vicinity of XS1 and 550 feet in the vicinity XS4. At both XS1 and XS4, the thalweg is located within a 70-foot to 80-foot wide trough adjacent to the right bank.

In general, the repeat surveys between 2006 and 2012 show no clear changes in the minimum bed elevation along the right bank. The largest peak flow event (recorded at the Hamilton City gage) between 2006 and 2012 surveys, was 102,500 cfs, which has a 3.1 year return interval.

3.1.1 Fish Screen Options

The existing M&T pumping station was designed by MWH with a capacity of 150 cfs and fitted with four cylindrical tee-screens, each 15 feet long and 54 inches (4.5 feet) in diameter, covered with stainless steel wedge-wire screen material. The screens were designed to comply with criteria established by the California Department of Fish and Game (February 1993) and the National Marine Fisheries Service (1995) (**Table 3**). The *approach velocity is the water velocity vector component perpendicular to the screen face* and the velocity is *measured approximately 3 inches in front the screen surface* (National Marine Fisheries Service, 1997). *The sweeping velocity is the water velocity vector component parallel to the adjacent screen face* (National Marine Fisheries Service, 1997).

Table 3. Design velocity criteria for the existing M&T pumps.

Approach Velocity	CDFW < 0.33 fps
	NMFS < 0.40 fps
Sweeping Velocity	CDFW – "at least two times the allowable approach velocity"
	NMFS – "greater than the approach velocity"

The following discussion is based on personal communications with Dennis Dorratcague, P.E. and Neil Schild, P.E. from MWH in May, 2013. Under the 2012 channel conditions along the right bank, it would be possible to construct a fish screen at XS1 and XS4, and there are three possible designs: (1) in-bank vertical screens (2) t-screens (the same as the existing M&T fish screens) and (3) cone fish screens. Sedimentation around the fish screens is the limiting factor for all the designs; the sedimentation reduces the pumping efficiency, increases the approach velocities, and depending on the design, prevents the screen cleaning system from working. Some fish screen designs have the capability to remove sand-sized material from around the fish screens, however, gravel-sized material is very difficult to manage and remove. MWH indicates that sediment deposition of 1-2 feet would likely render the screens inoperable.

An in-bank vertical screen design would require pumps and an underwater pipe to pump water into the sump at the existing M&T site. The pump would be set at an elevation higher than the 100-year water-surface elevation, which is at ~140 feet based on previous hydraulic modeling (Tetra Tech, 2011b). The channel thalweg is at an elevation of ~105 feet; therefore, the height of the intake structure would be approximately 35 feet. During the 100-year event, there would be about 8 feet of flow depth in the right overbank, and the pump structure would project above the flow. To gain access during high flows without impeding flows in the right overbank, a bridge structure would have to be built across the right bank flood plain.

The T-screen design would be similar to the existing M&T fish screens, which are 4.5 foot diameter cylindrical screens. These typically require a minimum of 8 feet of depth and are designed with 3 feet of freeboard at the minimum flow level. In addition, there needs to be sufficient depth to construct a concrete pad for the fish screen. Using these parameters and the representative flow depth of 9.5 feet, there is approximately 1.5 feet (9.5-4.5-3=1.5 feet) of depth remaining for the concrete pad and potential sediment deposition.

Cone screens are a relatively new design and are most similar to the T-screen design. The cone screens can be designed with the scrubbers on the inside or outside. If the scrubbers are on the inside, the screens can still function with limited sediment deposition; however, it is difficult to remove leaves attached on the outside of the screens. The cone screen would have to be built on pilings with room under the screen structure to allow for sediment deposition.

In summary, MWH indicates that it would be possible to construct a fish screen along the right bank that would meet the design velocity criteria, but sediment deposition remains the critical factor. Given the flow depth is approximately 9.5 feet at the minimum design flow of 5,000 cfs at the two proposed locations and the minimum required flow depth for a 4.5 foot diameter T-screen intake is 8 feet, there is a maximum of 1.5 feet of depth between the bed of the channel and the bottom of the fish screen for the concrete pad and potential sediment deposition.

3.2 Two-Dimensional Sediment Transport Modeling

A two-dimensional (2-D) hydraulic model was previously developed to evaluate the hydraulic and sediment-transport conditions, including the aggradation/degradation patterns, in the overall project reach (Tetra Tech, 2012b). The analysis was conducted using the developmental, mobile-boundary version of the SRH-2D computer program developed by the Bureau of Reclamation (BOR). The original model, referred to herein as the 2010 Baseline model, was used to evaluate erosion and deposition characteristics in the reach, and specifically the potential for future downstream migration of the bank-attached bar and the aggradation/degradation patterns in the vicinity of the existing pump intake.

Representative bed- and surface-material gradations were applied to the 2010 Baseline model and an inflow sediment-rating curve was developed based on the sediment transport characteristics at locations between River Road and the upstream end of the bank-attached bar, which was considered a reasonably uniform section of the river that experienced slight degradation between the 2010 and 2011 surveys. Representative bed material size gradations were used and Parker's (1990) equation was used in the model to predict sediment transport rates. The sediment-transport model was validated by simulating the 2011 peak flow hydrograph (Q_{peak}=102,538 cfs) and comparing the predicted bed geometry at the end of the simulation with the 2011 bed geometry. The 2-D model predicted the magnitude and patterns of aggradation and degradation reasonably well when compared to the measured changes that occurred between 2010 and 2011 in the reach between River Road and the City of Chico outfall (**Figure 12**[1]).

The model output from the simulation of the high peak flow hydrograph (WY2011) indicated deposition of approximately 0.5 feet near the right bank from Sta 1124+00 to XS3 (Sta 1118+88). No change occurred along the right bank in the portion of the reach from XS3 to the downstream end of the rock toe, including the previously identified deep area at XS1. In general, the model predicted that the channel is aggradational from the left side across to about the position of the station line shown in Figure 12, and from Sta 1124+00 (near the upstream end of the bank attached bar) to the downstream limits of the bank attached bar (Sta 1109+51), including the area near the downstream end of the rock toe (Sta 1111+03).

The model outputs from the 2008 low-flow and 2010 medium-flow hydrographs indicated very little change in bed elevation along the reach.

3.3 Physical Modeling

Colorado State University (CSU) conducted two physical model studies (CSU, 2008 and 2011). The first study was conducted primarily to determine if the proposed spur dikes (Figure 1) would create hydrodynamic conditions that would permit sustainable long-term operation of the existing pumps. The model was constructed at 1:75 Froude scale using the 2006 topography and extended from just upstream of River Road (Sta 1150+00) to approximately 2,000 feet downstream of the M&T pumps, a distance of 7,000 feet. In the 2006 baseline conditions model (i.e., without the dikes), the right bank of the river was constructed using erodible materials to represent conditions prior to the installation of the rock toe and the geometry of the bank-attached bar represented pre-dredging conditions. This model was run at steady-state discharges of 10,000 cfs (50-percent exceedence flow), 90,000 cfs (bankfull flow) and 145,000 cfs (the 10-year peak discharge, and the highest flow that could be simulated in the flume). The channel bed elevations were measured at the start and end of the simulations using LiDAR survey equipment, and the elevation differences were used to evaluate the aggradation/degradation patterns along the reach. Although the model represents conditions prior to construction of the rock toe, the results at the higher flows still provide useful information regarding the aggradation/degradation patterns along the reach.

The 2008 model with 90,000 cfs steady flow aggraded along the toe of the right bank from approximately 200 feet upstream of XS4 (Sta 1121+99) to downstream of the M&T pumps (**Figure 13**). The aggradation (blue color) primarily occurs to the west of the station line. The model degraded by a small amount (green/yellow colors) along the main flow path to the east of the station line, and aggraded over the bank-attached gravel bar.

The model run for 145,000 cfs had aggradation/degradation patterns similar to the 90,000-cfs run, with aggradation occurring along the entire length at the base of the rock toe (**Figure 14**). In

[1] Figures 12 and 13 were created by Tetra Tech based on the elevation changes between the pre- and post-simulation topography provided by CSU; these figures are not shown in the CSU (2008) report.

general, the entire area was slightly more aggradational under the 145,000 cfs run compared to the 90,000-cfs run (**Figure 15**).

The second CSU physical model study was constructed to evaluate the existing pump intake and two alternative pump intake sites located 2,200 and 3,500 feet downstream of the existing intake, respectively (Figure 1). The model was constructed at a 1:100 Froude scale and extended from upstream of River Road (Sta to 1131+00) to Sta 1057+00, approximately 5,000 feet downstream from the M&T pump for a total length of approximately 7,400 feet. Each of the three pump locations were modeled using the following three channel configurations:

1. The baseline conditions model was developed to represent the 2010 topography, including hardening of the west bank to represent the interim toe protection.
2. A gravel stockpile that could potentially be constructed on the floodplain on the west overbank approximately opposite the existing M&T pump intake.
3. A re-aligned bank configuration straightening of the bank spur located approximately 2,100 feet downstream from the M&T pumps intake. The new alignment was also hardened to simulate rock revetment that would be placed on the bank.

The second physical model was run at the same discharges as the first model: 10,000 cfs, 90,000 cfs and 145,000 cfs.

Although there is considerable scatter due to the ripple/dune bedforms in the model, the following aggradation/degradation patterns along the right bank can be seen from a comparison of the topography at the end of the run with the pre-run topography (**Figure 16**):

1. The channel is slightly degradational (0.02 feet to1 foot) at XS4
2. The channel is slightly aggradational at XS2 and XS3.
3. The channel is degradational from XS1 to the downstream end of the rock toe.

In addition, other patterns of aggradation in the main river include:

1. the area over the mid-channel bar, and
2. the area between the downstream end of the rock toe and just downstream of the M&T pump intake.

At the end of the 145,000-cfs model run, the channel is predominantly aggradational along the entire length of the rock toe and across the width of the channel, including the base of the rock toe and the bank-attached bar (**Figure 17**).

In summary, both the 2008 and 2011 physical model studies predict that, under the high-flow conditions of 90,000 and 145,000 cfs, the channel is predominately aggradational along the length of the rock toe.

3.4 Rock Toe Evaluations

The rock-toe/brush revetment that was placed on the west bank of the Sacramento River on the U.S. Fish and Wildlife Service's Capay Unit in October 2007 was intended as a temporary measure to prevent further bank erosion and river migration while a long-term solution to the problems at the existing pump intake was identified. The interim revetment was designed to provide toe protection only to the eroding bank, and there was a general expectation that the upper, nearly-vertical, unprotected portion of the bank would continue to erode until a lower bank angle developed that would be colonized by plants that would stabilize this portion of the bank. Approximately five tons/lineal foot of rock were placed at the base of the bank by excavators working from the top of the bank. In addition, six rock tie-backs were constructed along the rock toe to prevent flanking. The rock was sized using the 2-D model output from the 90,000-cfs run and USACE riprap design criteria. The specified median size of the rock used in the installation was 0.75 feet, the 30^{th} percentile was 0.63 feet and the largest particles were 0.94 feet. The top of the revetment was set at an elevation of about 119 feet, corresponding to the water-surface at a discharge of approximately 15,000 cfs. This discharge is exceeded about 20 percent of the time on an average annual basis, based on the mean daily flow-duration curve (Figure 3). The rock toe has been subjected to annual peak flows of 66,100 cfs, 55,500 cfs, 76,700 cfs, 102,500 cfs and 44,000 cfs since its construction in 2007.

Because the rock toe was designed as an interim and temporary measure, there was an expectation that some maintenance would be required. To evaluate the maintenance requirements, field inspections of the rock toe were conducted in April 2010 (Tetra Tech, 2010) and November 2011 (Tetra Tech, 2012b).

Specific items that could require maintenance and were assessed during the field inspections included the following:

1. Flanking of the -upstream end of the structure,
2. Loss of rock due to local scour at the base of the structure,
3. Loss of woody material incorporated into and placed on the structure,
4. Excessive erosion of the unprotected portion of the bank, and scour along the contact between the rock toe and the bank, and
5. Excessive erosion off the downstream end of the structure.

Following is the summary from the 2012 rock toe inspection report (Tetra Tech, 2012b).

Based on the observations of the interim revetment on April 12, 2010 and November 1, 2011, it is clear that there were no immediate requirements for maintenance of the site following a range of peak flows up to 102,528 cfs. At the highest discharge experienced since construction, the left overbank above the toe rock revetment was overtopped in 2011 but there was no evidence of either accelerated erosion of the upper bank or damage to the revetment itself. Both the upstream and downstream transitions into and from the revetment show no signs of significant erosion. Significant numbers of riparian plants have volunteered onto both the top of the rock revetment and onto the reduced-angle lower bank slope above the contact with the revetment. There does not appear to have been any loss of large woody debris from the structure. Based on the field observations it appears that the toe rock revetment is performing well and continues to maintain the current river alignment.

The rock sizing specifications were checked using output from the 1996 Bar model for 90,000-cfs and USACE riprap design criteria, and the results indicated the original rock size specifications meet the USACE riprap design criteria. Although the interim toe rock was not designed as a long-term structure, it has been subjected to greater than bankfull flows and it is appears to be performing well. It is, therefore, anticipated that, with continual monitoring and maintenance, the rock toe will continue to function well over the 40-year design life of the M&T pumps.

3.5 Meander Migration Modeling

The channel migration analyses performed by Larsen (2006 and 2008) were re-evaluated to assess the potential for the Sacramento River to migrate to the west of the rock toe. Larsen (2006) evaluated the impacts of the proposed spur dikes by modeling meander migration over a 50-year period for several scenarios, including: existing conditions (n1), eight spur dikes in place along the west bank of the river (n2), removed bank restraints at River Road with no dikes in place (n3), eight spur dikes in place on the west bank of the river and existing restraints removed at River Road (n4), and nine spur dikes in place on the west bank of the river with River Road restraint in place (n5). Although the rock toe was not specifically modeled, the 8-dike scenario (n2) approximates the extents of the rock toe. Larsen (2008) conducted meander modeling to evaluate the ecological benefit (channel migration and area reworked) of removing existing revetments at nine locations between RM 179 and RM 222. The sites were located within three general reaches of the river: Woodson Bridge (RM 220-222R, RM 216-217L), Hamilton City (RM 197-198R, RM196L, RM 191-192R, 191L, 196L and 196R) and Ord Ferry (RM 179R) (**Figure 18**). The 2004 river planform was used to represent the baseline conditions in and the model was calibrated to river behavior between 1980 and 2004. The modeled migration was performed from simulated WY2005 to WY2054 (in 5-year increments), which were based on the recorded flows from WY1939 to WY1988 from three different gages on the Sacramento River.

The results from the meander modeling predict that the channel does not migrate to the west of the rock toe either under existing conditions or with the spur dikes/groins in place. The model predicts that with the River Road revetment in place (n2), there is no significant change in channel alignment from the existing condition (n1) up- and downstream of the dike field. The model results (n2, n4) also showed that, regardless of the scenario modeled with the dikes in place, there is a tendency for continued westward migration of the right bank downstream of the dike field.

In summary, meander modeling conducted by Larsen (2006 and 2008) did not predict channel migration to the west of the rock toe with or without existing revetments at River Road or with the spur dikes in place.

4 TWO-DIMENSIONAL HYDRAULIC AND SEDIMENT-TRANSPORT MODELING

Additional modeling was conducted to facilitate a more detailed evaluation of aggradation/degradation conditions in the vicinity of the rock toe. The modeling was conducted using the developmental, mobile-boundary version of the SRH-2D Version 3 beta (BOR, 2010) with version 10.1 of the Aquaveo Surface Water Modeling System (SMS) graphical user interface (Aquaveo, 2010). As discussed above, this version of SRH-2D was previously used to evaluate the sediment-transport conditions and aggradation/degradation patterns in the overall study reach (Tetra Tech, 2012a).

SRH-2D (version 3) computes scour and deposition by simulating the interaction between sediment transport and the hydraulics of the flow. The model simulates vertical changes in bed elevations and changes in the surface bed material gradation. In general, SRH-2-D simulates bed elevation changes by estimating the bed-material transport capacity at each element based on the flow hydraulics and bed-material characteristics, comparing the estimated capacity with the upstream sediment supply, and adjusting the bed elevations to account for the differences between the sediment supply and the transport capacity (i.e., the net addition or loss of sediment to the element). SRH-2D routes the sediment through the reach by size-fraction; thus, model results reflect changes in the bed-material gradation that result from differences between the supply and transport capacity of the individual size fractions. This capability allows the model to simulate sorting and either coarsening or fining of the surface layer in response to differences between the sediment supply and capacity in each element, an important capability for coarse-bedded rivers such as the study reach for this project.

In this study, the "flow" option in SRH-2D was initially used to simulate steady-state flows ranging from 5,000 cfs to 90,000 cfs, and the model output was used to evaluate the depth and velocity patterns along the reach. The sediment-transport parameters, including the inflowing sediment rating-curve and bed-material gradations were then input to the model, and the model was run using the "mobile" option to perform sediment transport simulations over the duration of the low, medium and peak flow hydrographs (refer to Section 2).

4.1 Model Development

4.1.1 Model Mesh

Separate model meshes were developed for each of the following four channel conditions:

1. The 2010 Baseline model (Tetra Tech, 2012a) was updated with the 2012 topography and the mesh geometry was refined in the vicinity of the rock toe. This model is referred to as the 2012 Baseline model and was used to evaluate the response of the reach to the hydrographs under existing conditions for comparison with the other scenarios.

2. The 2012 Baseline model was modified to represent the realignment of the west bank by smoothing the bank line through the apex of the bend (red line in **Figure 19**) and backfilling to match the elevation of the existing rock toe (119 feet). This Bank Fill model was developed to determine if re-aligning the bank improves the hydraulic and sediment transport conditions along the right bank. The upstream limit of the bank re-alignment ties into the southern end of a straight section of bank at approximately Sta 1120+00. The downstream limit of the realignment ties into the downstream end of the existing rock toe located at the boundary with the Shaw Property (Figure 1). The re-aligned bank is approximately 980 feet long and an additional 19,700 yd^3 of material (either riprap or a combination of riprap and finer fill material) would be required for construction.

3. The 2012 Baseline model was modified to incorporate the 1996 mid-channel bar geometry to simulate conditions if the bar is allowed to rebuild. This model is referred to as the 1996 Bar model. It is assumed that if the M&T pumps were relocated to the west bank, no further dredging would occur and the bar would eventually reform with similar geometry to the 1996 conditions.

4. The Bank Fill model was further modified to incorporate the geometry of the 1996 bar. This model was used to evaluate conditions with the re-aligned bank and no future dredging of the mid-channel bar. This geometry is referred to as the Bank Fill + 1996 Bar conditions model.

4.1.2 Model Parameters and Model Validation

The model parameters applied to the 2010 model, which include the Manning's *n*-values, the downstream stage-discharge rating curve, and the calibrated sediment-transport rating curve were applied to the 2012 baseline model. Consistent with the previous studies, the Parker (1990) surface-based bed-load equation was used to calculate the sediment transport loads. A full description of these parameters and the values used in the modeling are detailed in the 2012 Tetra Tech report (Tetra Tech, 2012a).

Using these parameters,the agreement between the computed and measured water-surface elevations during the June 2012 survey when the discharge was approximately 11,345 cfs, is very good (**Figure 20**). The average difference between the predicted and measured values is 0.0 feet, and the maximum difference that occurs in the riffle near XS2 is 0.15 feet (Sta 1115+00).

4.2 Model Results

All four scenarios (2012 Baseline, Bank Fill, 1996 Bar, and Bank Fill + 1996 Bar) were modeled at steady state discharges of 5,000 and 90,000 cfs, and sediment-transport modeling was simulated over the low-, medium- and high-flood hydrographs. In addition, the 2012 baseline model was run at steady state discharges of 10,000, 15,000, 20,000, 30,000, 50,000 and 75,000 cfs to increase the resolution of the stage-discharge rating curves at XS1 and XS4. The model output was used to evaluate the hydraulic conditions along the reach with particular focus at XS1 and XS4, which were previously identified in Section 3.1 as possible pump intake locations.

4.2.1 2012 Baseline Conditions

4.2.1.1 <u>Steady State Modeling Results</u>

At 5,000 cfs, the channel is distinctly deeper along the base of the rock toe as shown by the predominantly yellow colors to the west of the station line (**Figure 21**). This deep area was previously described as a trough in the cross-section comparisons. At XS1 and XS4, the maximum depths are 9.8 and 9.4 feet, respectively. The velocity pattern along the rock toe indicates that the highest velocities of approximately 4.5 fps occur near the up- and downstream limits of the rock toe. At XS1 and XS4, the velocities are 1.8 and 4.3 fps at the deepest parts of the channel (**Figure 22, Table 4)**.

At 90,000 cfs, the depths at XS1 and XS4 are 27.2 and 26.6 feet, respectively (**Figure 23**). The velocity distribution at 90,000 cfs indicates that the high velocity zone occurs to the left (east) of the center of the channel (**Figure 24**). At XS1 through XS4, the highest velocities occur over the left side of the bar; whereas, under low-flow conditions, the highest velocities occurred near the right bank. The model predicts an eddy in the area between the west bank and the red colored bank re-alignment line, including the right end of XS3 and XS4. The existence of the eddy suggests that the area may be depositional. During the 2012 field surveys, the channel bed in the location of the eddy was composed of mostly sand-sized material, confirming that this is a low energy area. At 90,000 cfs, the predicted velocity near the right bank at XS1 is very low (0.1 fps in the upstream direction) and the velocity at XS4 is approximately 5.1 fps in a downstream direction (Table 4).

Table 4. Summary of velocities at 5,000 and 90,000 cfs for the four scenarios at XS1 and XS4.

Condition	Location	5,000 cfs		90,000 cfs	
		Depth (ft)	Velocity (ft)	Depth (ft)	Velocity (ft)
2012 Baseline	XS1	9.8	1.8	27.2	-0.1[a]
	XS4	9.4	4.3	26.6	5.1
Bank Fill	XS1	5.3	4.2	22.5	3.4
	XS4	9.4	3.8	26.6	5.1
1996 Bar	XS1	9.8	2.6	27.2	-0.3[a]
	XS4	9.4	4.8	26.6	5.4
Bank Fill + 1996 Bar	XS1	5.3	5	22.5	3.7
	XS4	9.4	4.3	26.6	5.5

[a] negative sign indicates velocity is in an upstream direction

Stage-discharge curves were developed at XS 1 (Sta 1114+53) and XS4 (Sta 1121+99) for the range of flows from 5,000 to 90,000 cfs (**Figure 25**). The depths are reported at the location identified as the deepest part of the channel during the field surveys (Figure 6). The flow depths over the range of flows were integrated with the flow-duration curve (Figure 3) to develop depth-duration curves at XS1 and XS4 (**Figure 26**). The depth-duration curves indicate that flow depths are greater than 11.3 feet at XS1 and greater than 10.8 feet at XS4 about 50 percent of the time. Similarly, the flow depths at XS1 and XS4 exceed 12.2 and 10.2 feet, respectively, 75 percent of the time, and exceed 10.0 and 9.6 feet, respectively, 90 percent of the time.

4.2.1.2 Sediment Transport Modeling Results of the 2011 Flood.

The 2012 baseline model was run for the 2011 peak flow event ("high" peak flow hydrograph) and the predicted changes in bed elevation at the end of the 2011 flood simulation were evaluated (**Figure 27**). The bank re-alignment and 1996 bar features are shown on the following figures for illustrative purposes. The results of the 2011 flood simulation indicate the following:

1. The model predicts approximately 0.3 feet of aggradation at XS4 and along the right bank just upstream from XS4. At the thalweg of XS4, the channel aggrades by 0.4 feet at 600 hours, and then degrades slightly to 0.3 feet of aggradation at the end of the simulation (**Figure 28**). In general, the reach is predominately depositional across the channel between XS4 and the downstream end of the rock toe.

2. At XS1 (Figure 28), XS2 and XS3, the model predicts no change in bed elevation near the right bank. The west margin of the deposition approximately follows the red bank re-alignment line.
3. The model predicts approximately 3 to 5 feet of aggradation over the mid-channel bar.

The model results from the low and medium flow hydrographs predicted virtually no change in bed elevations along the reach, including in the vicinity of the rock toe.

4.2.2 Bank Fill Conditions

4.2.2.1 Steady State Modeling Results

At 5,000 cfs, the thread of deep flow follows the approximate alignment of the station line (**Figure 29**). In the Bank Fill and 1996 Bar + Bank Fill conditions models, the depth and velocity measurement location for XS1 is father east than for existing conditions, due to the bank realignment. The predicted depth at XS1 and XS4 is 5.3 and 9.4 feet, respectively.

The highest velocities of approximately 5.2 fps occur along the project station line, centered at XS2 (**Figure 30**). In general, the velocities along the right bank are approximately 0.5 fps higher than under baseline conditions (Figure 22) due to the mid-channel bar forcing more flow to the west side of the channel, and hence increasing the velocities. At XS1 and XS4, the velocities are 4.2 and 3.8 fps, respectively (Table 4).

At 90,000 cfs, the velocity distribution along the project reach is similar to 2012 Baseline conditions, including over the bank fill, where an eddy is still predicted to occur (**Figure 31**). The velocities at XS1 and XS4 are 3.4 and 5.4 fps, respectively (Table 4). The depth distribution is very similar to baseline conditions, except over the bank fill where the depths are approximately 13 feet. The predicted depth at XS1 is 22.5 feet.

4.2.2.2 Sediment Transport Modeling Results

The mobile-boundary Bank Fill model predicts approximately 0.3 feet of aggradation along the right bank from upstream of XS4 to downstream from the rock toe (**Figure 32**). Similar to the baseline conditions, the model predicts approximately 3 to 5 feet of aggradation over the mid-channel bar.

4.2.3 1996 Bar Conditions

4.2.3.1 Steady State Modeling Results

At 5,000 cfs, the predicted depth distribution from the 1996 Bar model is very similar to the 2012 baseline conditions (**Figure 33**). The contraction created by the bar does not significantly raise the water-surface elevations or increase the flow depths. Under the 1996 Bar conditions, the velocities adjacent to the right bank increase by approximately 0.5 fps due to the reduction in channel width created by the bar (**Figure 34**). The velocity zone adjacent to the bank that has velocities in the 4 fps to 5 fps range under baseline conditions is no longer present (Figure 22). The velocities at XS1 and XS4 are 2.6 and 4.8 fps, respectively (Table 1). At 90,000 cfs, the velocities increase by approximately 0.3 fps in the vicinity of XS4 and the predicted velocities at

XS1 and XS4 are 0.3 fps in an upstream direction and 5.5 fps in the downstream direction, respectively. The magnitudes and distribution of velocities at 90,000 cfs are very similar to baseline conditions.

4.2.3.2 Sediment Transport Modeling Results

The mobile boundary model results for the Bank Fill conditions show the following bed elevation responses (**Figure 35**)**:**

1. The model predicts about 0.5 feet of degradation along the west bank for approximately 200 feet up- and downstream of XS4. The bed degrades at a steady rate on the falling limb between 400 to 500 hours, then the degradation remains relatively uniform at about 0.5 feet from 500 hours until the end of the simulation (Figure 28).
2. At XS1 and XS2, the model predicts approximately 0.2 feet of aggradation, this is somewhat higher than baseline conditions, and is most likely due to deposition of the material that is eroded from near XS4.
3. The model predicts an increase in degradation along the bank near the downstream end of the rock toe (Sta 1110+00), most likely due to the bar forcing more flow towards the right bank. For comparison, the Baseline model predicted degradation at Sta 1110+00 slightly to the west of the channel centerline; however, under 1996 Bar Conditions, the focus of the deposition moved closer to the right bank. An increase in erosion along the west bank, and subsequent widening of the channel opposite the existing pump intake, could result in further downstream movement of the mid-channel bar. This could lead to burial of the pump intake with sediment.

4.2.4 Bank fill + 1996 Bar Conditions

4.2.4.1 Steady State Modeling Results

At 5,000 cfs, the depth distribution is very similar to Bank Fill conditions (**Figure 36**). The contraction created by the combination of rebuilding of the 1996 gravel bar and the bank fill does not significantly raise the water-surface elevations or increase the flow depths. The flow contraction created by the bank fill and bar increases the velocities significantly along the right bank in the vicinity of XS2 to 6 fps to 7 fps (**Figure 37**). The velocities at XS1 and XS4 are 5.0 fps and 4.3 fps, respectively, compared to 1.8 fps and 4.3 fps, respectively, under the baseline conditions.

At 90,000 cfs, the velocities at XS1 and XS4 are 3.7 fps and 5.7 fps, respectively, compared to -0.1 fps (upstream direction) and 5.5 fps (downstream direction), respectively, under baseline conditions.

4.2.4.2 Sediment Transport Modeling Results

The results from the mobile-boundary Bank Fill + 1996 Bar conditions model with the 2011 peak flow event indicate the following (**Figure 38**):

1. The contraction created by the bank re-alignment and 1996 bar increases the degradation along the west bank. Approximately 0.5 feet of degradation occurs along the west bank from approximately 200 feet upstream from XS4 to just downstream from XS2. The predicted bed change at XS4 is steady degradation from approximately 400 to 500 hours, then the degradation remains relatively uniform at about 0.5 feet from 500 hours until the end of the simulation (Figure 28).
2. Under baseline conditions, there was slight aggradation at XS4.
3. The model predicts little approximately 0.2 feet of aggradation at XS1 which is higher than baseline conditions, and is likely the result of deposition of the material eroded from near XS4.
4. The model predicts up to 3 feet of degradation along the bank near the downstream end of the rock toe (Sta 1110+00), due to the bar forcing more flow towards the right bank. This also occurred for 1996 Bar conditions, and the erosion along the west bank could result in the mid-channel bar moving downstream which could lead to burial of the existing M&T pump intakes with sediment.

5 SUMMARY AND CONCLUSIONS

5.1 Summary

The objectives of this study were to address the following issues:

- Is there a location for a new intake along the existing interim toe protection that will provide adequate flow depths and appropriate sweeping velocities for the fish screens and limit sedimentation issues at the intakes over the range of flows over which the pump facility must operate (discharges as low as 5,000 cfs)?

- If so, will the interim toe protection, in its current condition, prevent additional westward migration of the river that could jeopardize the proposed intake over the projected 40-year life of the facility?

To meet the objectives, a review of previous surveys, 2-D hydraulic and sediment-transport analyses, interim toe-rock surveys, meander migration analyses and physical modeling analyses were re-evaluated. In addition, two-dimensional sediment transport modeling was performed with a particular focus on evaluating the sediment transport patterns in the vicinity of the interim toe protection.

Following is a summary of the study findings:

1. The 5,000 cfs, minimum discharge at which pumping can occur is equaled or exceeded approximately 95 percent of the time based on the mean-daily flow-duration curve.

2. Data from the 2006, 2010, 2011 and 2012 hydrographic surveys indicated two areas that met the minimum design depth of eight feet at 5,000 cfs. These locations occur in the vicinity of XS1 and XS4 where the depths for the 2012 geometry were approximately 9.8 and 9.4 feet, respectively. The length of the channel bed with depth greater than 8 feet is

approximately 260 feet in the vicinity of XS1 and 550 feet in the vicinity of XS4. The minimum channel elevations are located in an approximately 80-foot wide trough adjacent to the right bank at both cross sections. Comparisons of the repeat cross section surveys indicate that XS1 aggraded by approximately 1 foot between 2006 to 2010. At XS4, the cross-section shape and the minimum bed elevations remained relatively constant over the four surveys.

3. MWH indicates that it would be possible to construct a fishscreen along the right bank based on the 2012 channel geometry, and they suggest three possible designs: T-screen, in-bank vertical screen, and cone screen. All three options can be designed to meet the approach and sweeping velocity criteria set by NMFS and CDFW. MWH indicated that sedimentation would be the limiting factor for all three designs.

4. The repeat survey data indicate the following aggradation/degradation patterns along the reach:

    a. From 2006 to 2010, the reach generally aggraded along the right bank.

    b. From 2010 to 2011, there was up to 5 feet of aggradation at XS1 and little or no change at XS4.

    c. From 2011 to 2012, there was very little change in bed elevations along the right bank at either location.

5. The 2006 Physical Model Study that was conducted to assess the viability of the proposed spur dikes predicted aggradation along the entire length of the rock toe at 90,000 and 145,000 cfs.

6. The 2010 Physical Model Study that was conducted to further assess the sedimentation patterns in the vicinity of the existing pump intake and the viability of moving the pump intake to two alternative, downstream locations predicted the following:

    a. At 90,000 cfs, the channel is slightly degradational at XS4, aggradational between XS1 and XS4, and degradational from XS1 to the downstream end of the rock toe. At 90,000 cfs, the general pattern across the river is aggradational between the up- and downstream limits of the bank-attached bar.

    b. At 145,000 cfs, the model predicted aggradation along the entire length of the rock toe.

7. Meander modeling conducted by Larsen (2006, 2008) did not predict channel migration to the west of the rock toe.

8. The 2010 baseline 2-D model simulation for the high peak flow hydrograph predicts approximately 0.4 feet of aggradation at XS4 and no change at XS1, XS2 or XS3. The 2010 baseline model simulation of the low and medium peak flow hydrographs predicted little or no systematic changes along the reach, including along the rock toe.

9. The interim toe rock inspections indicated that *the toe rock revetment is performing well and continues to maintain the current river alignment.* The rock sizing was rechecked using the output from 1996 Bar conditions model and indicated that the rock size meets the current

USACE riprap criteria. Although the interim toe rock was not designed as a long-term structure, it has been subjected to greater than bankfull flows, and it appears to be performing well. It is, therefore, anticipated that, with continual monitoring and maintenance, the rock toe should continue to function well over the 40 year design life of the M&T pumps.

10. 2-D modeling of the 2012 baseline conditions at a steady flow of 5,000 cfs indicate depths at XS1 and XS4 of 9.8 and 9.4 feet, respectively, and velocities of 1.8 and 4.3 fps, respectively.

11. The sediment transport simulation of the 2012 baseline conditions over the 2011 flood predicted the following aggradation/degradation patterns:

    a. Little or no change in bed elevation along the right bank at XS1, XS2 and XS3.

    b. Approximately 3 to 5 feet of aggradation over the mid-channel bar.

12. The 2-D modeling of the Bank Fill conditions at a steady flow of 5,000 cfs predicted the depths at XS1 and XS4 are 5.3 feet and 9.4 ft, respectively. The depth at XS1 should be treated with caution, as the channel bed represents 2012 conditions, and will likely adjust following construction of the bank fill due to local scour along the face of the new bank protection that would be required to insure stability of the fill.

13. The sediment transport simulation of Bank Fill conditions over the 2011 flood predicted the following aggradation/degradation trends:

    a. Slight aggradation along the right bank from upstream of XS4 to downstream from the rock toe.

    b. Similar to the baseline conditions, approximately 3 to 5 feet of aggradation over the mid-channel bar.

14. 2-D modeling of 1996 Bar conditions at a steady flow of 5,000 cfs, predicted the same depths at XS1 and XS4 as the baseline conditions (9.8 feet and 9.4 feet, respectively). The velocity at XS1 increased from 1.8 fps under baseline conditions to 2.6 fps under 1996 Bar conditions due to the contraction created by the bar.

15. The sediment transport simulation of the 1996 Bar conditions over the 2011 flood predicted the following aggradation/degradation patterns:

    a. a small amount of degradation (0.5 feet) along the right bank for approximately 200 feet up- and downstream of XS4, is likely due to the bar forcing more flow towards the right bank. This will likely help maintain a deeper channel adjacent to the right bank over the long term.

    b. a small amount of aggradation (0.2 feet) at XS1 and XS2, somewhat greater than under the 2012 baseline conditions. This aggradation likely results from deposition of material eroded from near XS 4.

c. an increase in degradation along the bank near the downstream end of the rock toe, likely due to the bar forcing more flow towards the right bank. This will likely help maintain a deeper channel adjacent to the right bank just downstream of the toe rock.

d. an increase in erosion along the west bank and subsequent widening of the channel opposite the M&T pumps which could result in the mid-channel bar moving downstream and potential burial of the M&T pumps with sediment.

16. The 2-dimensional modeling of the Bank fill + 1996 Bar conditions at a steady flow 5,000 cfs predicted the same depths at XS1 and XS4 (5.3 and 9.4 feet, respectively) as the Bank Fill conditions. The velocity at XS1 increased from 1.8 fps under baseline conditions to 5.0 fps under the 1996 Bar conditions due to the contraction created by the bar.

17. The sediment transport simulation of the Bank Fill + 1996 Bar conditions over the 2011 flood predicted the following aggradation/degradation patterns:

    a. The contraction created by the bank re-alignment and 1996 bar increases the degradation along the west bank to approximately 0.5 feet from approximately 200 feet upstream from XS4 to just below XS2. Under baseline conditions, there was slight aggradation at XS4.

    b. a small amount of (~0.7 feet) of aggradation at XS1, which is higher than under baseline conditions and likely the result of the deposition of material eroded from near XS4 .

    c. up to 3 feet of degradation along the bank near the downstream end of the rock toe (Sta 1110+00) due to the bar forcing more flow towards the right bank. This observation was also made for 1996 Bar condition. The erosion along the west bank could result in the mid-channel bar moving downstream, potentially burying the existing pump intake.

5.2 Conclusions

Two sites were identified in the 2012 topography (XS1 and XS4) that meet the specified minimum requirement of 8 feet of depth at the minimum discharge for pumping of 5,000 cfs, and there is likely sufficient in-channel area (length and width) to install the fish screens. The flow depth at XS1 is about 9.8 feet and the depth at XS4 is about 9.4 feet, which provides 1.8 feet and 1.4 feet, respectively, of depth for construction of the concrete pad and sediment deposition. The channel behavior between 2006 and 2012, results from the CSU physical modeling studies, and the 2-D mobile boundary modeling collectively indicate a reasonable likelihood of aggradation in at least portions of the deeper part of the channel along the rock toe that currently meet the depth requirements for the intake. Together with the limited available depths for construction and sedimentation, there appears to be significant risk that sufficient flow depths for a new pump intake will not be sustained in this area.

6 REFERENCES

Alden Research Laboratory, 2012. Three-Dimensional Numerical Flow Analysis of the M&T Pumps, Chico, California. Submitted to Tetra Tech Inc.,Fort Collins, CO. July, 32 p.

Aquaveo, 2010. SMS Surface-water Modeling System, Version 10.1, User's Manual.

Bureau of Reclamation, 2010. SRH-2D Version 3.0. Two-dimensional river flow modeling computer program. March.

Colorado State University, 2008. M&T Pump Station Intake Physical Model. Submitted to Mussetter Engineering, Inc., Fort Collins, Colorado, and Ducks Unlimited, Rancho Cordova, California, August, 461 p.

Colorado State University, 2011. M&T Pump Station Intake Second Physical Model Report. Submitted to Tetra Tech, Fort Collins, Colorado, and Ducks Unlimited, Rancho Cordova, California, April, 254 p.

Larsen, E.W., 2006. Future Meander Bend Migration and Floodplain Development Patterns near River Miles 200 to 191 of the Sacramento River, Phase III Report. Prepared for Ducks Unlimited, 25 p.

Larsen, E.W., 2008. Modeling Revetment Removal and Implications for Meander Migration of Selected Bends RM 222 to RM 179 of the Sacramento River, January 10, 2008. Prepared for Ducks Unlimited, 31 p.

Mussetter Engineering, Inc. 2005. Two-dimensional Modeling to Evaluate Potential River Training Works at M&T Pumping Plant, Sacramento River, RM192.5. Prepared for Ducks Unlimited, January, 52 p.

Mussetter Engineering, Inc, 2005. Two-dimensional Modeling to Evaluate Potential River Training Works at M&T Pumping Plant Sacramento River, RM 192.5. Prepared for Ducks Unlimited.

Mussetter Engineering, Inc, 2006. Two-dimensional Modeling to Evaluate Potential River Training Works at M&T Pumping Plant Sacramento River, RM 192.5 (2005 Topography and Bed Material). Prepared for Ducks Unlimited.

Mussetter Engineering, Inc., 2008. Phase II Two-dimensional Modeling to Evaluate Potential River Training Works at M&T Pumping Plant, Sacramento, River, RM 192.5. Submitted to Ducks Unlimited, Rancho Cordova, California, April, 42 p.

National Marine Fisheries Service Southwest Region, 1997. Fish Screen Criteria for Anadromous Salmonids, January, 14pp.

Parker, G., 1990. The "Acronym" series of Pascal programs for computing bed load transport in gravel rivers. University of Minnesota, St. Anthony Falls Hydraulic Laboratory, External Memorandum No. M-220.

Steering Committee, 2006. M&T/Llano Seco Fish Screen Facility, Short-Term/Long-Term Protection Project, Technical Memorandum: Workshop #4 Summary. Prepared for Ducks Unlimited.

Tetra Tech, Inc. 2010. M&T/ Llano Seco Fish Screen Facility Short-Term/Long-Term Protection Project (Project No. US-CA-62-2) Maintenance Inspection and Reports (Subtask 14.1). Inspection Report No.1. Submitted to Ducks Unlimited, Rancho Cordova, California, November, 17 p.

Tetra Tech, Inc. 2011a. June 7-9, 2011 Resurvey of the M&T/Llano Seco Pumping Plant and City of Chico Outfall reach of the Sacramento River. Submitted to Ducks Unlimited, Rancho Cordova, California, June, 36 p.

Tetra Tech, 2011b, Phase III Two-Dimensional Modeling of M&T/Llano Seco Pumping Plant Reach, Sacramento River, RM192.5, California. Submitted to Ducks Unlimited, Rancho Cordova, California, October, 77 p.

Tetra Tech, Inc. 2012a. Two-dimensional Sediment-transport Modeling of the M&T/Llano Seco Pumps Reach. Submitted to Ducks Unlimited, Rancho Cordova, California, April, 42 p.

Tetra Tech, Inc. 2012b. M&T/ Llano Seco Fish Screen Facility Short-Term/Long-Term Protection Project (Project No. US-CA-62-2) Maintenance Inspection and Reports (Subtask 14.1). Inspection Report No.2. Submitted to Ducks Unlimited, Rancho Cordova, California, May, 25 p.

U.S. Army Corps of Engineers, Sacramento District, 2008. Hamilton City Flood Damage Reduction Project, Sacramento River, California. Prepared by Ayres Associates, Fort Collins, Colorado, March.

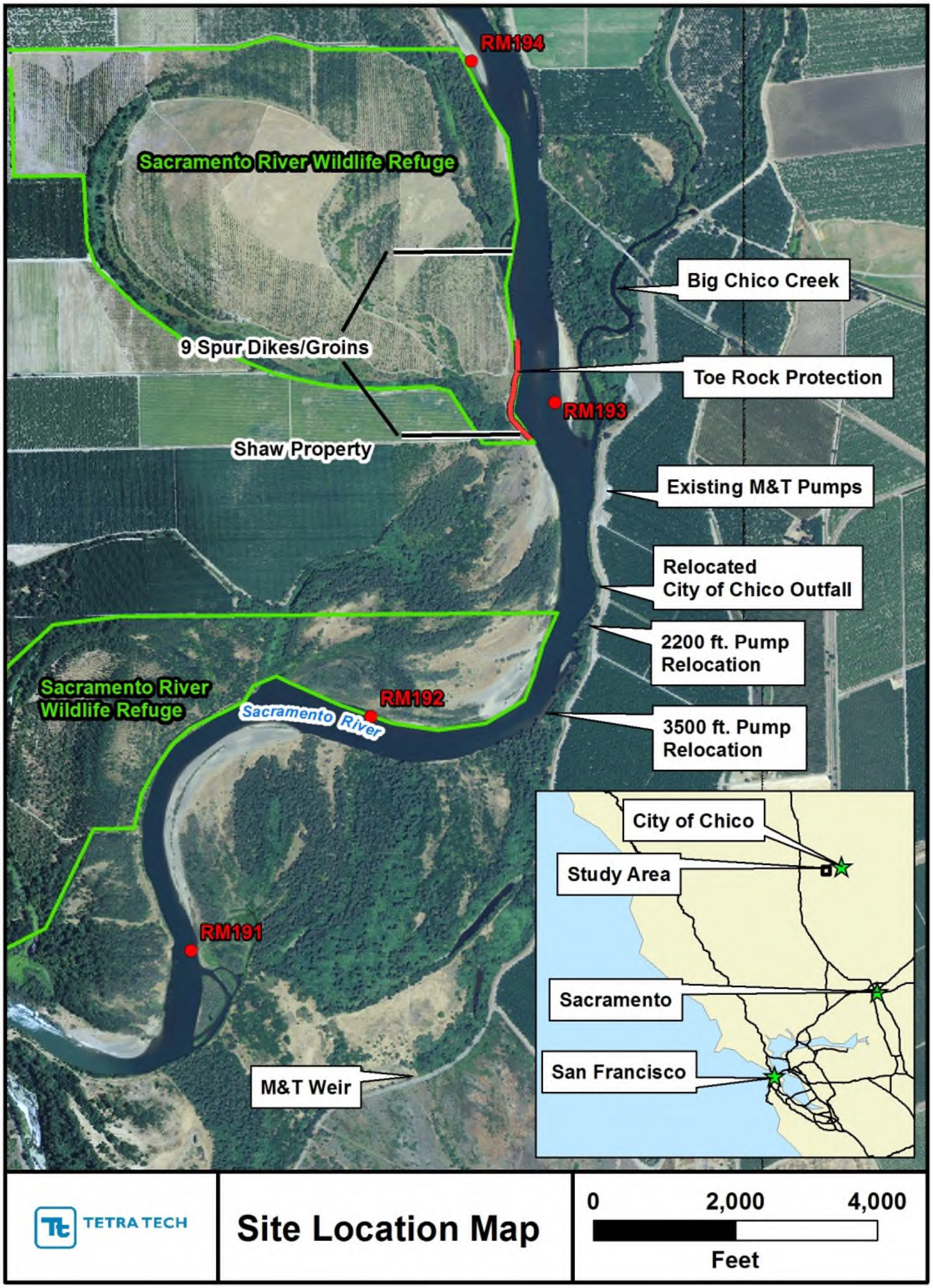


Figure 1. Location of the Sacramento River and the study site.

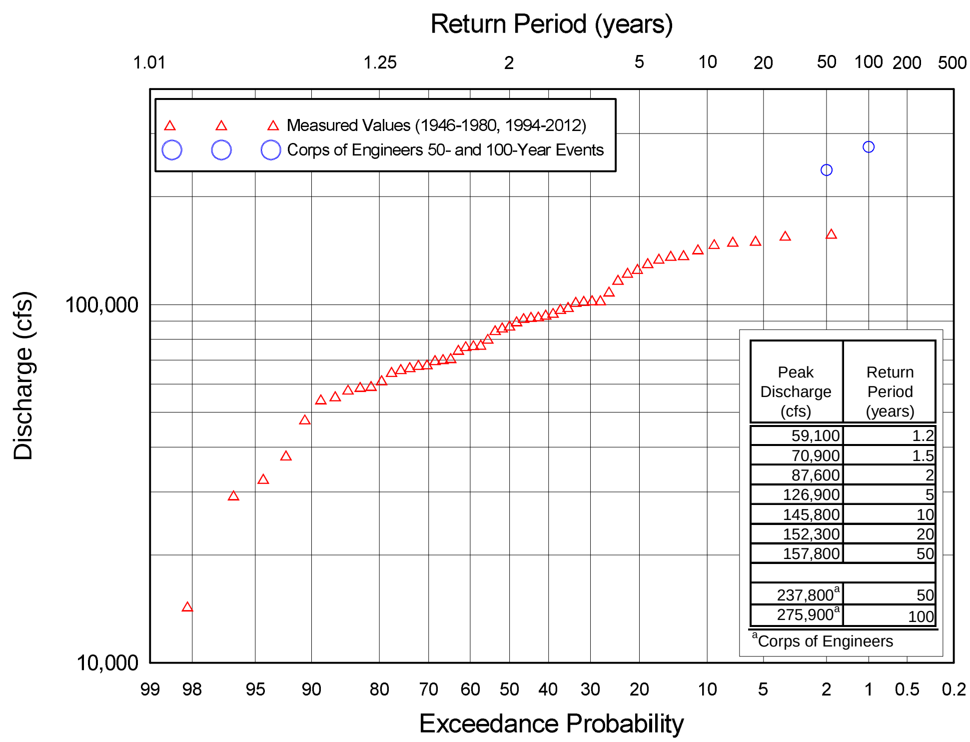


Figure 2. Post-Shasta Dam flood-frequency curve for Hamilton City gaging station.

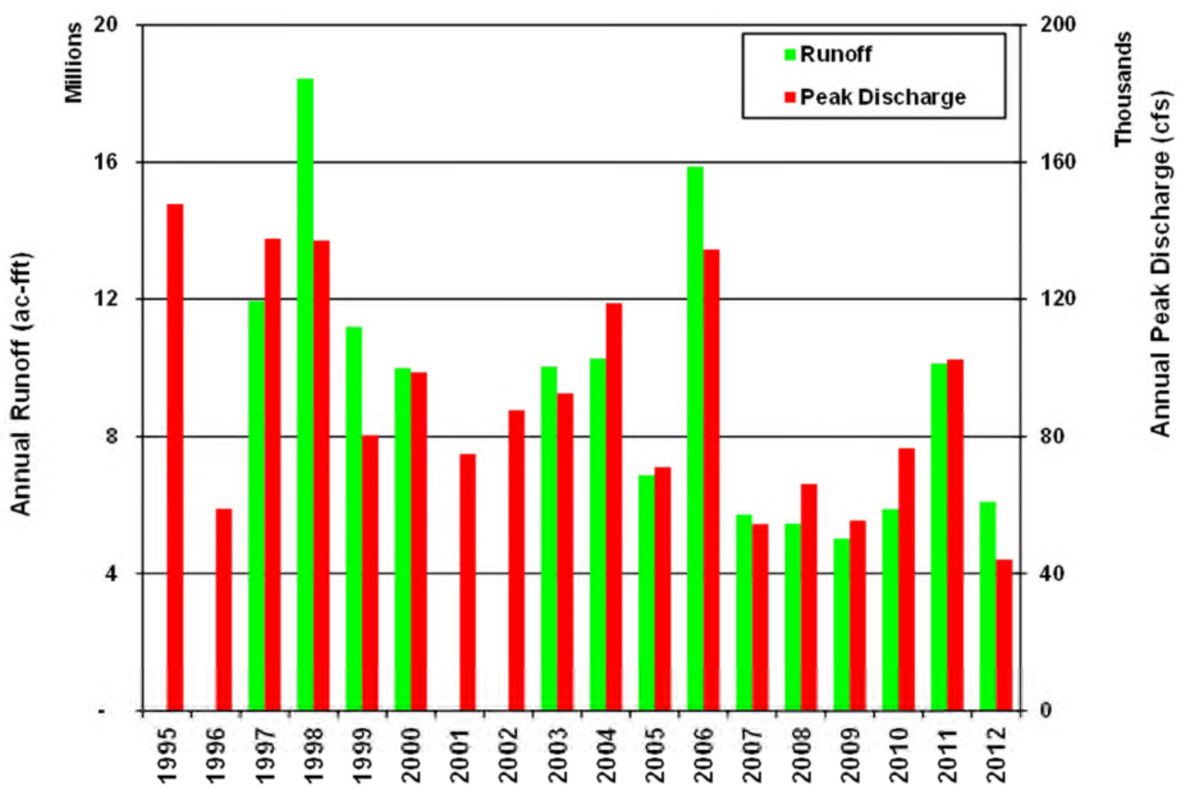


Figure 3. Annual peak flow and volume runoff for the period from WY1995 to WY2012.

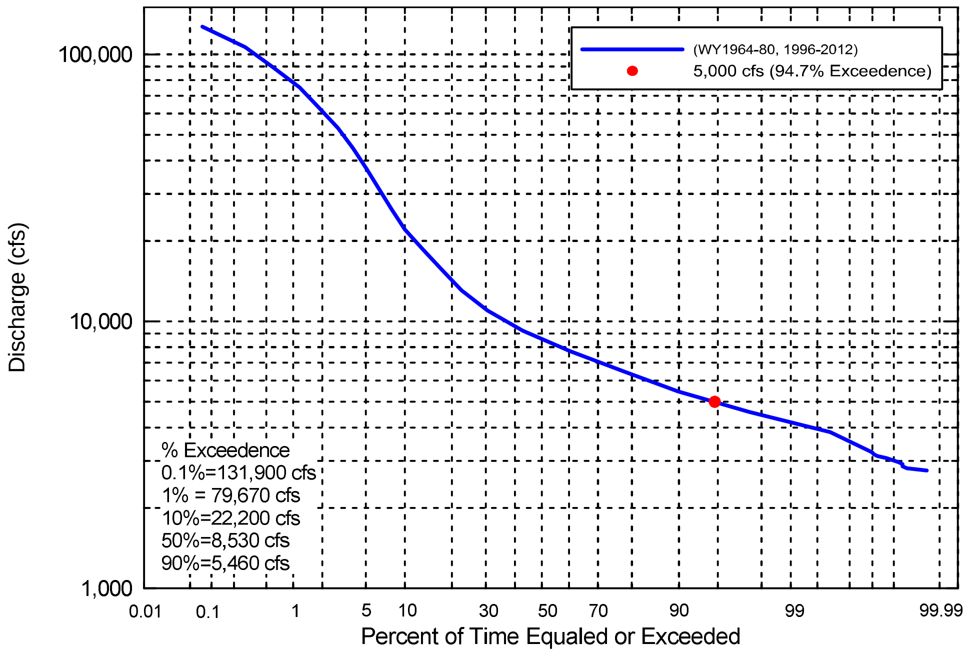


Figure 4. Post-Shasta flow-duration curve for Hamilton City gaging station.

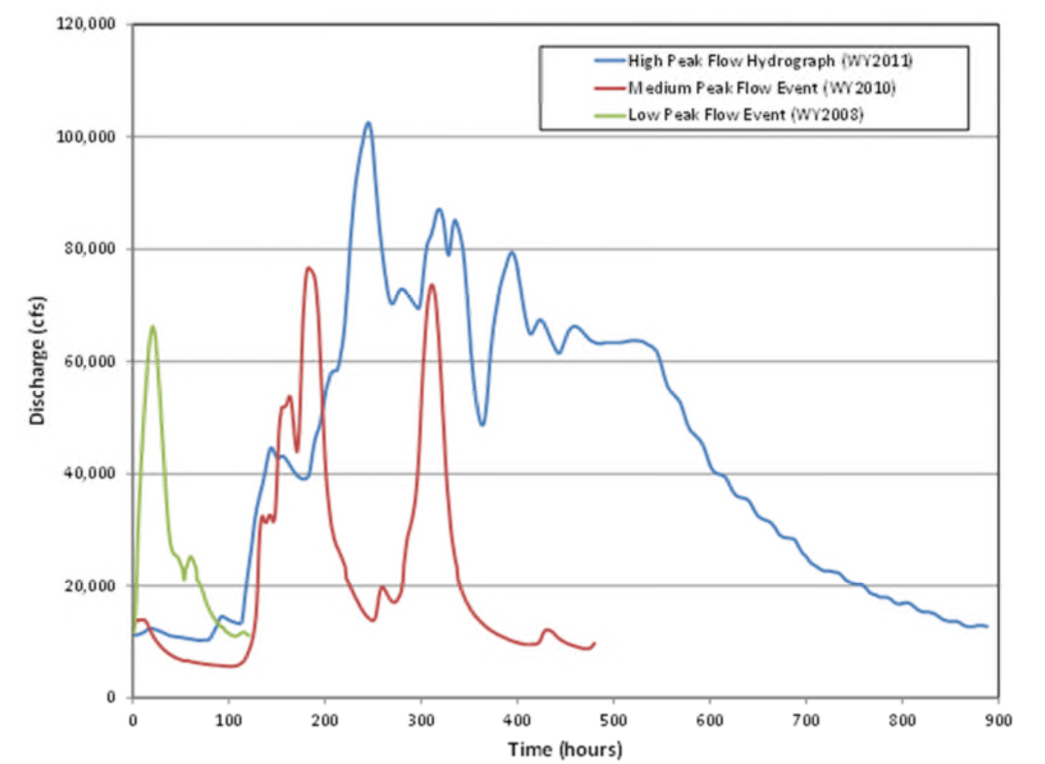


Figure 5. Representative hydrographs for the high, medium and low peak flow events.

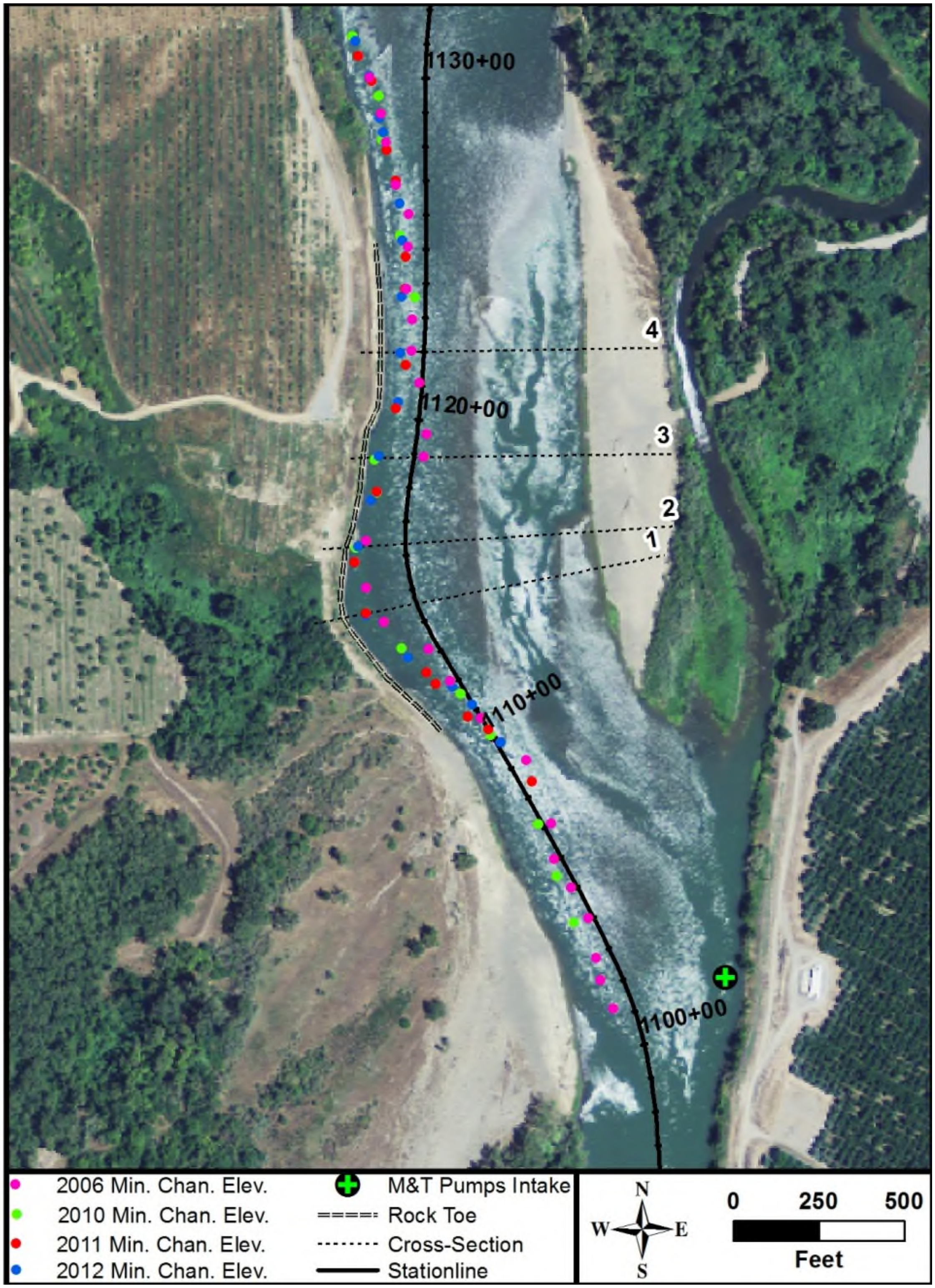


Figure 6. Location of the minimum channel elevation adjacent to the right bank during the 2006, 2010, 2011 and 2012 surveys.

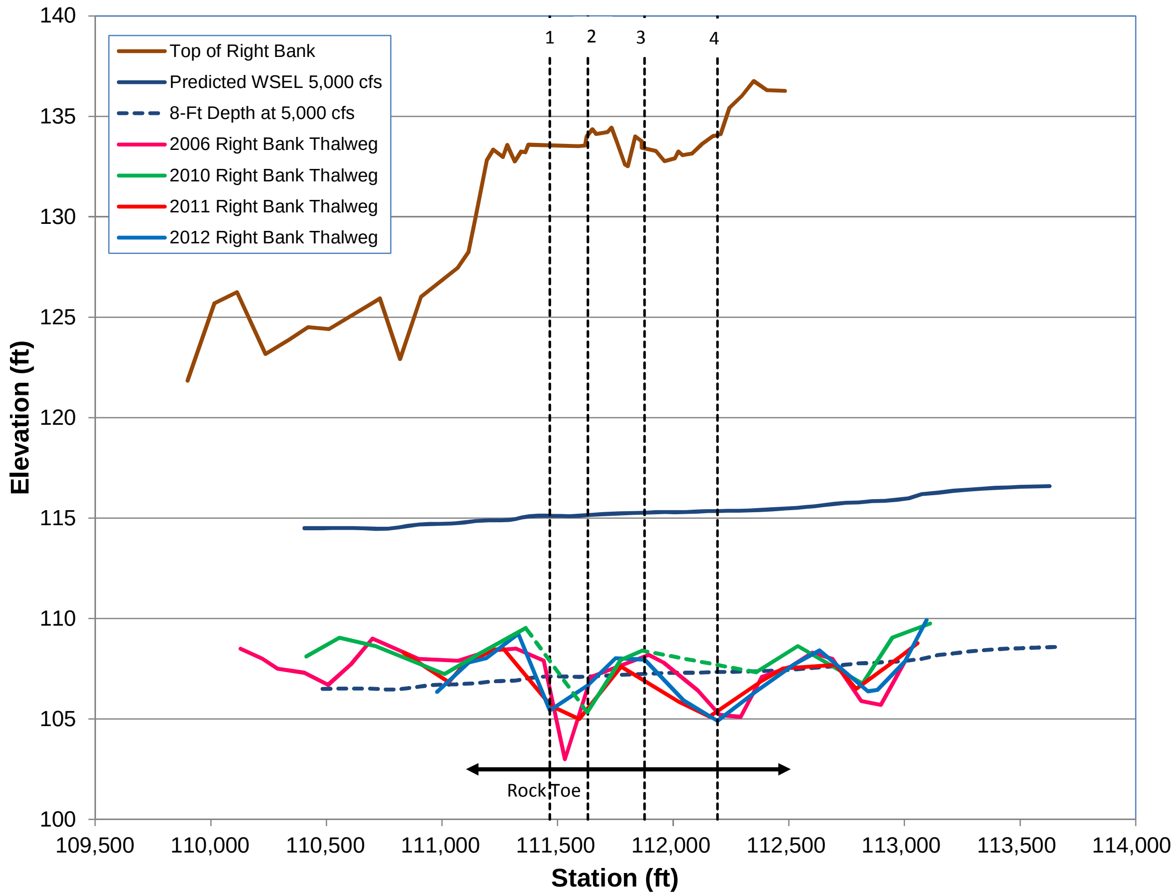


Figure 7. Comparison of the minimum channel elevations adjacent to the right bank, the top of bank elevation, the predicted water-surface elevation at 5,000 cfs and the 8-foot depth line.

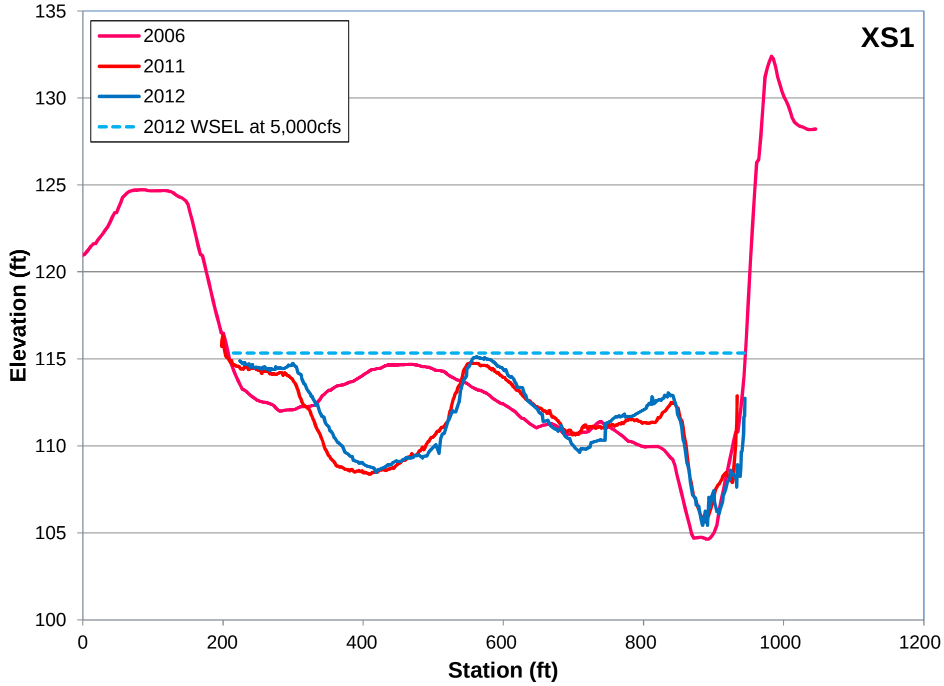


Figure 8a. Comparison of the 2006, 2011 and 2012 surveyed cross sections at XS1. The cross-section for 2010 is not shown, because there were no survey data located near to the cross-section line. The view is looking in a downstream direction and the extents of the cross-section are shown in Figure 1.

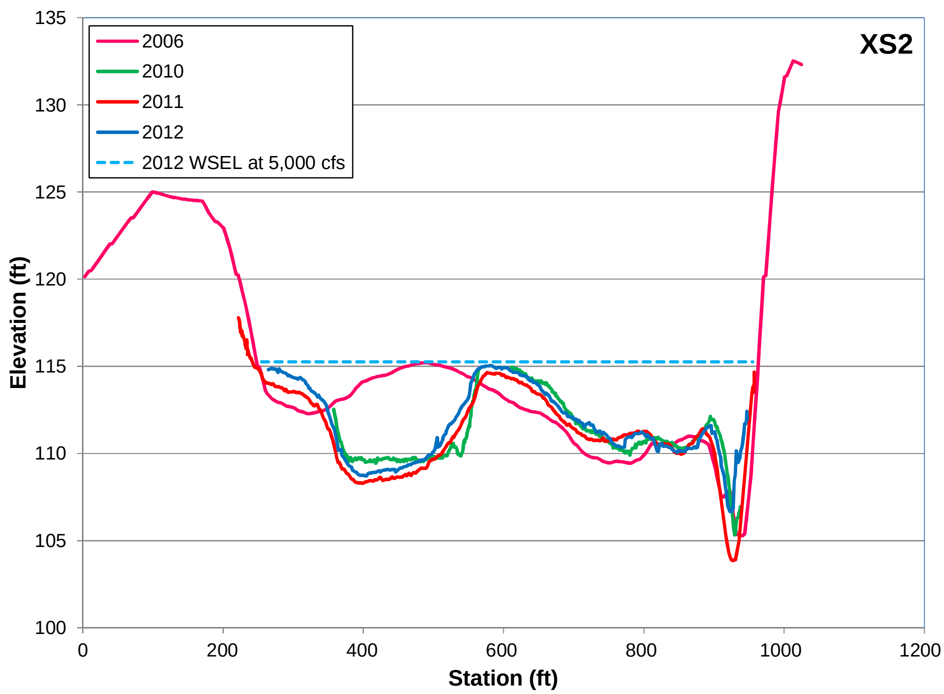


Figure 8b. Comparison of the 2006, 2010, 2011 and 2012 surveyed cross-sections at XS2. The view is looking in a downstream direction and the extents of the cross-section are shown in Figure 1.

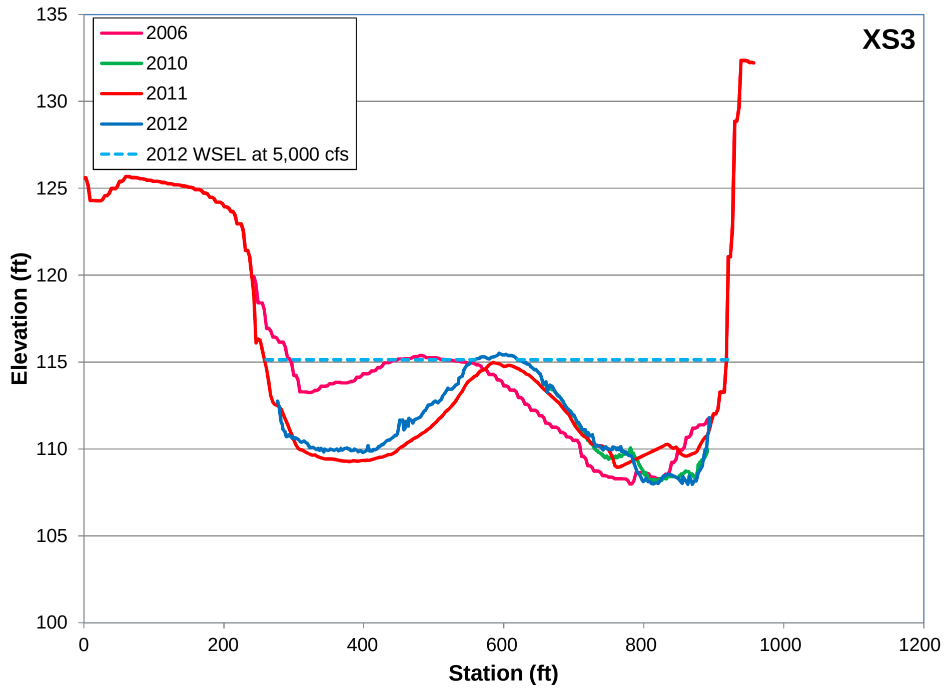


Figure 8c. Comparison of the 2006, 2010, 2011 and 2012 surveyed cross-sections at XS 3. The view is looking in a downstream direction and the extents of the cross-section are shown in Figure 1.

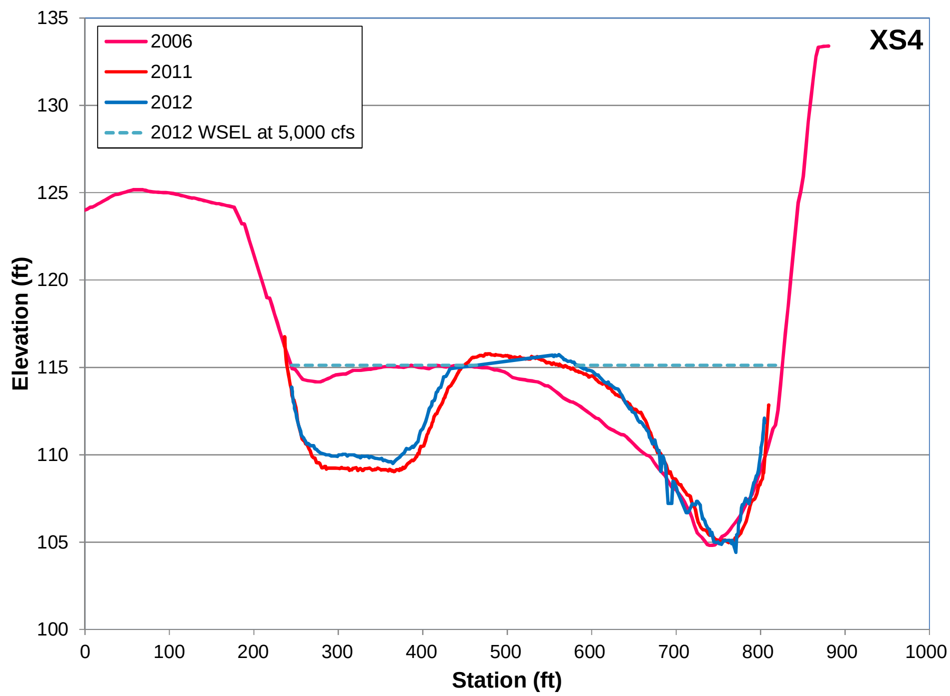


Figure 8d. Comparison of the 2006, 2011 and 2012 surveyed cross-sections at XS 4. The view is looking in a downstream direction and the extents of the cross-section are shown in Figure 1.

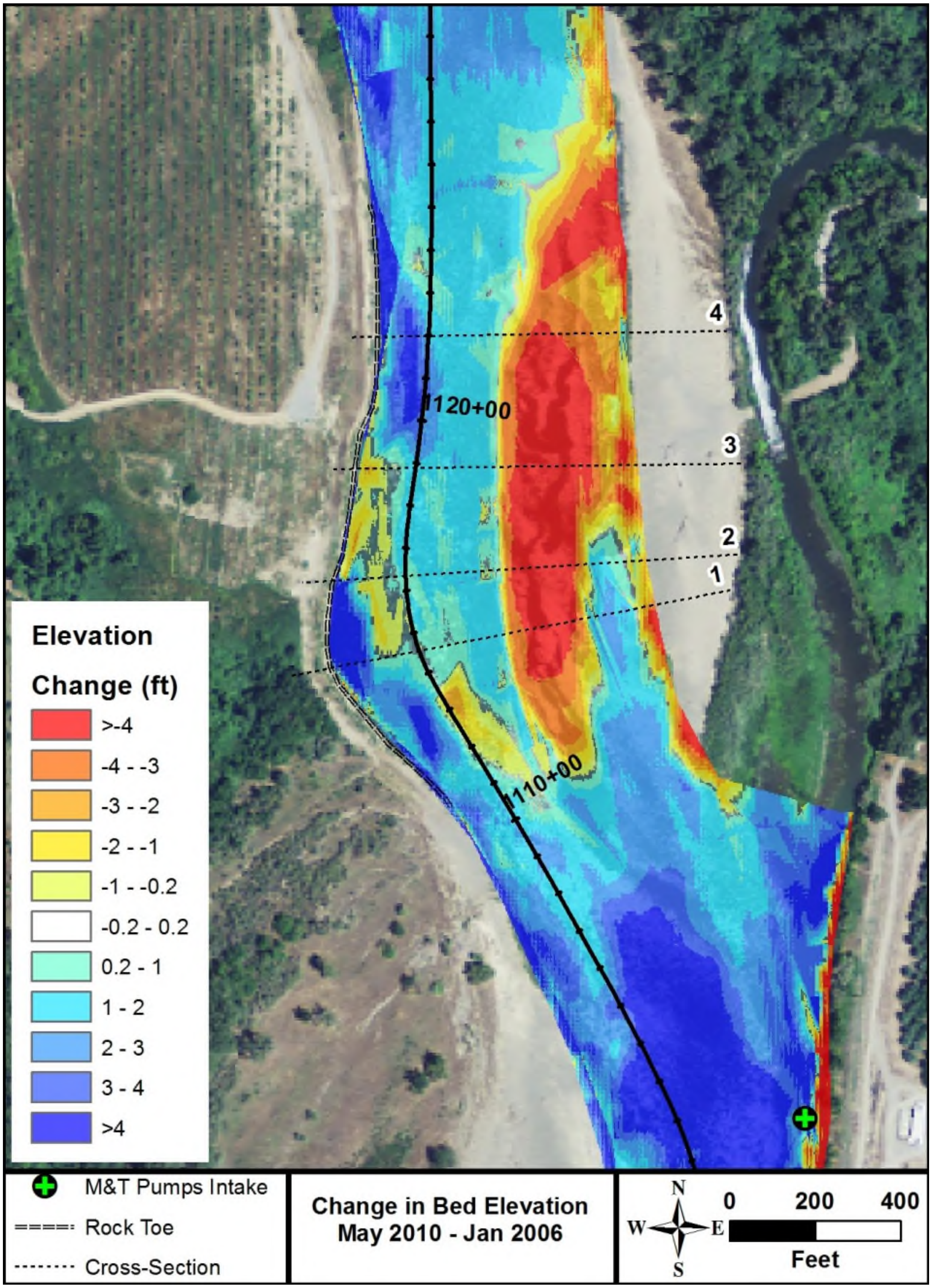


Figure 9. Change in bed elevation between the January 2006 and May 2010. Note: the Rock Toe was installed in 2007.

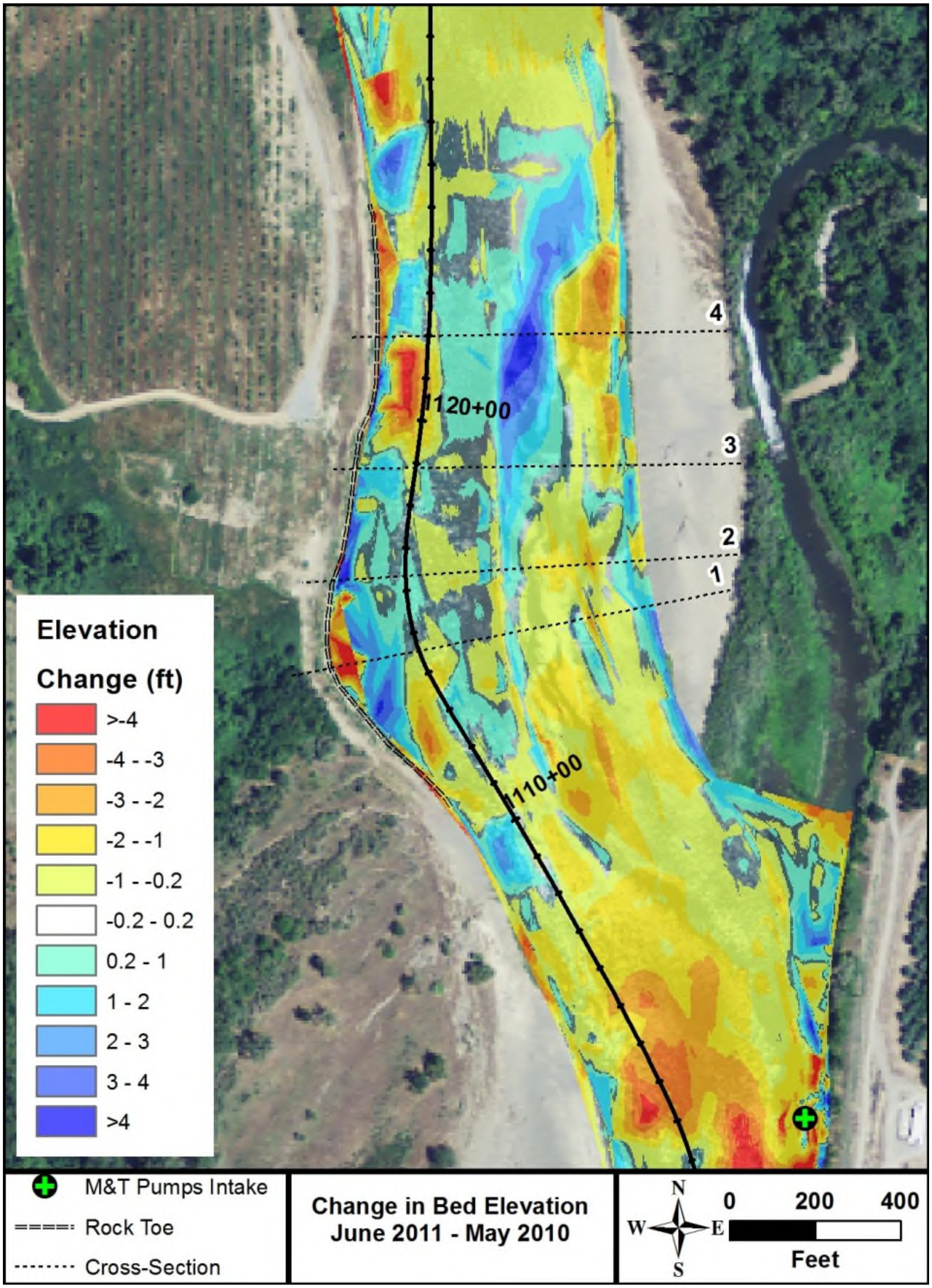


Figure 10. Change in bed elevation between the May 2010 and June 2011.

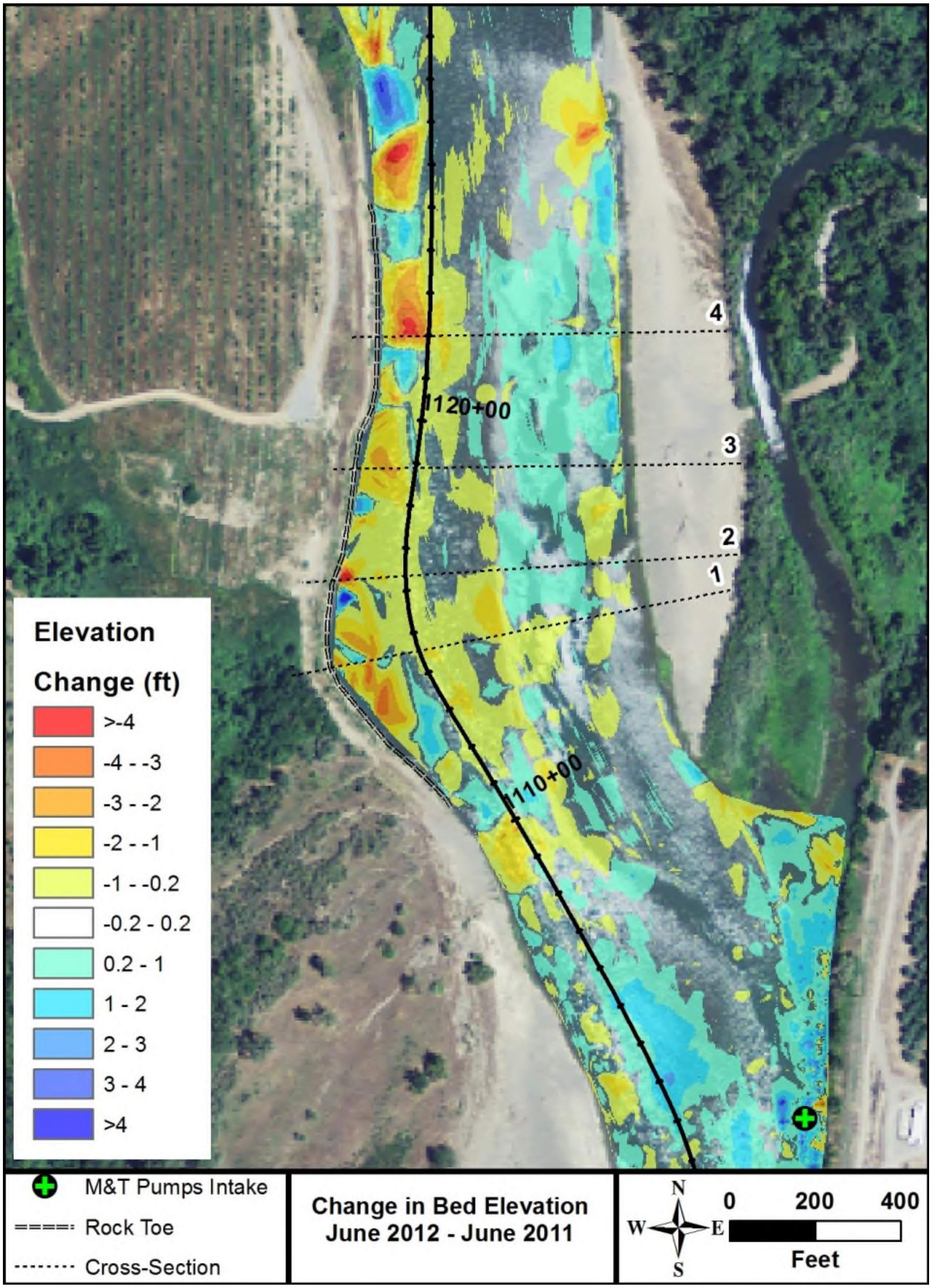


Figure 11. Change in bed elevation between the June 2011 and June 2012.

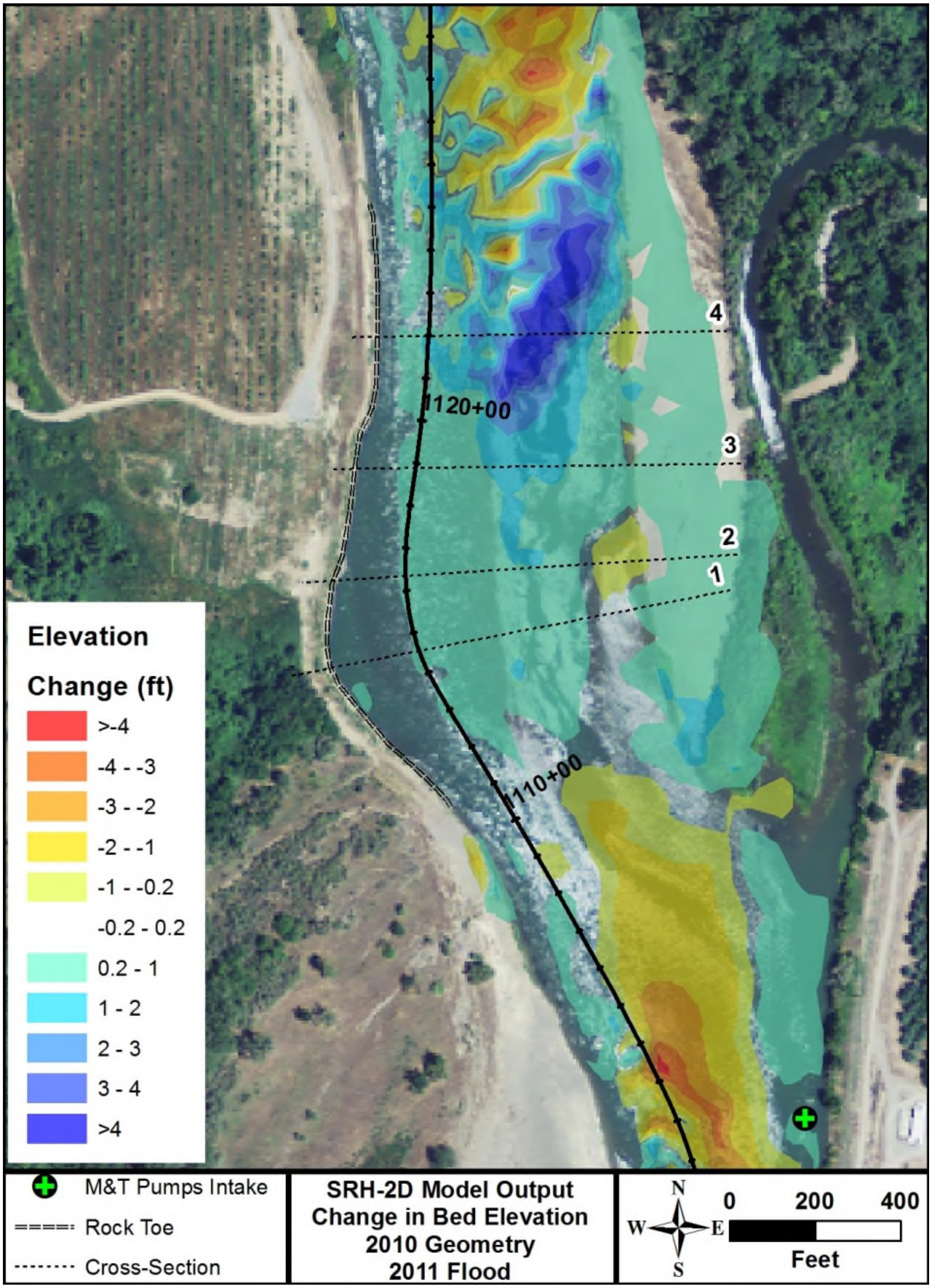


Figure 12. Predicted change in bed elevation over the duration of the 2011 peak flow hydrograph.

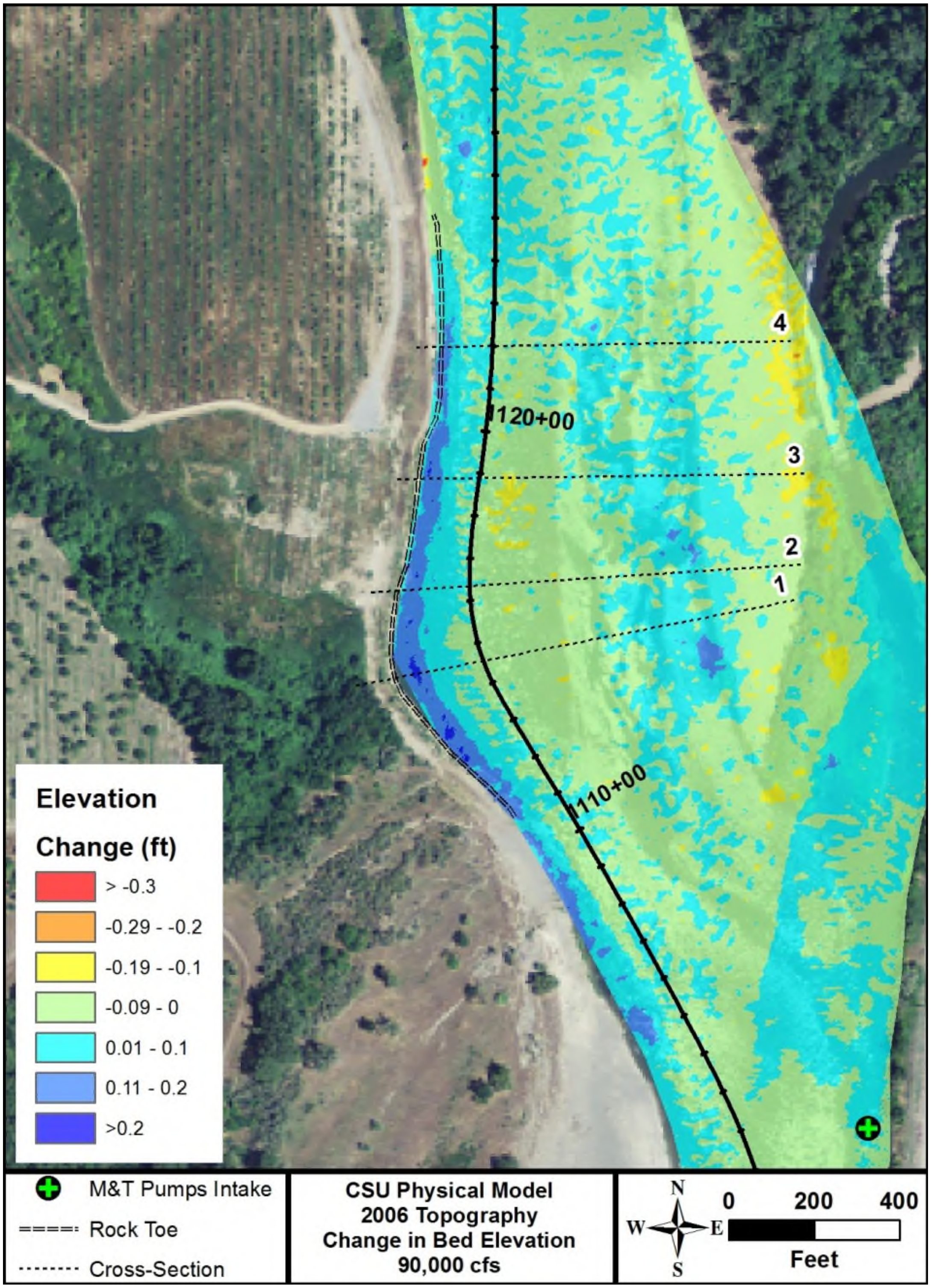


Figure 13. Change in bed elevation in the 2008 Physical model study over the duration of the 90,000 cfs steady state run. Note: the CSU model was developed based on the 2006 topography. The rock toe was constructed in 2007, and therefore, was not included in the physical model. The rock toe is shown on this figure for comparative purposes with the other figures.

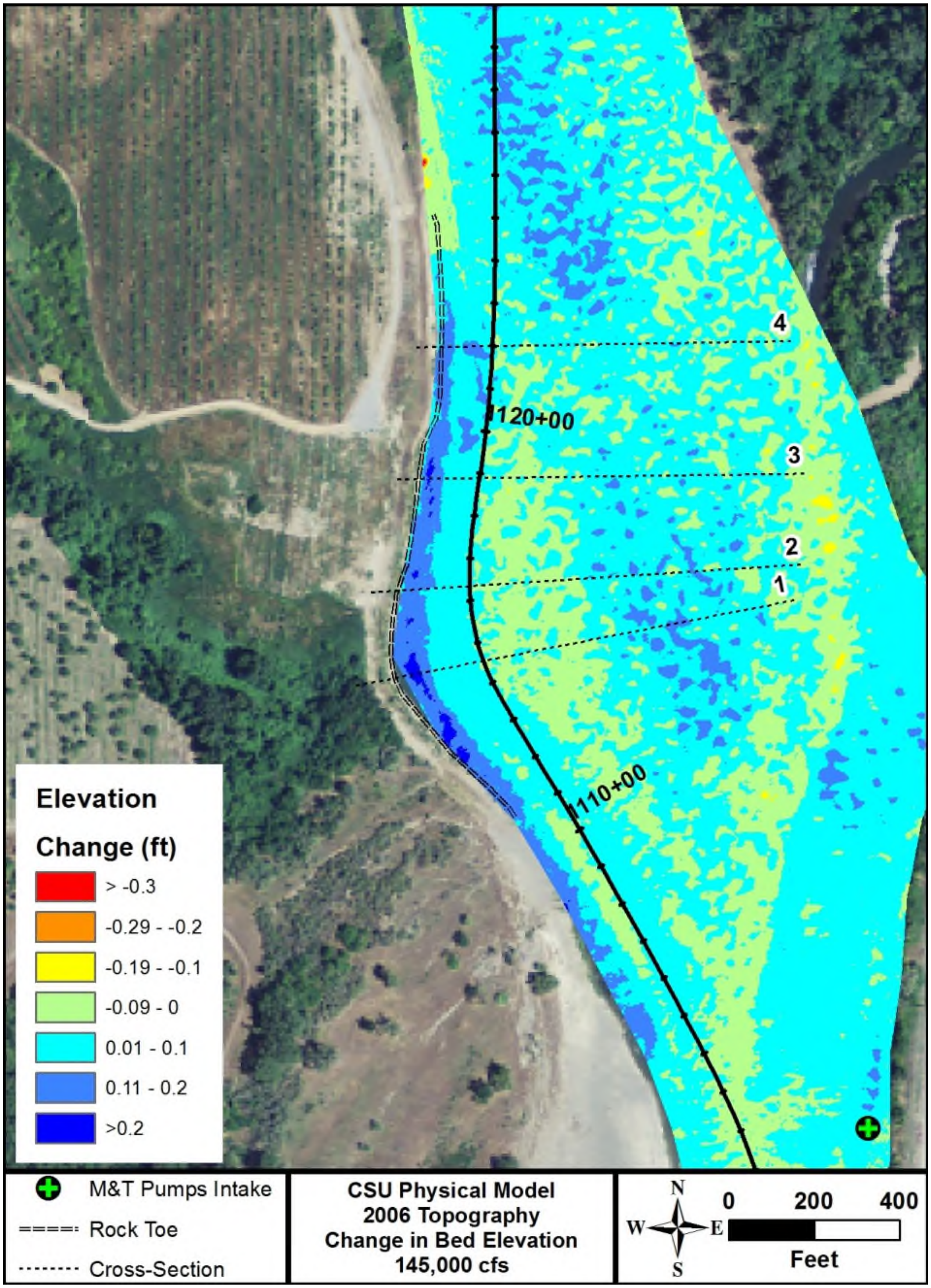


Figure 14. Change in bed elevation in the 2008 Physical model study over the duration of the 145,000-cfs steady state run. Note: the CSU model was developed based on the 2006 topography. The rock toe was constructed in 2007, and therefore, was not included in the physical model. The rock toe is shown on this figure for comparative purposes with the other figures.

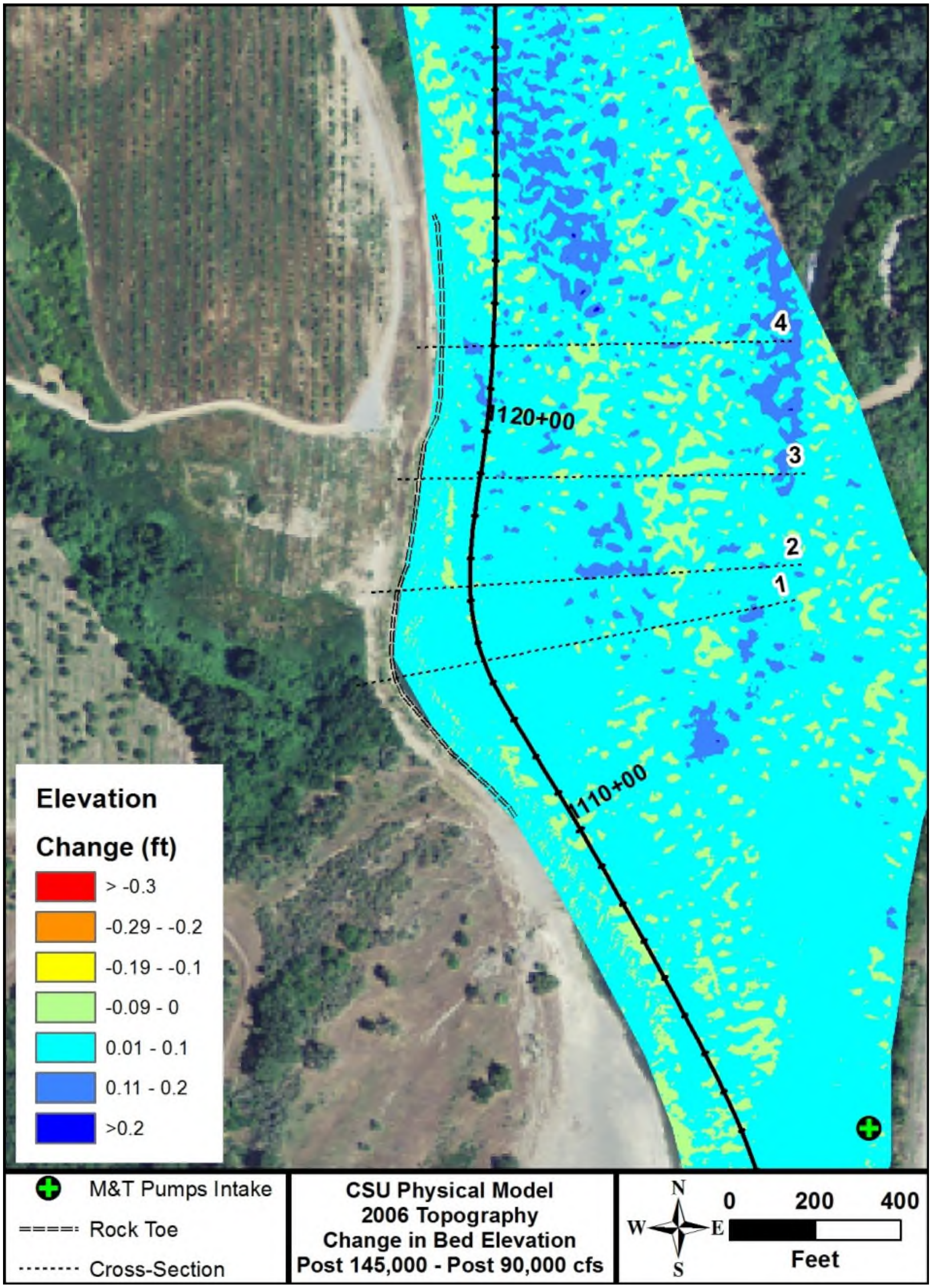


Figure 15. Difference in bed elevation in the 2008 Physical model study between the end of the 145,000- and 90,000-cfs runs. This figure was modified from Figure 5.56 in the CSU (2008) report. Note: the CSU model was developed based on the 2006 topography. The rock toe was constructed in 2007, and therefore, was not included in the physical model. The rock toe is shown on this figure for comparative purposes with the other figures.

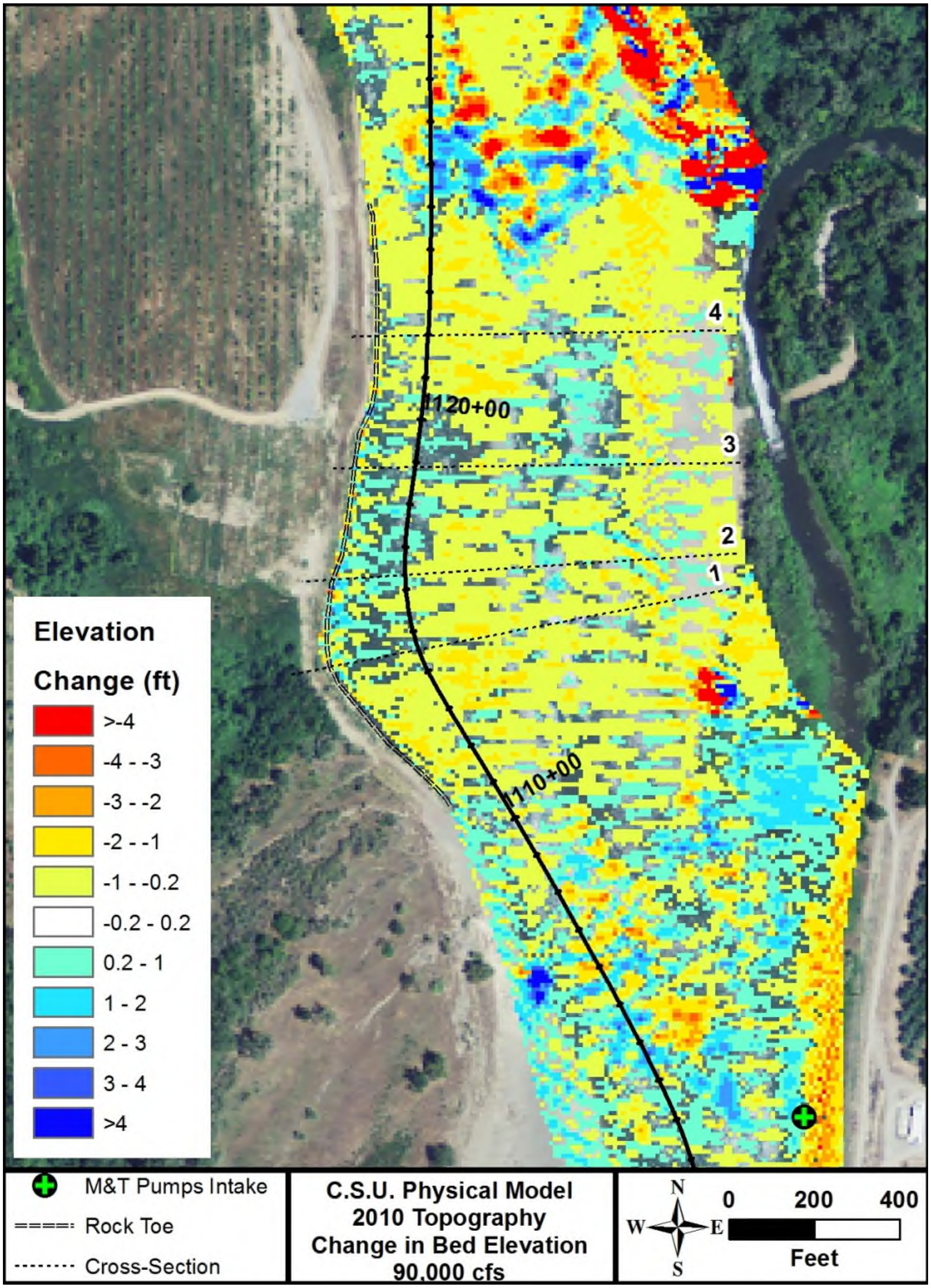


Figure 16. Change in bed elevation between the end of the 90,000-cfs model run and the end of the 10,000-cfs model run. Figure was modified from the CSU (2011) physical model study.

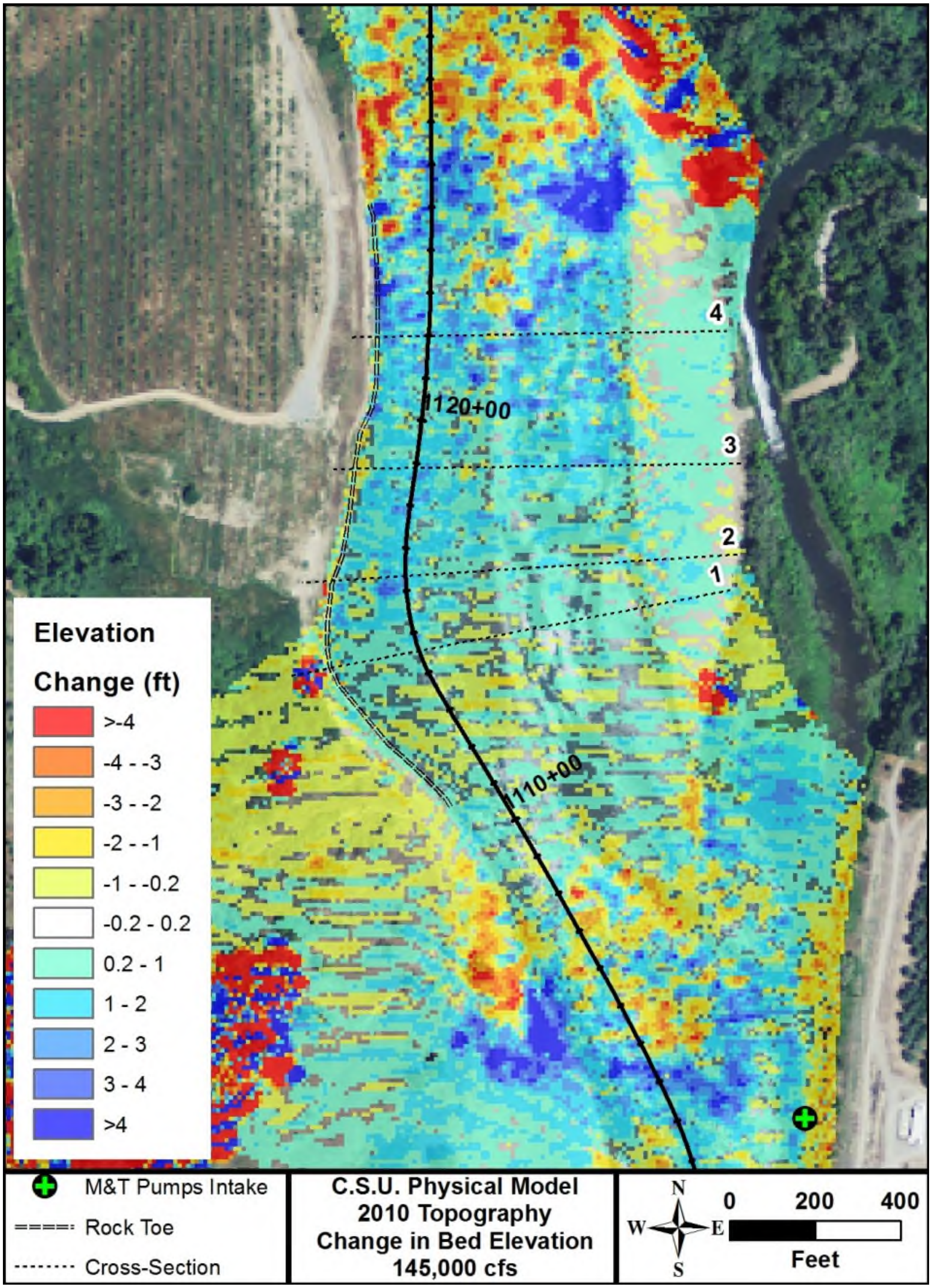


Figure 17. Change in bed elevation between the end of the 145,000-cfs model run and the end of the 10,000-cfs model run. Figure was modified from the CSU (2011) physical model study.

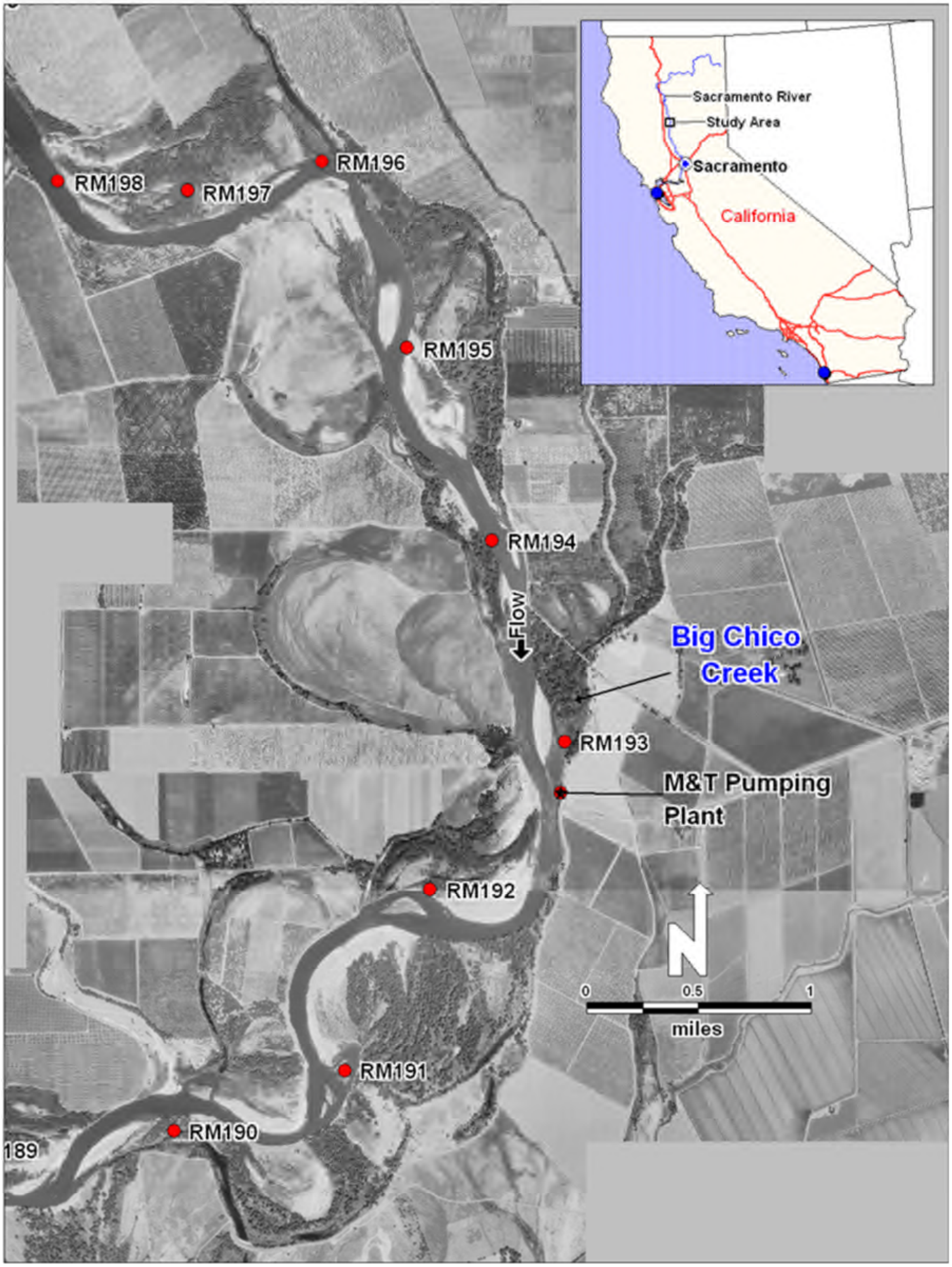


Figure 18. Sacramento River Mile (RM) stationing.

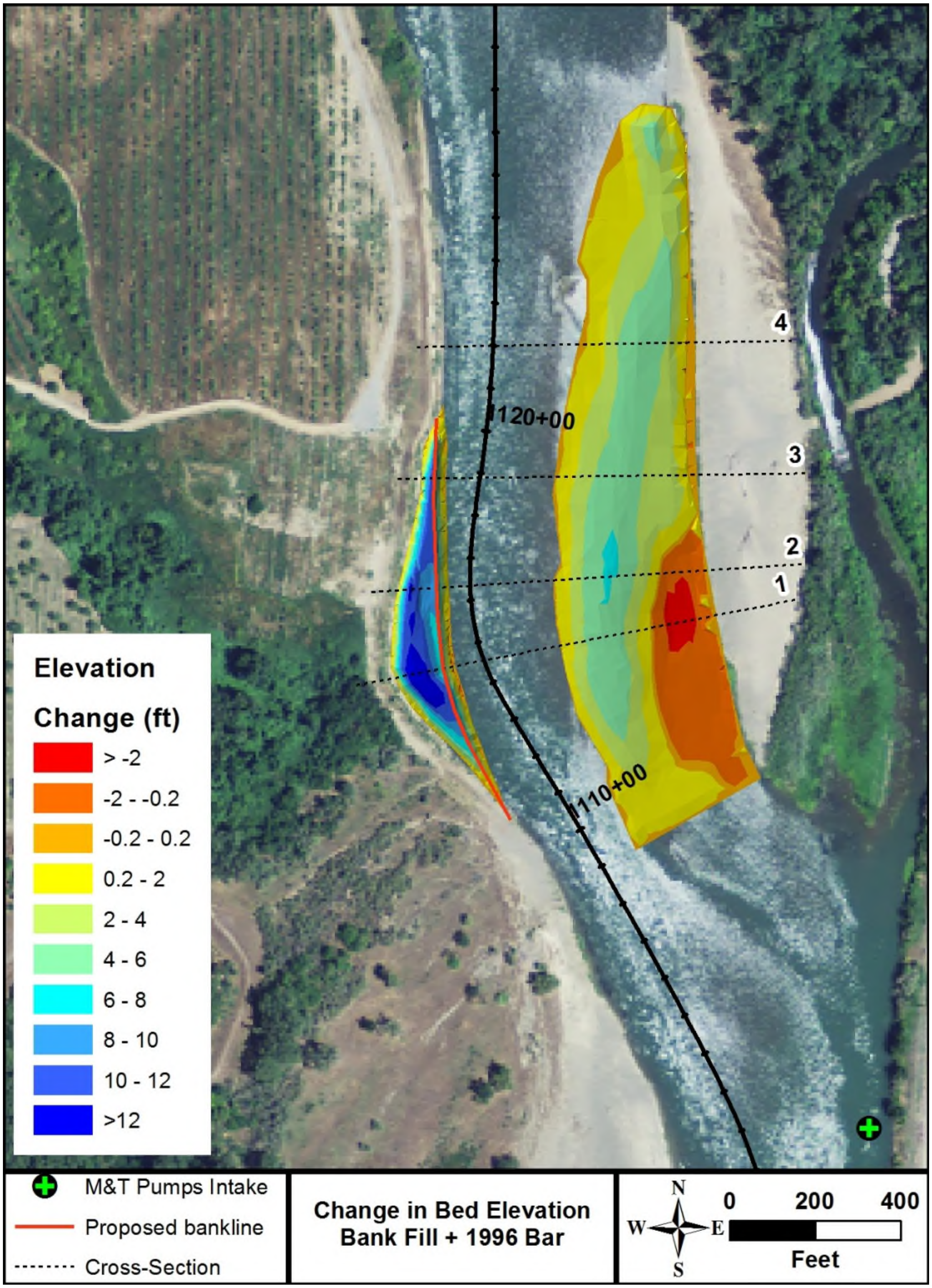


Figure 19. Proposed west bank re-alignment and SRH-2D mesh elevation changes to represent the bank fill and 1996 bar.

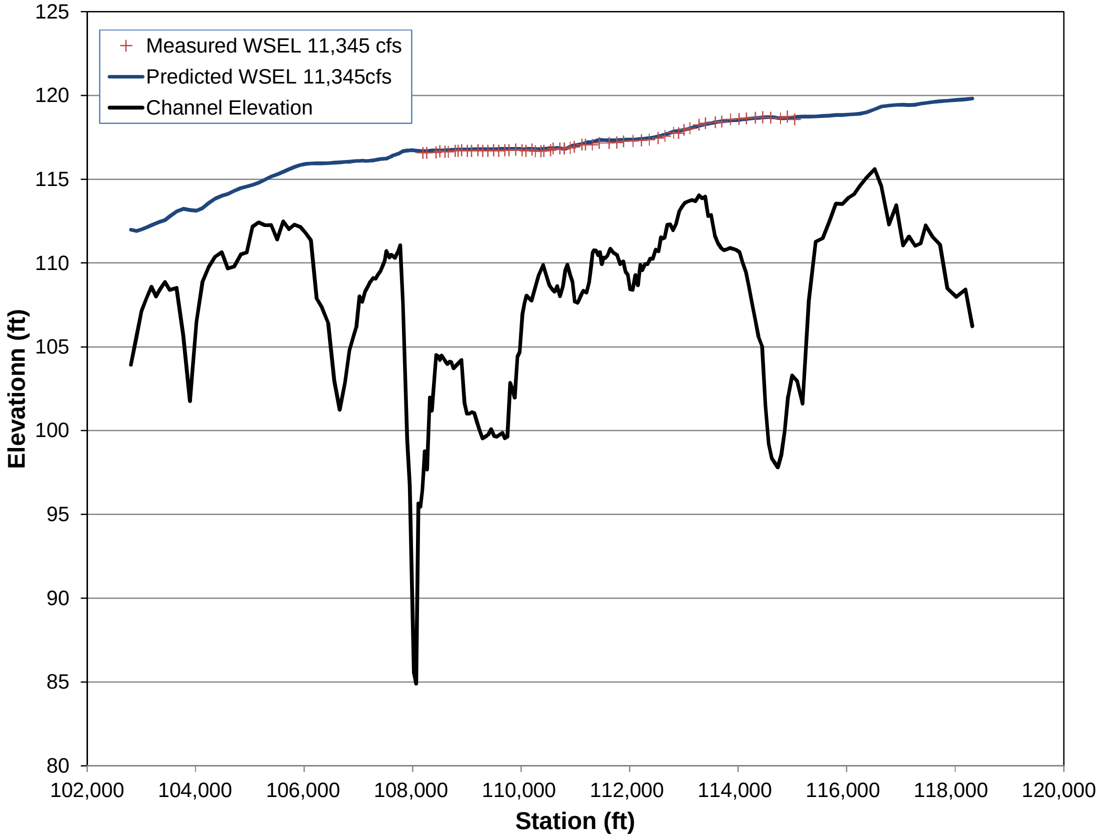


Figure 20. Comparison between the measured and predicted water-surface elevations at 11,345 cfs.

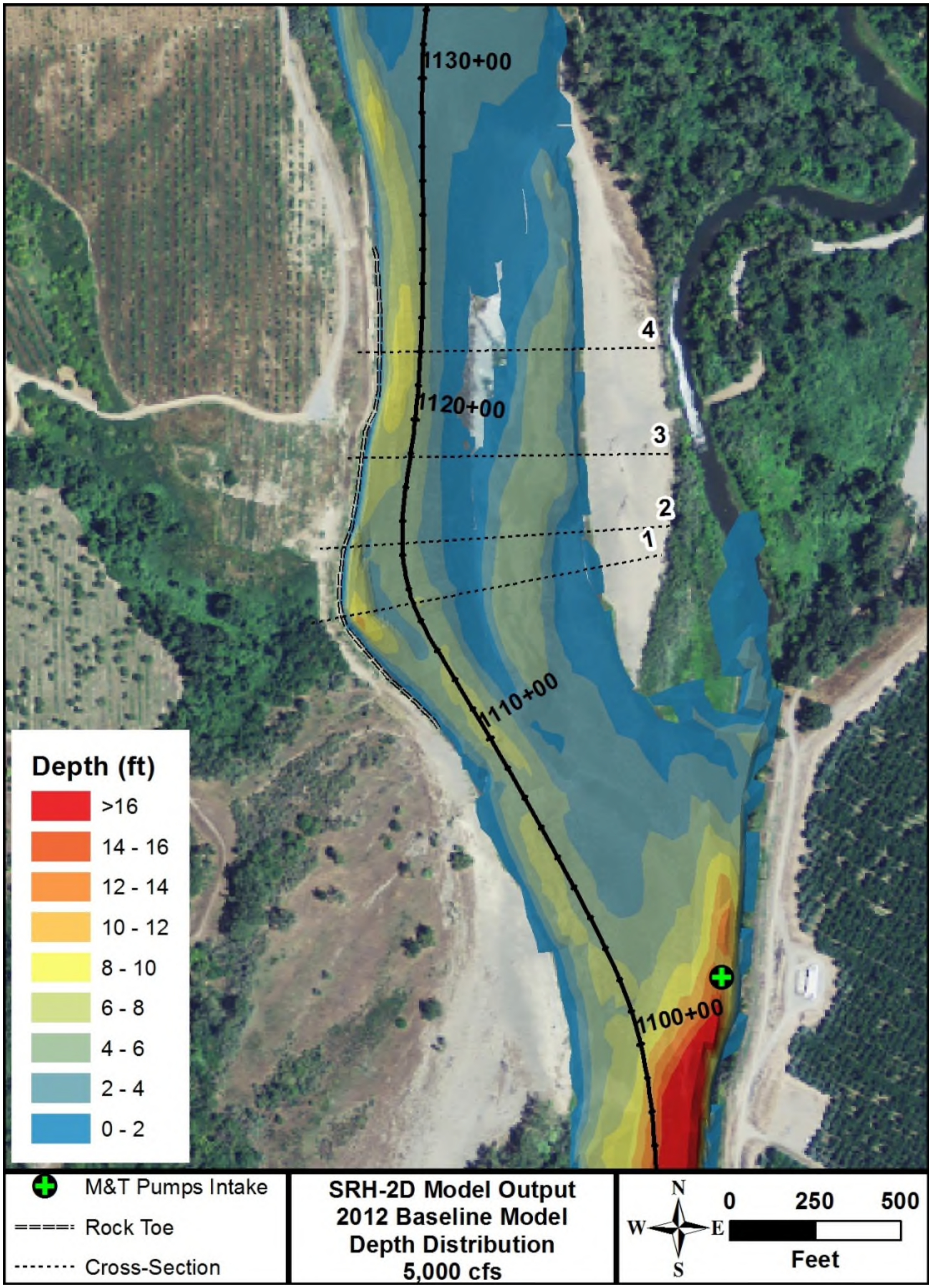


Figure 21. Predicted depth distribution at 5,000 cfs

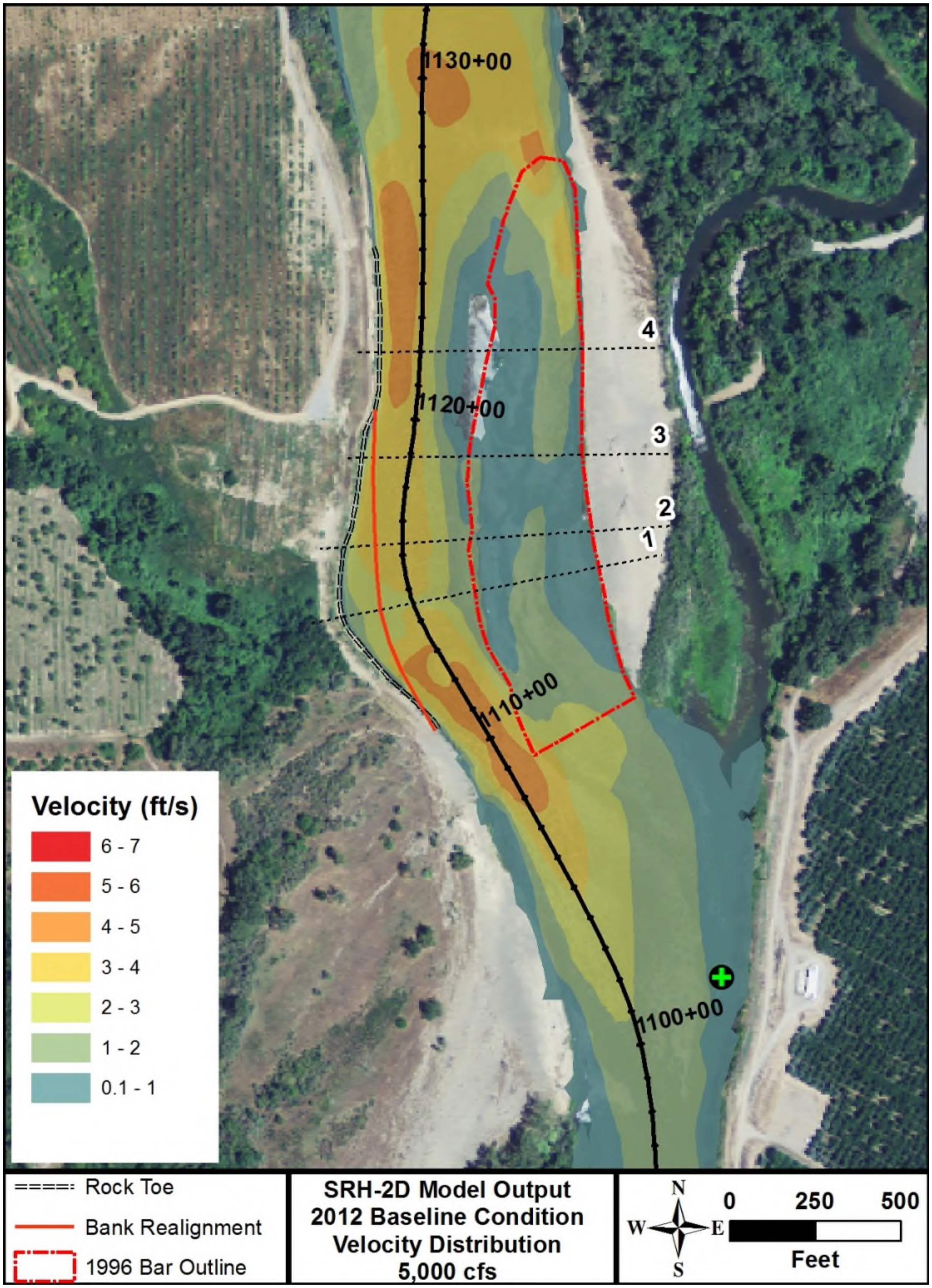


Figure 22. Predicted velocity distribution at 5,000 cfs.

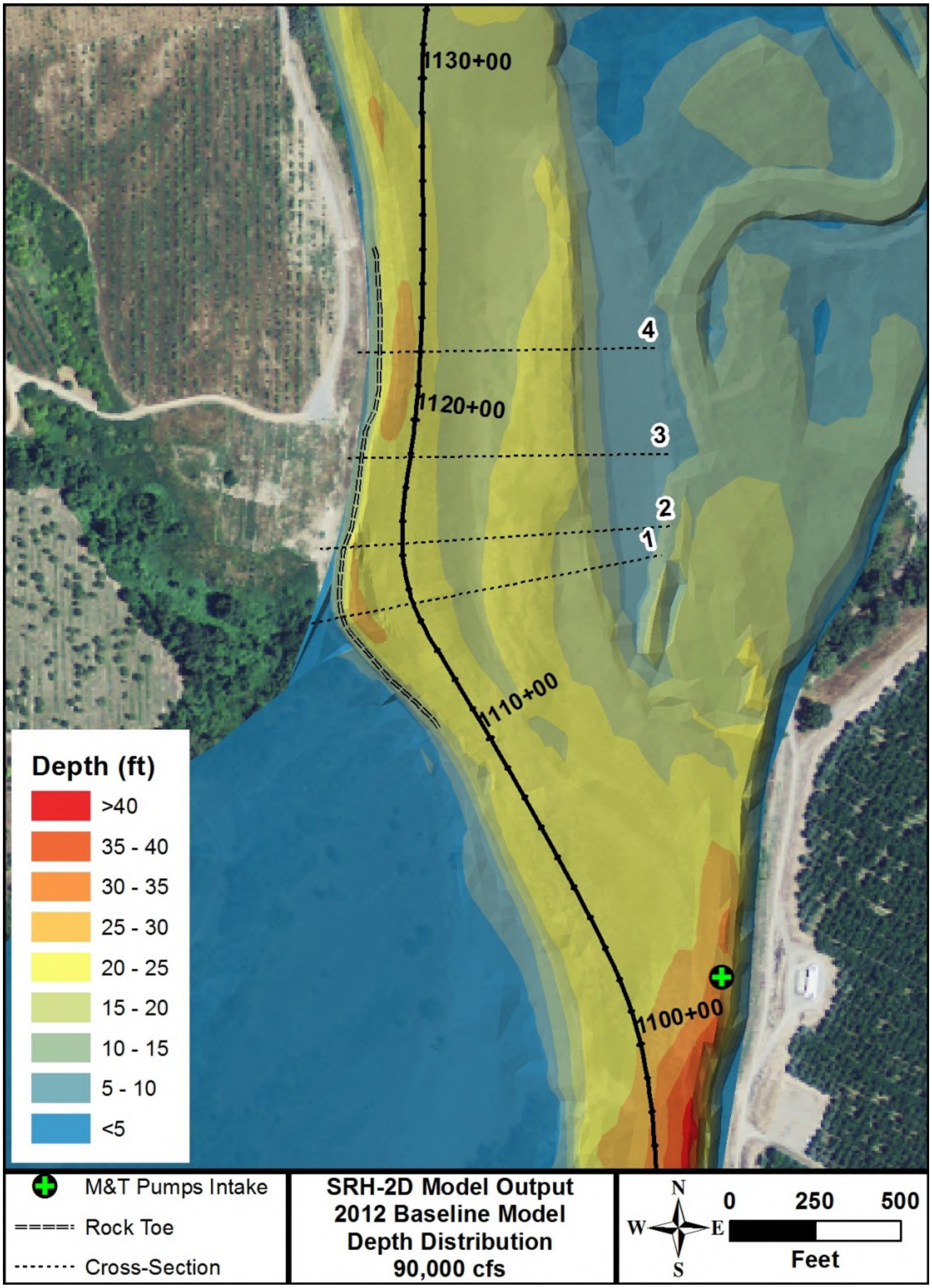


Figure 23. Predicted depth distribution at 90,000 cfs.

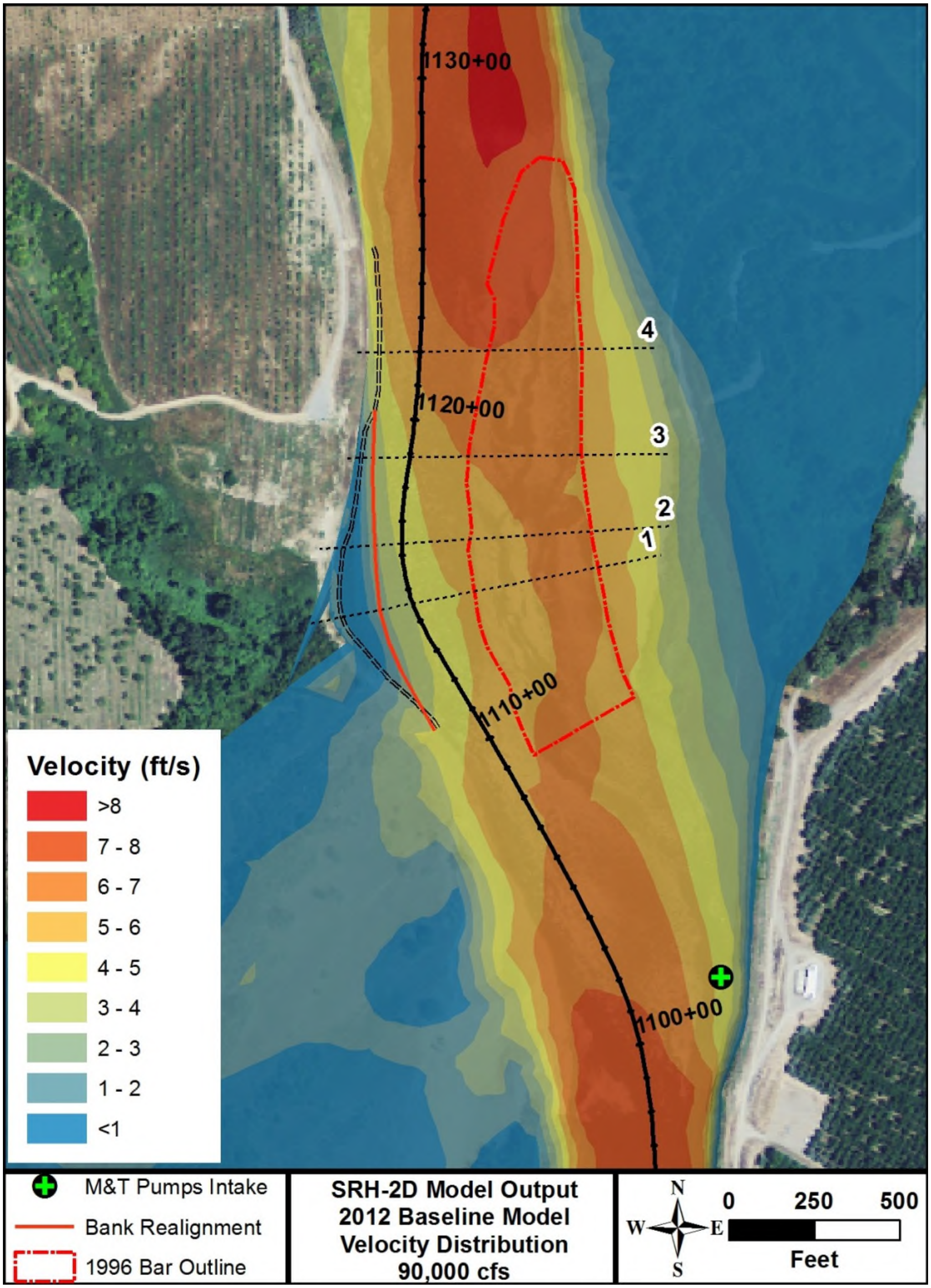


Figure 24. Predicted velocity distribution at 90,000 cfs.

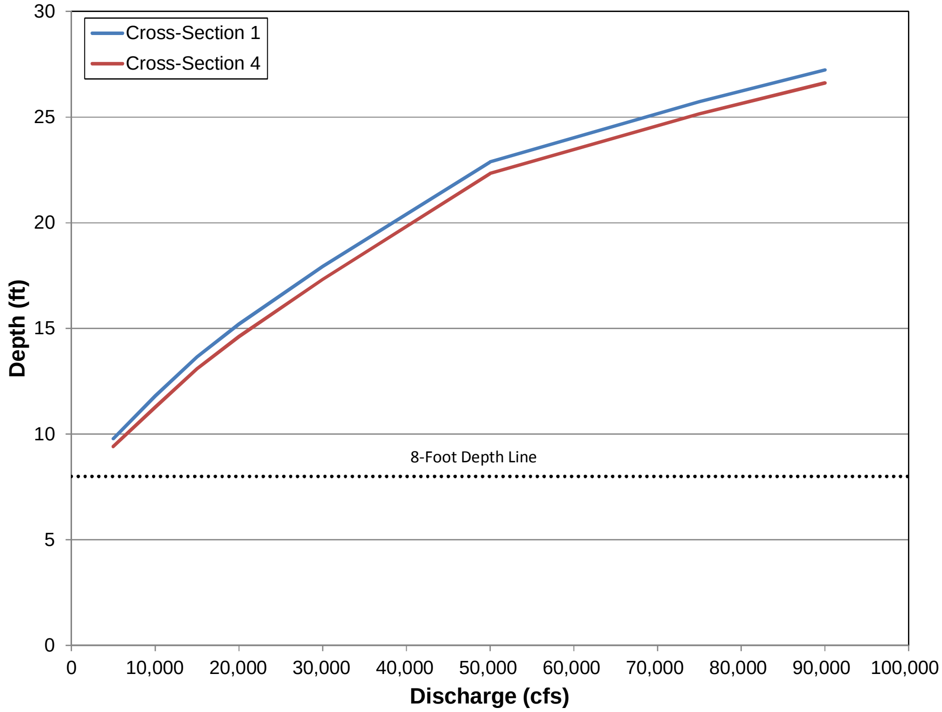
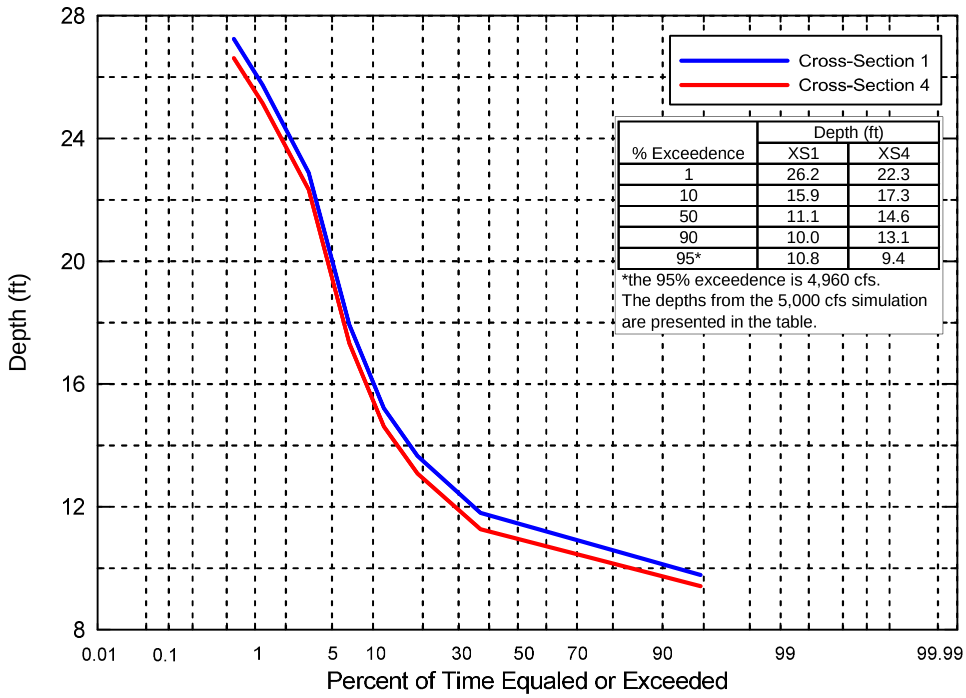


Figure 25. The predicted depth-discharge rating curves at Cross Sections 1 and 4.



% Exceedence	Depth (ft)	
	XS1	XS4
1	26.2	22.3
10	15.9	17.3
50	11.1	14.6
90	10.0	13.1
95*	10.8	9.4

Figure 26. Mean daily-depth curves at Cross Sections 1 and 4.

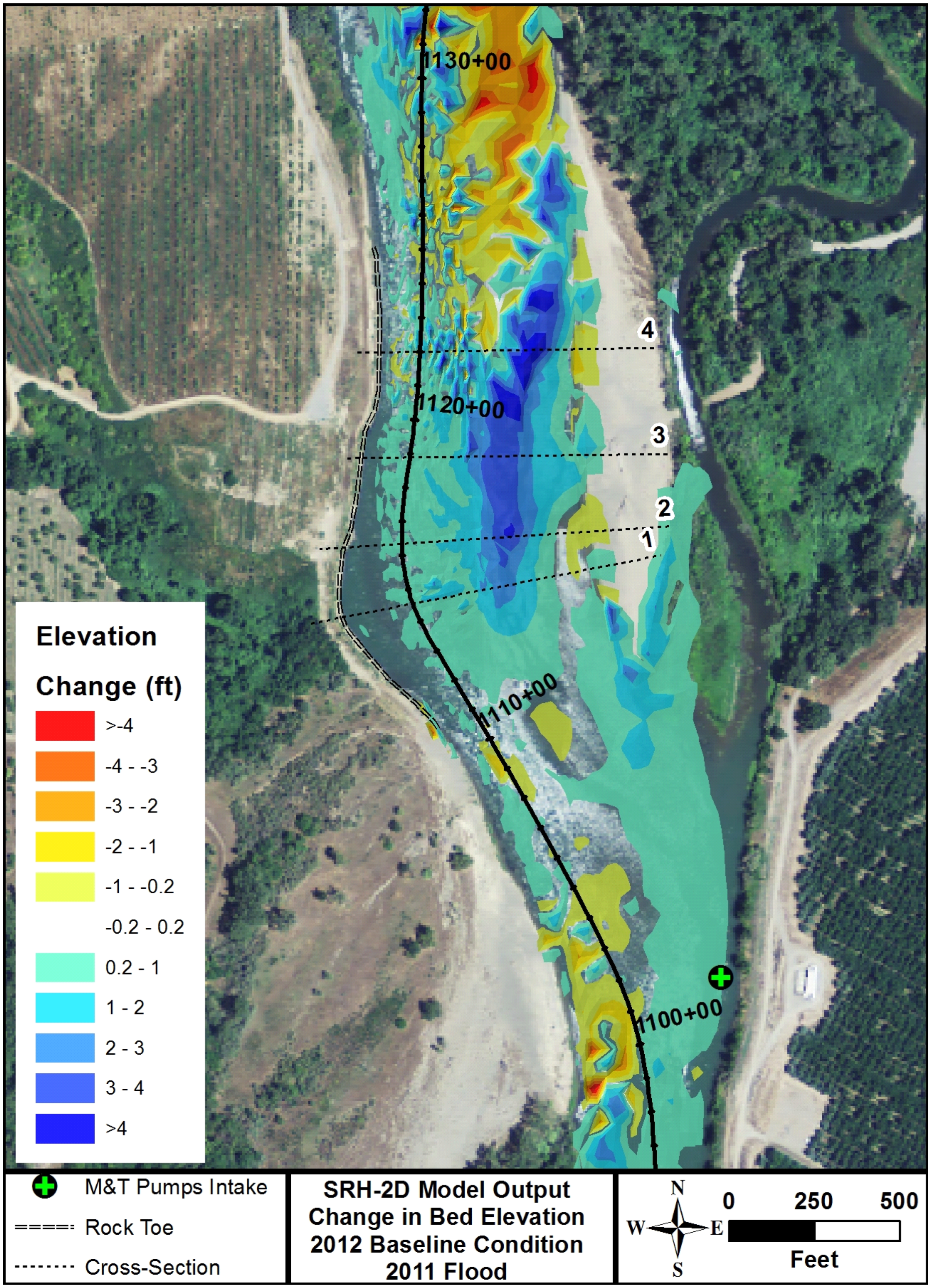


Figure 27. Predicted change in bed elevation for the 2012 Baseline Conditions over the duration of the 2011 peak flow hydrograph.

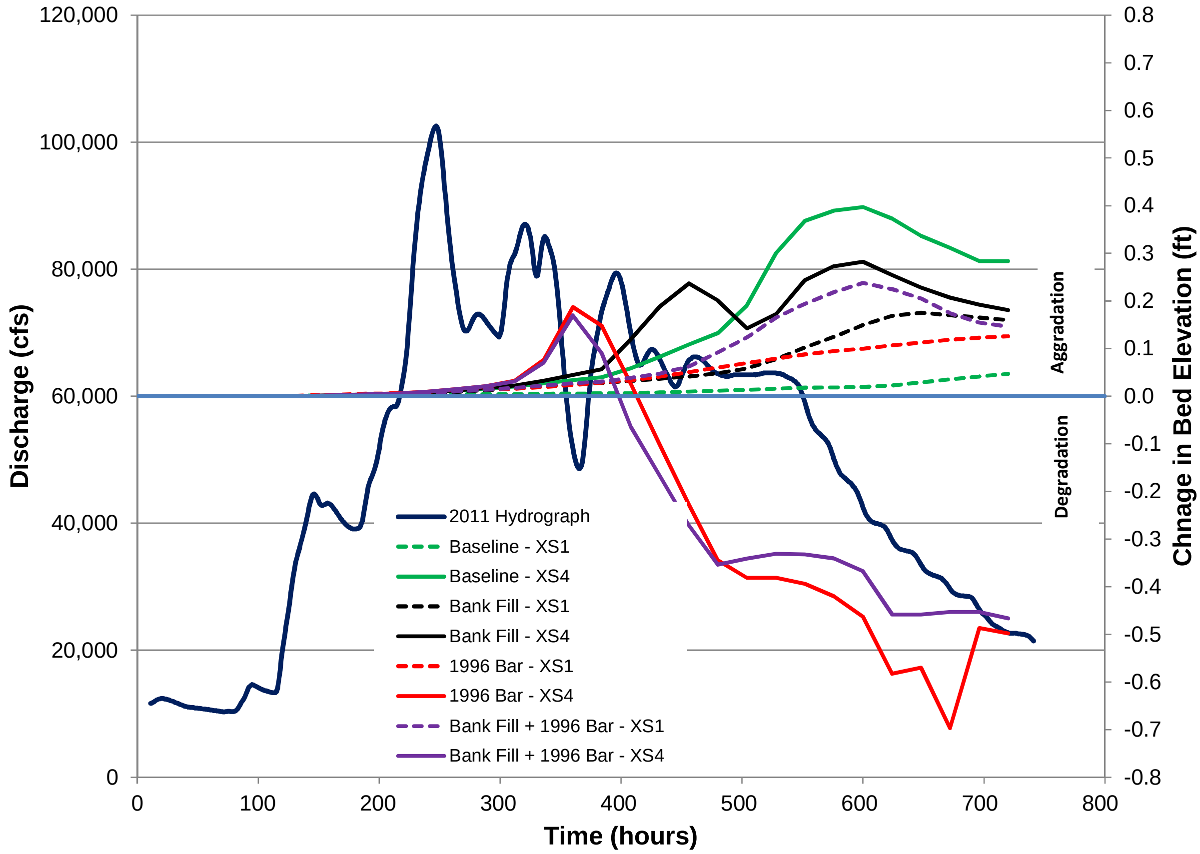


Figure 28. Predicted change in bed elevation at XS1 and XS4 for the 2012 Baseline Conditions, Bank Fill, 1996 Bar and Bank Fill + 1996 Bar conditions over the duration of the 2011 peak flow hydrograph.

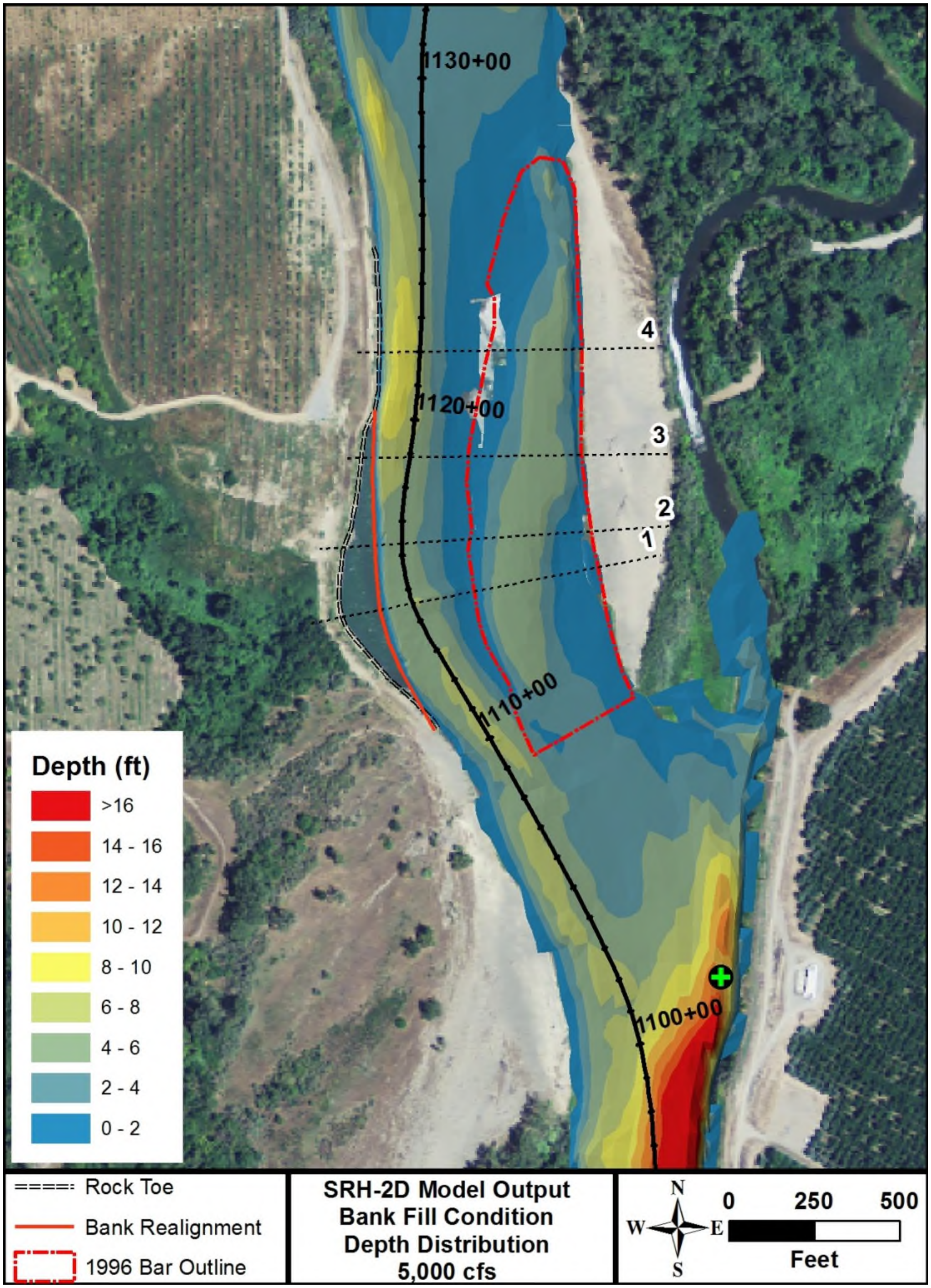


Figure 29. Predicted depth distribution for the Bank Fill Conditions at 5,000 cfs.

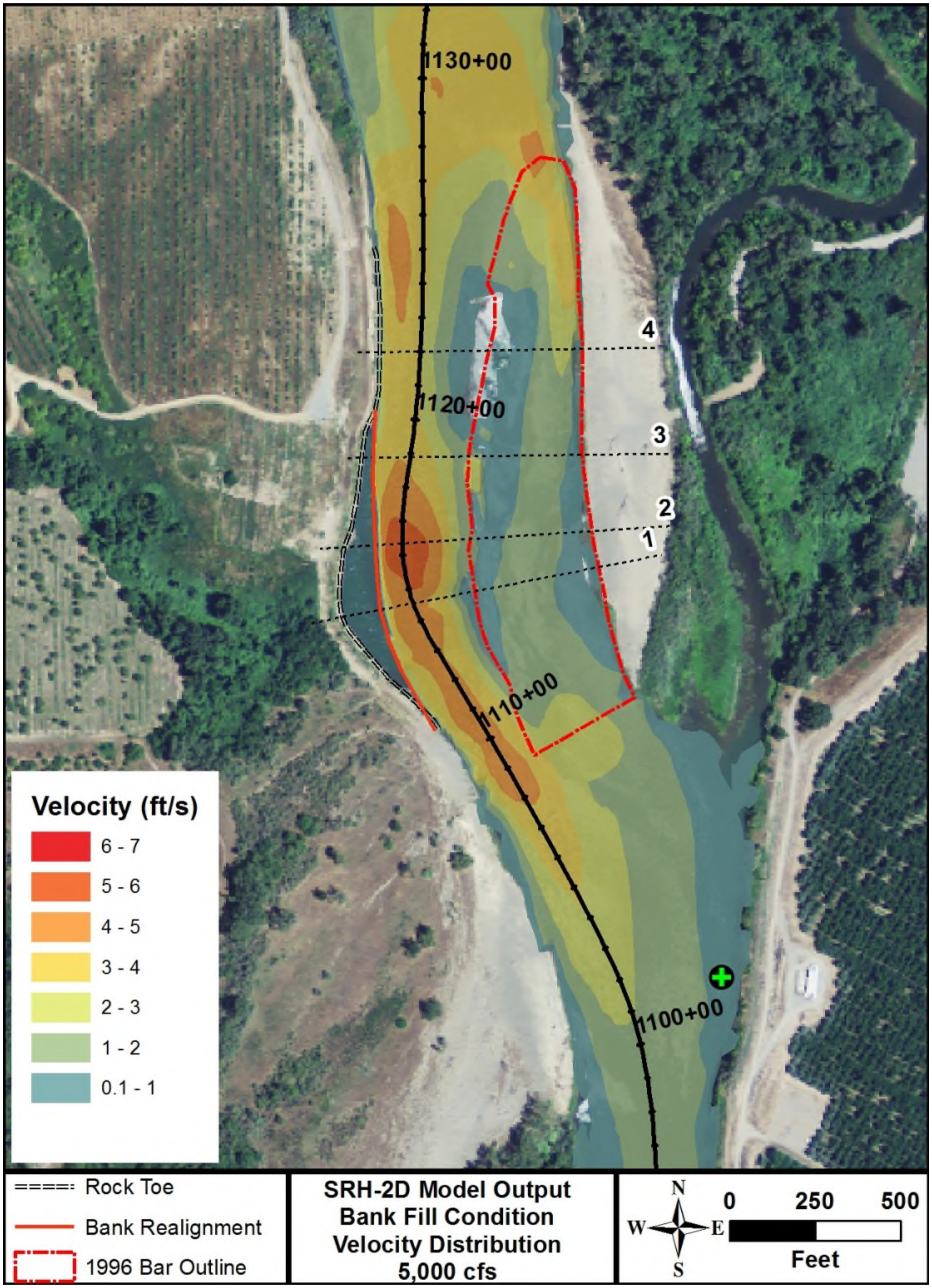


Figure 30. Predicted velocity distribution for the Bank Fill Conditions at 5,000 cfs.

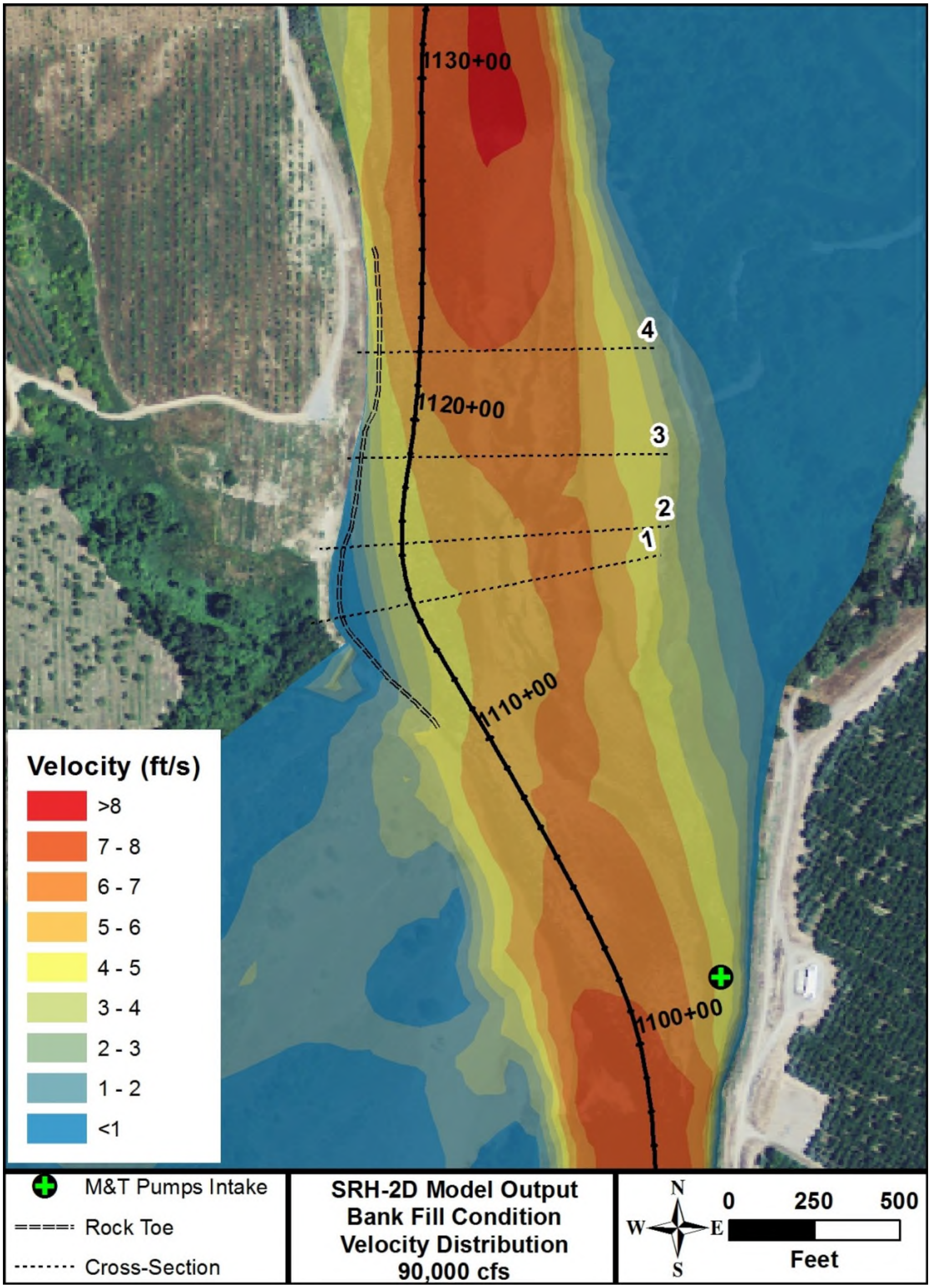


Figure 31. Predicted velocity distribution for the Bank Fill Conditions at 90,000 cfs.

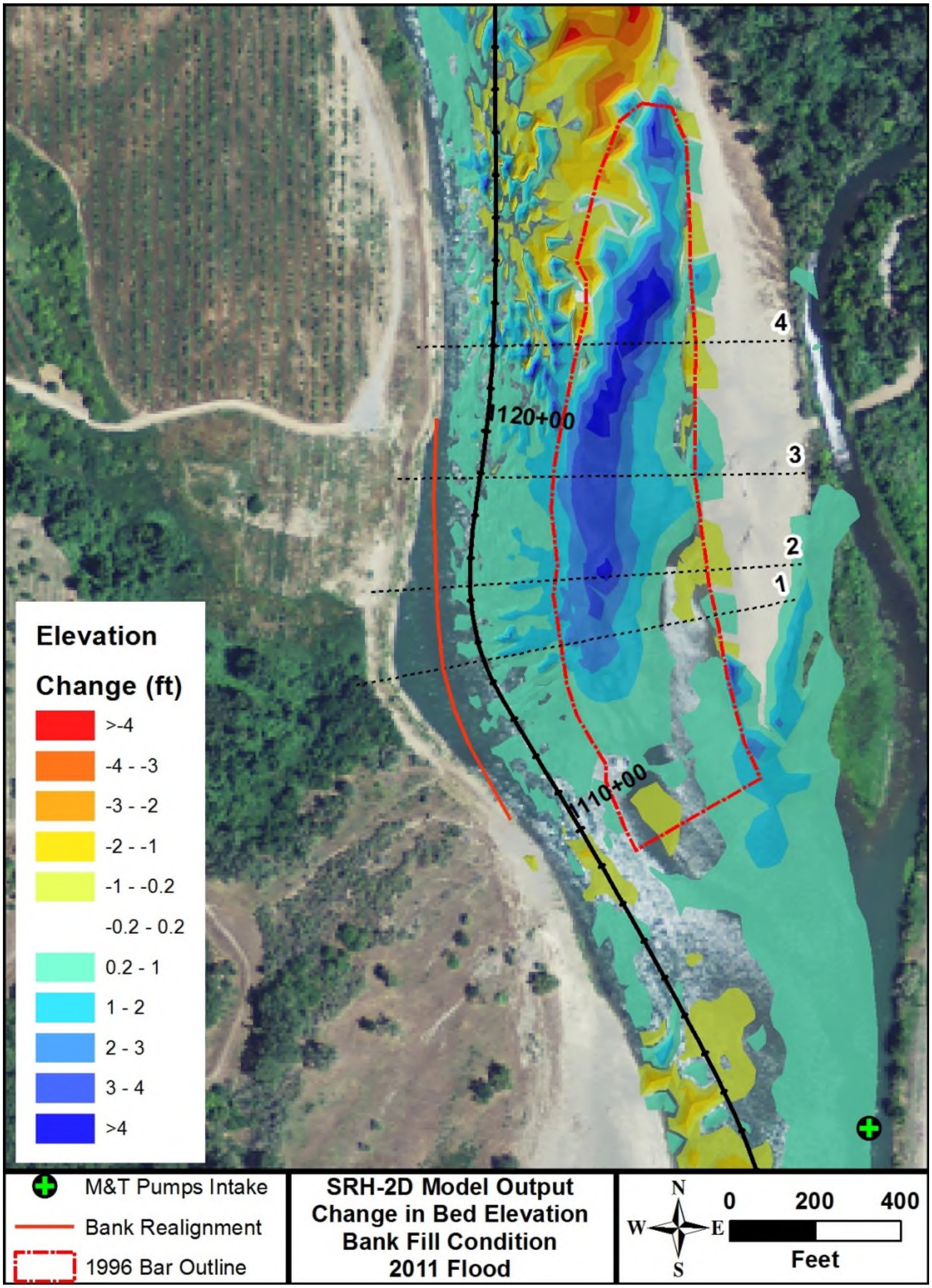


Figure 32. Predicted change in bed elevation for the Bank Fill Conditions over the duration of the 2011 peak flow hydrograph.

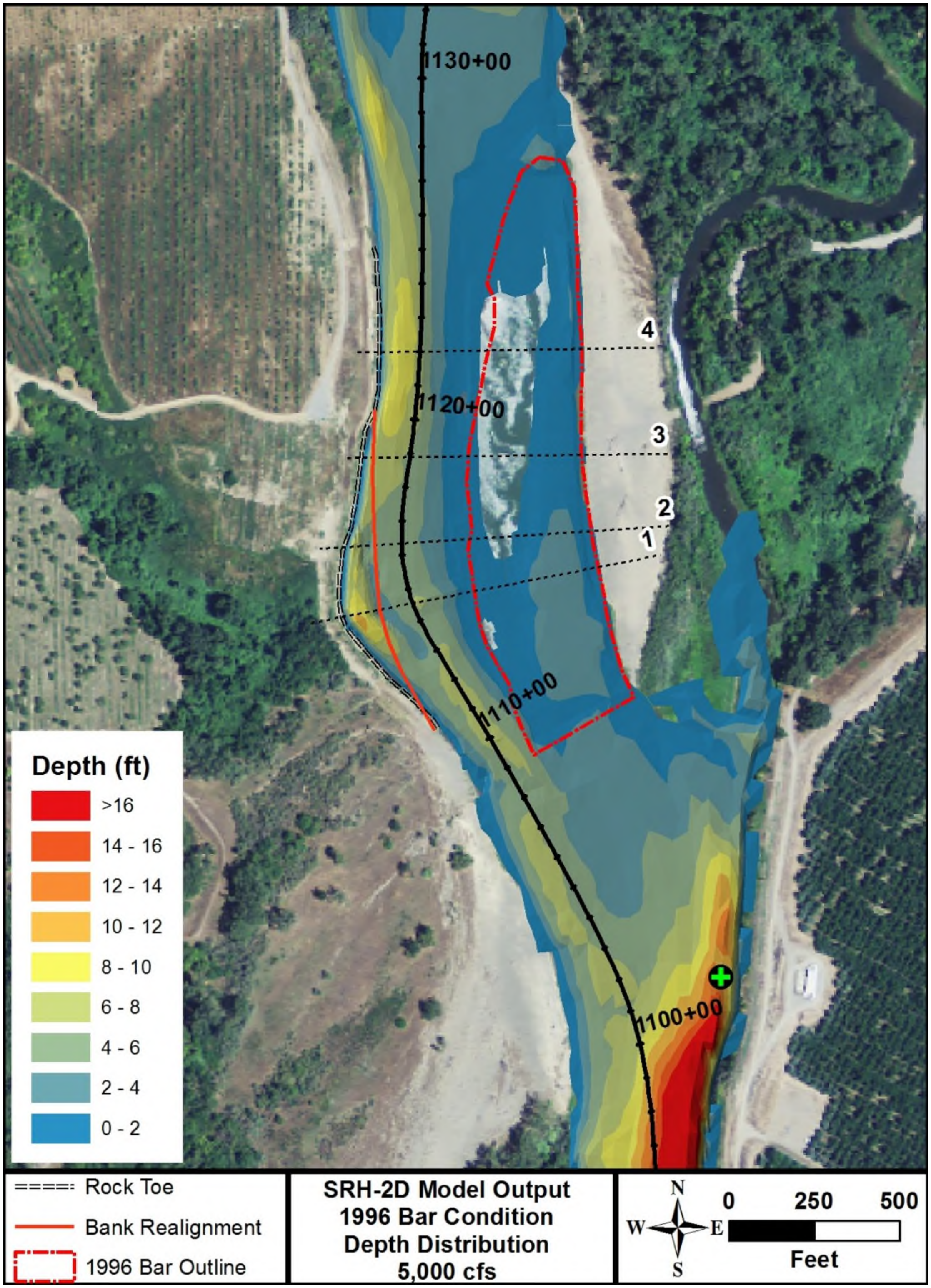


Figure 33. Predicted depth distribution for the 1996 Bar Conditions at 5,000 cfs.

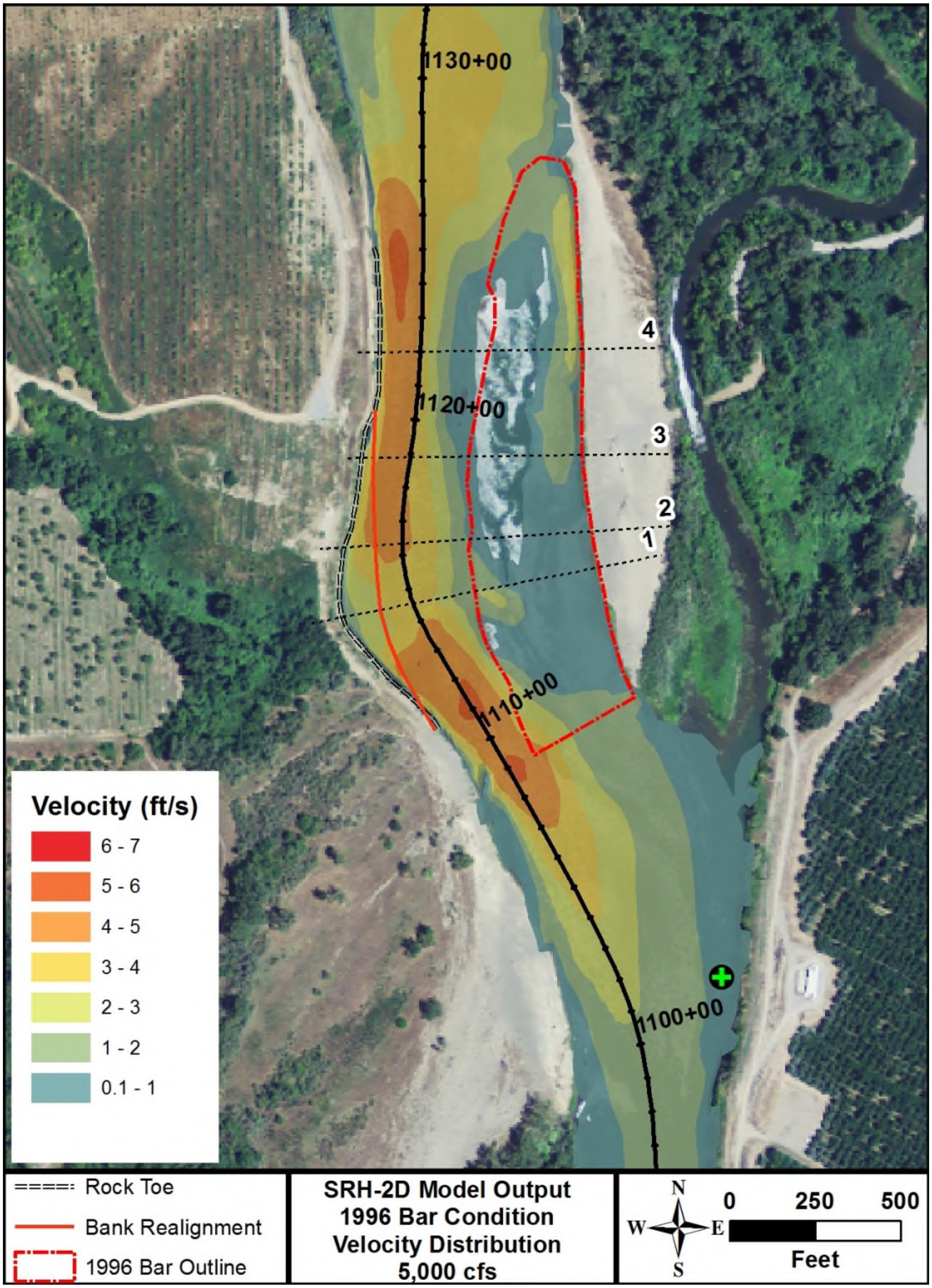


Figure 34. Predicted velocity distribution for the 1996 Bar Conditions at 5,000 cfs.

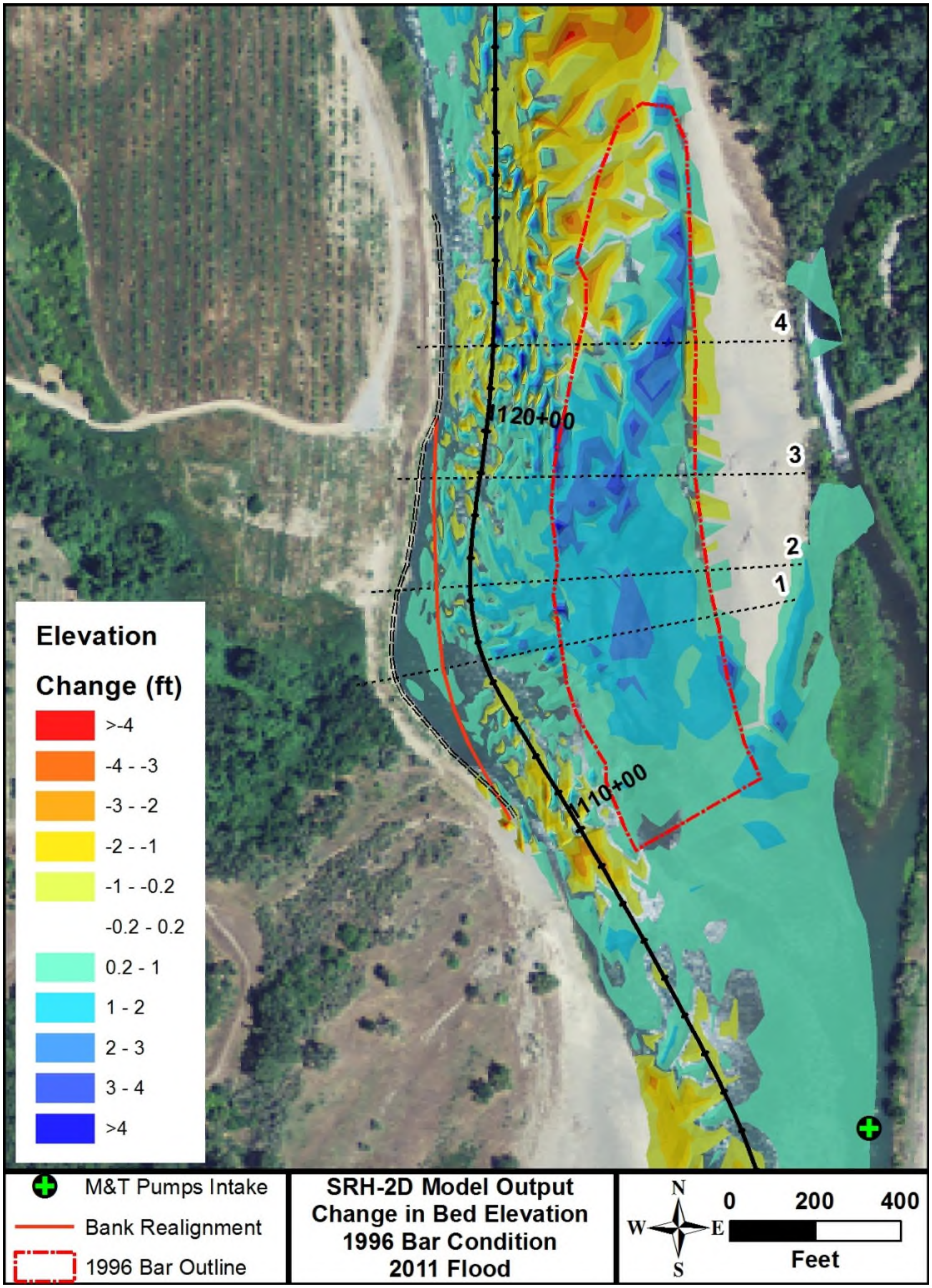


Figure 35. Predicted change in bed elevation for the Existing Conditions + 1996 Bar over the duration of the 2011 peak flow hydrograph.

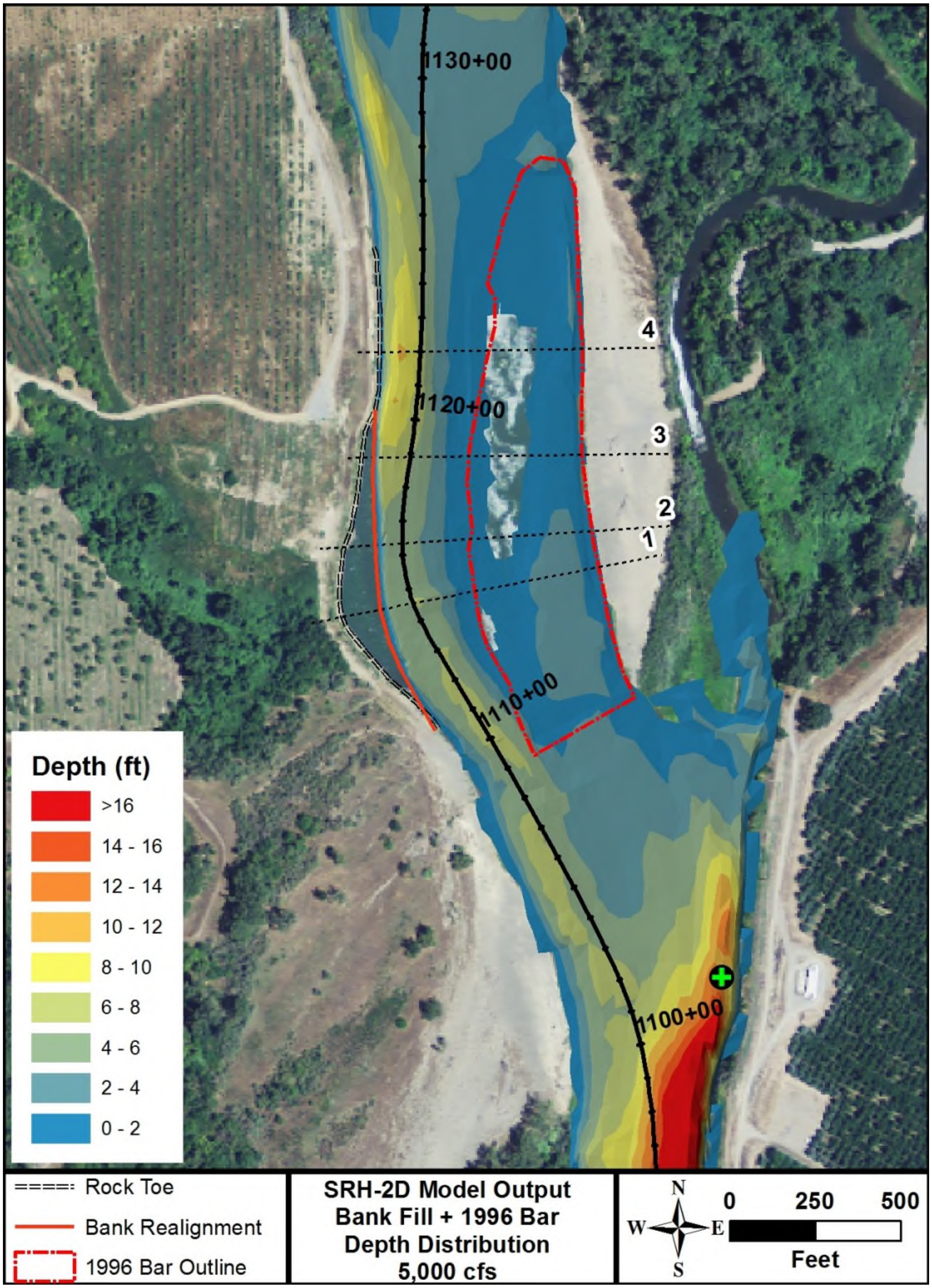


Figure 36. Predicted depth distribution for the Bank Fill + 1996 Bar Conditions at 5,000 cfs.

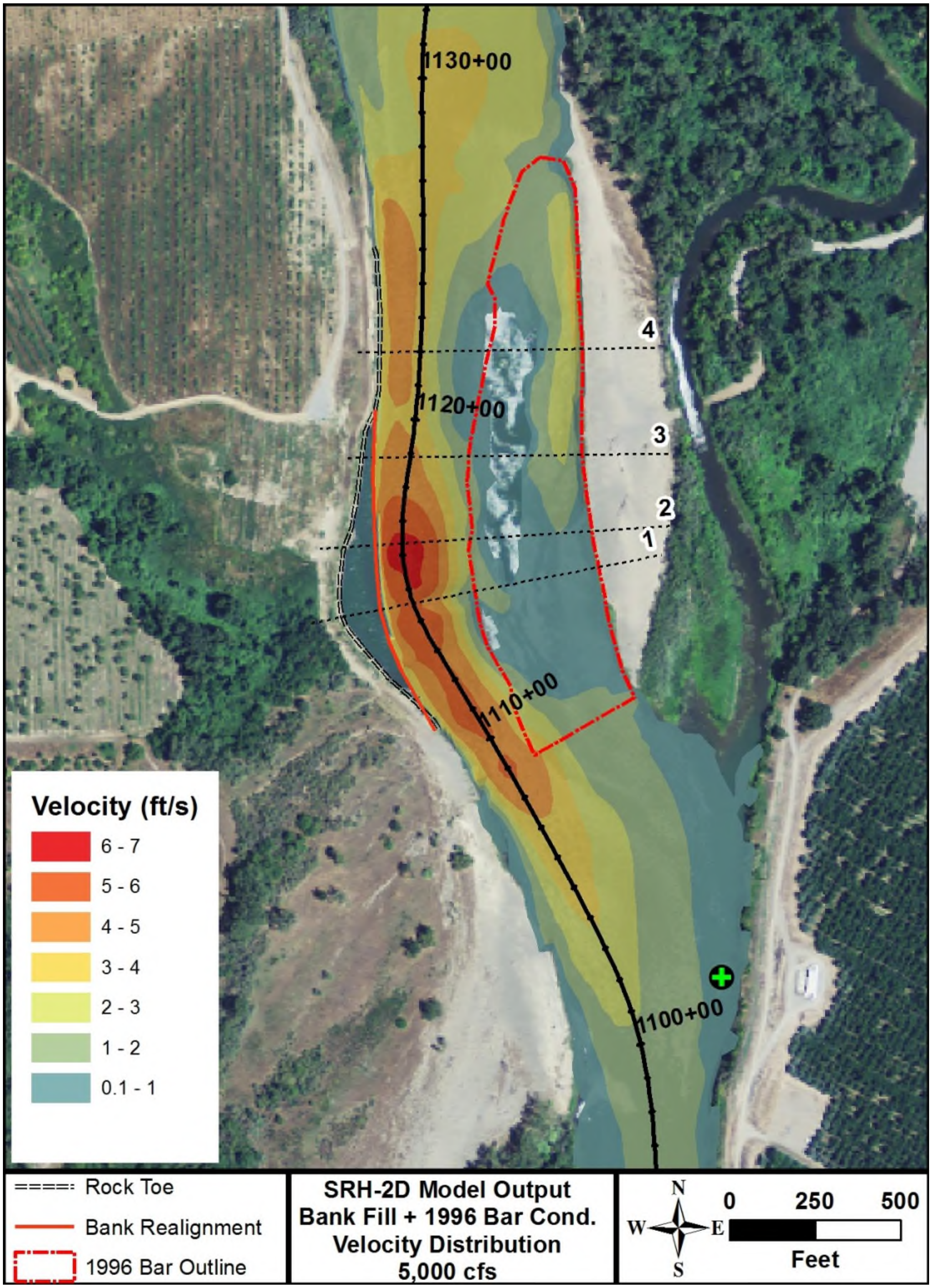


Figure 37. Predicted velocity distribution for the Bank Fill + 1996 Bar Conditions at 5,000 cfs.

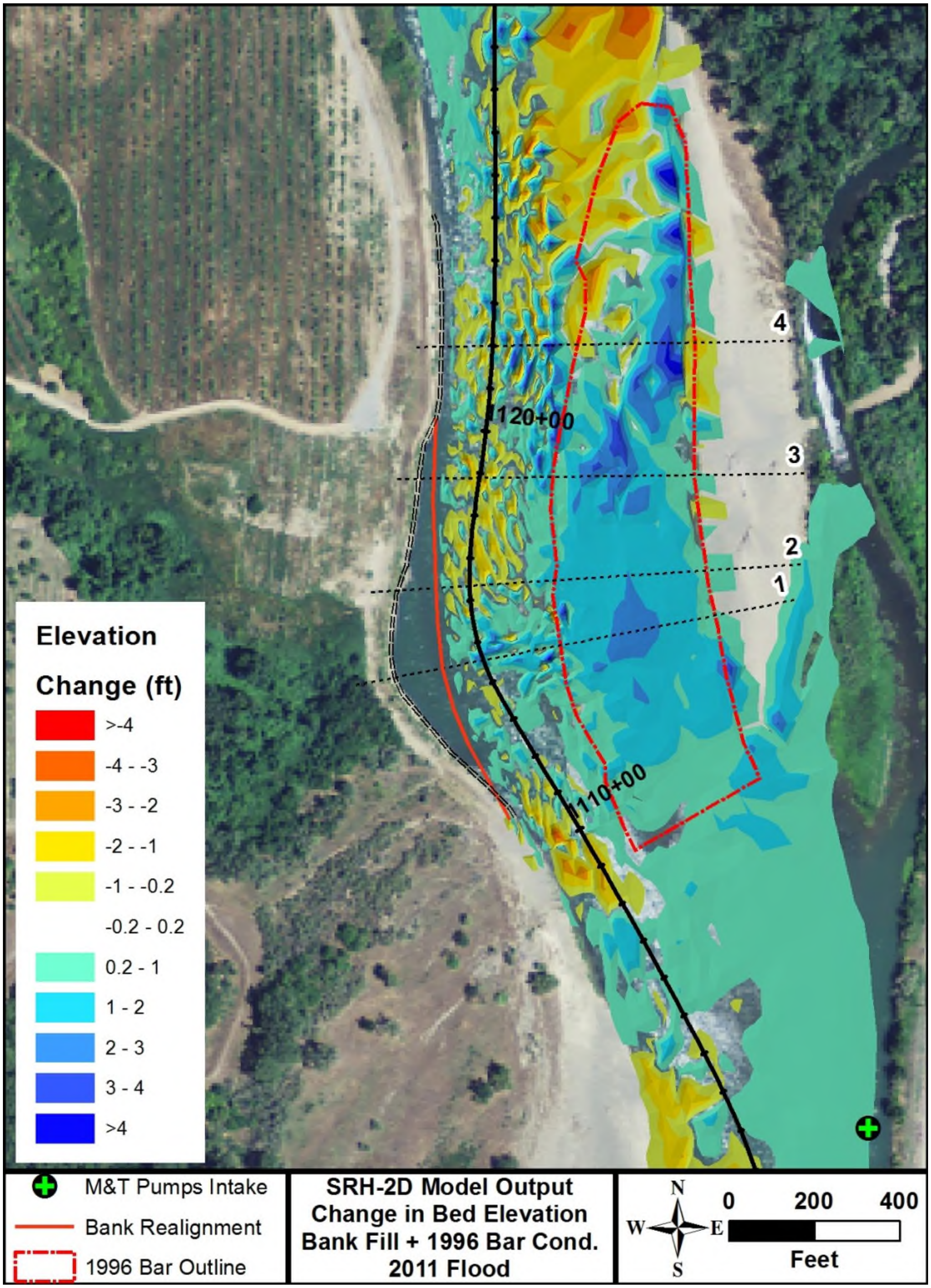


Figure 38. Predicted change in bed elevation for the Bank Fill + 1996 Bar condition over the duration of the 2011 peak flow hydrograph.