MEMORANDUM

TO: Jim Well, Ducks Unlimited
FROM: Mike Harvey, PhD, PG, Bob Mussetter, PhD, PE, Dai Thomas, PE
SUBJECT: **Two-dimensional Sediment-transport Modeling of the M&T/Llano Seco Pumps Reach**
Project No. US-CA-62-2
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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 migrating toward the fish-screened diversion. 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 CDFG fish-screen criteria. Periodic maintenance has been required to reduce the size of the gravel bar and prevent interference with the facility. In 2001 and 2007, 200,000 and 100,000 tons of material, respectively, were excavated from the gravel bar as a short-term solution to limit sedimentation impacts. Additionally in 2007, 1,500 feet of short-term, rock toe and brush bank protection was installed on the west side of the Sacramento River on the U.S. Fish and Wildlife Service's (USFWS) Capay Unit of the Sacramento River National Wildlife Refuge to prevent further channel meander upstream of the pump inlets.

Hydrographic and topographic surveys of the M&T/Llano Seco reach of the Sacramento River between River Mile (RM) 192 and RM 193.5 have been used to monitor geomorphic changes in the reach, including aggradation of the bed as well as bank erosion and lateral migration of the river. Surveys were conducted by Mussetter Engineering Inc. (MEI) in December 2005 and May 2006, and by Tetra Tech Inc. (Tt) (formerly MEI) in January 2010 and June 2011.

Analyses of changes in the reach were performed by comparing the bed elevations from the four surveys (Tetra Tech 2011a). The results of the analyses indicate that bed dynamics within the M&T/Llano Seco reach appears to be tied to the peak flow hydrology. With the exception of WY2004, the peak flows in the six years prior to 2005 were less than the bankfull discharge (~90,000 cfs) in the M&T/Llano Seco reach, and this sequence of flows appears to be responsible for aggradation in the channel. In WY2006, the peak flow was about 135,000 cfs and there was some degradation in the reach, especially in the vicinity of the fish screens and pump inlets. Between WY2006 and WY2010, the peak flows were again less than bankfull and significant aggradation (4 to 10 feet) occurred in the vicinity of the fish screens and pumps.

The peak flow in WY2011 was about 102,500 cfs and this appears to have caused degradation of up to 2 feet in the immediate vicinity of the fish screens subsequent to the 2010 survey (**Figure 2**). Approximately 400 feet downstream from the fish screen, the channel degraded by up to 13 feet along the left side of the channel which is revetted. In the area between the downstream end of the bar and the M&T pumps, the channel degraded approximately 2 feet. In the area that was dredged in 2007, the channel aggraded by up to 4 feet, and in the area between the upstream end of the bank-attached bar and River Road, the channel degraded by approximately 1 foot. In general, the locations of the greatest bed degradation occurred along the riprap bank downstream from the M&T pumps and adjacent to the toe rock installed along the right bank.

Based on the response of the system over the four surveys, it appears that cyclic behavior occurs within the M&T/Llano Seco reach with the less than bankfull flows generally causing aggradation and the higher-than-bankfull flows generally causing scour. The scour tends to occur along the banks, and it is hypothesized that this is due to the formation of a helical flow cell along the riprap that lines the east bank of the river in the vicinity of the fish screens and pump inlets because of flows that approach the riprap obliquely from upstream. Based on the results of dive surveys, a weaker helical flow cell appears to prevent deposition in the immediate vicinity of the fish screens and pump inlets at less than bankfull flows. At higher flows, the strength of the helical flow cell likely increases and this erodes previously deposited material in the general vicinity of the fish screens and pump inlets. If this assessment is correct, the cyclic behavior depends on the maintenance of the existing general alignment of the river. If the west bank is allowed to erode and continue to migrate westward, it is likely that the flow alignments would change and the helical flow cell would probably not be maintained in the vicinity of the fish screens and pump inlets, leading to burial with sediment.

Two-dimensional hydraulic and sediment-transport analyses (MEI 2005, 2006, 2008) and physical model studies (CSU, 2008 and 2011) were conducted to determine if spur dikes installed along the west bank of the river upstream of the M&T/Llano Seco pumping plant inlets and fish screens (RM 192.75) could recreate hydrodynamic conditions that will permit sustainable operation of the pumps for the next 40 years. The results of the analyses indicated that the proposed 9-dike configuration would prevent continued migration of the west bank and prevent downstream migration of the east bank-attached bar upstream of the fish screens and pumps. The estimated cost of the 9-dike scenario was $8.7 million based on 2006 costs (MEI, 2006). If the cyclic behavior hypothesis generated by the observations of the gravel bar is correct, the existing pump inlets and fish screens could potentially continue to meet the irrigation demands and fish-screen criteria provided that the current channel alignment is maintained. Downstream extension of the rock and brush toe may provide a viable alternative to either dikes or pump relocations.

1.1. Scope of Work

To evaluate the hypothesis that the observed cyclic behavior of aggradation and degradation in the reach is tied to the peak flow events and the current alignment of the river, an existing SRH-2D hydraulic model (Tetra Tech, 2011b) was used to evaluate the hydraulic and sediment-transport conditions in the reach. The analysis was conducted using the developmental, mobile-boundary version of the SRH-2D computer program developed by the Bureau of Reclamation (BOR).

The mobile-boundary option in SRH-2D was used to perform unsteady hydraulic and sediment-transport simulations for a range of bed-material sizes using low, intermediate and high peak flow hydrographs that were developed from the Hamilton City Gage records, and the model output was used to evaluate the aggradation and degradation patterns in the reach. Although the mobile boundary version of the software has not been publicly released at the time of this study, Tetra Tech has received permission from the BOR to use the model on a limited basis.
The work performed for this study included the following tasks:

1. The existing Phase III model (Tetra Tech, 2011b) developed using the 2010 channel geometry was used to perform sediment-transport and mobile-bed simulations using measured bed-material gradations and recorded flow hydrographs. Only one significant flood event (March-May 2011) occurred between the 2010 and 2011 surveys. The SRH-2D model, with 2010 geometry, was calibrated, to the extent possible, by performing a simulation of the 2011 flood event and comparing the model results at the end of the simulation with the 2011 bed geometry.
2. The calibrated sediment-transport model (baseline model) was used to test the hypothesis that the observed cyclic behavior of aggradation and degradation in the reach is tied to the peak flow events if the current bank alignment remains fixed. This hypothesis was tested by performing sediment-transport simulations with low, intermediate and high peak flow hydrographs. The model results were used to evaluate the potential for downstream migration of the bank-attached bar and the aggradation/degradation patterns in the vicinity of the M&T pumps and fish screens.
3. The 2-D mesh was modified in the area opposite the M&T pumps to represent erosion of the right bank and widening of the channel. This “eroded-bank” model was run with the 2011 flood hydrograph, and the results were used to evaluate the behavior of the gravel bar and aggradation/degradation patterns in the vicinity of the pump intakes. The current alignment of the right bank appears to create hydraulic conditions that are preventing further downstream migration of the bank-attached bar. It is hypothesized that, if the right bank erodes further and the channel widens, the associated change in sediment-transport conditions will cause the bank-attached bar to migrate downstream, interfering with the fish screen hydraulics and potentially burying the pump intake.
4. To further evaluate the effectiveness of the proposed dikes, the geometry of the baseline conditions model was modified to represent the 9-dike configuration. The model was run using representative high, medium and low peak flow hydrographs and the model results were used to evaluate the behavior of the gravel bar and aggradation/degradation patterns in the vicinity of the pump intake.

2. HYDROLOGY

Hydrologic input to the model consists of an unsteady flow time series developed using recorded discharges at the Sacramento River near Hamilton City gage (California Data Exchange Center Station HMC).

Three peak flow hydrographs were selected to represent the high, medium and low peak flow events (**Figure 3**). Each of the three representative hydrographs was simulated in the 2-D model under baseline conditions, the eroded bank condition and the 9-dike geometry.

The representative high peak flow hydrograph was developed using the WY2011 event which occurred on March 21, 2011, and had a reported peak discharge of 102,530 cfs. This hydrograph was used to calibrate the model by running the SRH-2D sediment-transport model (with 2010 geometry) over the 2011 flood event and comparing the model results at the end of the simulation with the 2011 bed geometry. The high-flow hydrograph had an initial discharge of 11,270 cfs, increased to the peak flow of 102,530 cfs at 341 hours (14.2 days), and then receded to 12,850 cfs at 888 hours (37 days) (Figure 3). This hydrograph had a total flow volume of approximately 2.9 million ac-ft.

The medium peak flow hydrograph was developed using measured flows in January and February, 2010 (the WY2010 hydrograph). 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 bathymetric survey of the M&T reach that was conducted between January 11 and 15, 2010, was carried out to develop the Phase III model (Tetra Tech, 2011b); this survey was conducted approximately 10 days before the WY2010 peak flow events. The medium flow hydrograph had an initial discharge of 11,600 cfs, increased to the first peak flow of 76,710 cfs at 183 hours (7.6 days), decreased to 13,780 cfs then increased to the second peak 73,700 cfs at 311 hours (13.0 days), and then receded to 9,778 cfs at 480 hours (20 days). The representative medium peak flow hydrograph has 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 (the WY2008 hydrograph). This hydrograph had a peak flow of 66,186 cfs that occurred on 26^{th} of January, 2008. The hydrograph had an initial discharge of 11,669 cfs at 121 hours (5.0 days), increased to the peak of 66,186 cfs at 20 hours and then receded to 11,130 cfs at 121 hours (20.0 days). The representative medium peak flow hydrograph has a total flow volume of approximately 287,000 ac-ft.

3. TWO-DIMENSIONAL SEDIMENT-TRANSPORT MODELING

The 2-D sediment transport model was developed from a modified version of the Phase III hydraulic model (Tetra Tech, 2011b) (**Figure 4**) that was modified to perform sediment-transport and mobile-bed simulations using measured bed-material gradations and the three representative flow hydrographs. This model is referred to as the baseline model in this report.

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). The previous SRH-2D modeling for this project was conducted using Version 2.0 a depth-averaged, finite volume, hydrodynamic model that computes water-surface elevations and horizontal velocity components for sub- super- and trans-critical, free-surface flow in 2-D flow fields. The developmental, mobile-boundary version was chosen for this project because it is one of the few available 2-D sediment-transport models capable of modeling the conditions observed in the M&T reach, and because the SRH-2D model meshes had been previously developed.

SRH-2D v3 computes scour and deposition in rivers and reservoirs 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 coarsening of the surface layer, as occurs in the M&T/Llano Seco reach.

To facilitate interpretation of the model results, a previously developed station line that represents the distance along the approximate centroid of the flow was used (Tetra Tech, 2011a). The downstream end (Sta 0+00) is located at the downstream boundary of the USACE Butte Basin 2-D model (**Table 1**). Along this station line, the up- and downstream ends of the baseline model are located at Sta 1183+00 and Sta 1028+00, respectively (**Figure 4**). The M&T pumping plant is located at Sta 1101+18 on the left (east) bank of the river.

Table 1. 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
1130+01	Upstream end of bank-attached bar
1147+86	River Road
1183+00	Upstream end of 2-D model

3.1. Model Development

3.1.1. Model Mesh

Separate model mesh files were developed for each of the three scenarios. The existing conditions Phase III model (Tetra Tech, 2011b) that was based on the 2010 geometry was used as the baseline model for comparison with the other scenarios. The baseline model mesh was then modified to represent erosion of the right bank (subsequently referred to as the "eroded bank" model) and the presence of the proposed nine spur dikes (subsequently referred to as the "9-dike" model).

3.1.2. Bed-material Gradations

Representative sediment gradations were applied to the model to define the distribution of bed and subsurface materials. A representative surface bed-material gradation curve was developed for the study reach based on the average of three measurements that were made during the December 2005 survey using the pebble count method (Wolman, 1954) (**Figure 5**). Pebble counts WC1 and WC2 were collected on top of the primary gravel bar and WC3 was collected on the lower elevation mid-channel bar adjacent to the primary bar. The representative gradation curve has median (D_{50}) and D_{84} sizes of 39 and 60 mm, respectively (**Figure 6**). A bulk sample of the subsurface material was also collected at the same location as WC1 after removing the coarser surface layer. The material in this sample is representative of the gradation of the sediment that was being transported at the time the bar was being formed, and it had a median size (D_{50}) of 9.5 mm and a D_{84} of 32.9 mm (Figure 6).

During the 2010 survey, the boundary of a sand deposit, which extended from the downstream end of the bank-attached bar to approximately opposite the M&T pumps, was mapped using a Real Time Kinematic (RTK) Global Positioning System (GPS) (**Figure 7**). The margin of the sediment deposit was located by wading in the river and probing the bed of the channel with a spade to determine the approximate depth and extent of the sand deposit. No sampling of the sand deposit was conducted to determine the gradation.

The representative bed-material gradation shown in Figure 6 (D_{50}~39mm) was applied to the majority of the channel bed portion of the model. The surface layer thickness was assumed to be constant and the same as the D_{90} of the representative gradation (approximately 0.2 feet) to represent the coarse cobble surface layer. The measured subsurface gradation (D_{50}= 9.5 mm) was applied to the subsurface layer of the model. In the area at the downstream end of the bank-attached bar and at the mouth of Chico Creek, the surface of the channel was represented using a sand-sized material with D_{50} of 1 mm and estimated thickness of 1.5 feet. A third material type was used to represent the nonerodible materials, including the riprap along the east and west banks and the overbank areas. The SRH-2D model will simulate deposition on the nonerodible material type and subsequent removal of the deposited material, where appropriate, but does not allow erosion of the base material.

3.1.3. Bed-material Transport Capacity Relationship and Upstream Sediment Supply

The Parker (1990) surface-based bed-load equation was selected for use in this study because it was developed for conditions that are similar to those found in the study reach, and because it has been successfully applied in previous studies in this reach (MEI, 2005 and 2006).

The sediment supply to the upstream end of the model is either calculated by the model using a user-defined input sediment-rating curve, a sediment-transport hydrograph, or the “capacity” option in the model that estimates the supply based on the transport capacity at the upstream boundary. No sediment-transport measurements were available within the project reach to directly develop a sediment-rating curve or a sediment inflow hydrograph. Initial testing of the capacity option indicated that this method overestimated the inflowing sediment load by approximately two orders of magnitude causing physically unrealistic deposition. As a result, a sediment-transport versus discharge rating curve was developed using the methodology described in the next paragraph. The rating curve was developed for eight different bed-material size fractions that ranged from sand- to gravel-sized material (0.425 to 90 mm) for a range of flows from 10,000 to 110,000 cfs. A significant effort was made to develop the rating curve to ensure that the model correctly predicts the observed magnitude and patterns of bed elevations change in the reach compared to the 2011 conditions.

To develop the inflowing sediment-rating curves, the baseline sediment-transport model was run using the parameters described above for a series of 11 steady-state discharges from 10,000 to 110,000 cfs in increments of 10,000 cfs. The initial inflowing sediment loads at the upstream boundary were calculated by the SRH-2D model using the “capacity” option. The initial sediment-rating curve was then modified by iteratively reducing the inflowing sediment loads until the predicted bed elevation changes matched, as closely as possible, the measured bed elevation changes that occurred between 2010 and 2011. In addition, the sediment-transport rates for each discharge and size fraction were compared to the predicted values 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. Where necessary, the inflowing sediment-transport rates were adjusted so that the shape of the rating curve had similar characteristics to the rating curves predicted by the model in the vicinity of the upstream end of the bank-attached bar (**Figures 8 and 9**). The predicted transport rates show a hysteresis effect with higher transport on the rising than on the falling limb of the hydrograph. This effect has been documented in many other rivers, and is believed to be realistic. The inflow sediment rating curve is consistent with the average of the rising and falling limbs from the model results at the two cross sections.

The measured changes in bed elevation between the 2010 and 2011 surveys indicate a general degradational trend throughout the M&T reach (Figure 2) and the following specific observations can be made from the data:

1. In the area between River Road and the upstream end of the bank-attached bar, bed degradation was generally about 1 foot (Cross Sections 4 through 6). Aggradation of approximately 1 foot also occurred near the right side of the main channel in the area between Cross Sections 4 and 5.
2. In the area of the gravel bar that was dredged in 2007 (Cross Section 7), aggradation in the range of 1 to 4 feet occurred.
3. In the area between the downstream end of the bank-attached bar to opposite the M&T pumps, 1 to 4.5 feet of degradation occurred. Approximately 1.5 feet of degradation occurred at the pump intake, and up to 4 feet of degradation occurred near the center of the channel and toward the left bank at Cross Section 9 (near the pump intake). The mapped sand deposit at the downstream end of the bank-attached bar degraded by approximately 1.5 feet.
4. Up to 13 feet of degradation occurred along the left bank just downstream from the M&T pump intake.
5. Degradation of approximately 1 foot occurred towards the center of the channel in the vicinity of the City of Chico outfall.
6. In the area downstream from the City of Chico outfall (Cross Section 10), the main channel was mostly degradational, with magnitudes ranging from 0.5 to 7 feet and averaging about 1 foot.

In general, the most significant amounts of degradation occurred along the riprapped banks on the left (east) side of the main channel. For example, up to 5 feet of degradation occurred along the base of the rock toe just upstream from Cross Section 7 and up to 3 feet of degradation occurred immediately upstream from Cross Section 8. The largest amount of degradation occurred along the left bank from just downstream from the M&T pumps to the end of the riprap, which is located where the left bank projects into the river (approximately 600 feet downstream from the City of Chico outfall).

3.2. Model Results

The 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 9)**.** In addition, the cross-sectional geometry at the end of the simulation was compared to the 2010 and 2011 geometry at Cross Section 7 (approximate longitudinal center of the primary gravel bar) and Cross Section 9 (near M&T Pump intake) (**Figures 10 and 11**). The results of the 2011 flood simulation indicate the following:

1. In the area between River Road and the upstream end of the of the bank-attached bar (Cross Sections 4 through 6), the predicted bed degradation ranges from 1 to 4 feet, compared to the measured change of approximately 1 foot in this area. The predicted degradation is generally located in the main flow path which extends from near the left bank to about the center of the channel. Approximately 1 foot of aggradation occurred along the right side of the channel compared to the measured scour of approximately 1 foot.
2. The model predicts aggradation of 1 to 4 feet over the gravel bar that was dredged in 2007. The amount and the extents of the aggradation match the measured changes reasonably well over the area of the dredged bar, but the model does not predict the degradation that occurred along the right bank (Figure 10).
3. Between the downstream end of the bank-attached bar and opposite the M&T pumps, the model predicts 1 foot to 3.5 feet of degradation near the center of the channel, which is very similar to the measured values (Figure 11). However, the model does not predict degradation adjacent to the banks because it does not have the ability to simulate the hypothesized three-dimensional (3-D) helical flow.
4. The model predicts between 0.5 and 1.5 feet of aggradation across the entire channel in the vicinity of the City of Chico outfall (Cross Section 10), whereas, the 2011 survey indicated approximately 5 feet of degradation along the left bank and approximately 0.5 feet of degradation across the rest of the main channel (Figure 10).
5. The model results indicate that the main channel is generally aggradational (on the order of 1 foot) in the area downstream from the City of Chico outfall (Cross Section 10), compared to the measured degradation of approximately 1 foot.

The variation in sediment-transport capacity along the reach was quantified by calculating the total volume of sediment passing each of the 11 cross sections over the duration of the hydrograph (**Figure 12**). (Note: Cross Section 0 is located at the upstream model boundary, Cross Section 9 is located at the M&T pumps and Cross Section 10 is located at the City of Chico outfall.) The results indicate that the transport capacity in the vicinity of Cross Section 8, which is located at the downstream end of the bar and upstream of the M&T pump, is very low compared to both the up- and downstream reaches. These conditions strongly favor continued development of the bar and deposition in the vicinity of the M&T pumps.

In summary, the 2-D model predicts the magnitude and patterns of aggradation and degradation reasonably well compared to the measured changes that occurred between 2010 and 2011 in the reach between River Road and the City of Chico outfall. The model does not, however, predict the significant degradation that occurred along the left bank between the M&T pumps and the bank spur located downstream from the City of Chico outfall. This model limitation occurs because the 2-D model does not have the capability to accurately represent the most likely 3-D flow fields associated with the formation of the helical cells.

To further assess the behavior of the model, the predicted median size of the surface bed material in the area that was dredged in 2001 and 2007 over the duration of the hydrograph was evaluated. The model results show that the median (D_{50}) size decreases from 37 mm at the start of the simulation to approximately 7 mm at the peak of the hydrograph (**Figure 13**; see Figure 5 for location). By the end of the simulation, the median size coarsens back to about 18mm at this location, and it will continue to coarsen over time as the finer material is transported from the reach. This behavior appears to occur because the supply of sand and fine gravel is very low during the beginning of the rising limb of the hydrograph, but increases significantly during the peak period, and the relatively low-energy dredged area becomes depositional. As the hydrograph recedes, the upstream supply of sand and fine gravel diminished, but there is still sufficient energy over the dredged area to mobilize the deposited material; thus, it is removed and the bed tends to coarsen back toward the initial condition. In addition, the model indicates that at the end of the simulation, the bed material at the M&T pumps is comprised of sand-sized material with median size of 1.4 mm, which corresponds to the observations of the bed material during the historical dive surveys.

Based on the comparison of the predicted and measured changes in bed elevation during the 2011 flood, the comparison of the sediment-transport rating curves and the changes in bed material size over the period of the hydrograph, the model appears to be reasonably well calibrated to the 2011 flood event, and the developed inflowing rating curve should be applicable to the other flow and channel scenarios.

3.2.1. Baseline Model Results at 90,000 cfs (Bankfull Discharge)

To compare the hydraulic conditions under the different scenarios, the baseline model was run at a steady state discharge of 90,000 cfs which corresponds to the bankfull conditions in the reach. The model results indicate that the maximum main channel velocities range from 6 to 9 fps and maximum channel depths range from 17 to 42 feet (**Figures 14 and 15**). (The maximum flow depth of 42 feet occurs near the M&T pump intakes.) The highest velocities occur along the left side of the channel between River Road (Cross Section 4) and midway along the bank-attached bar (Cross Section 7), and the lowest velocities of approximately 5.5 fps occur in the expansion zone near the downstream end of the gravel bar, which creates the conditions for sediment deposition and further bar development. The velocity over the gravel bar at 90,000 cfs is in the range of 4 to 5 fps, and the flow depth is approximately 8 feet. The primary flow path upstream from the bar is directed slightly towards the left (east) bank (**Figure 16**), which may be causing the hypothesized helical flow and associated scour along the bank in the vicinity of the M&T pumps. A flow expansion occurs at the head of the bar, and the majority of flow is orientated mostly in line with the bar. Velocities in the deep area adjacent to the M&T pump intakes are approximately 6 fps at 90,000 cfs.

3.2.2. Evaluation of the Low and Medium Peak Flood Events

The baseline sediment transport model was also run for the representative low (WY2008 Q_{peak}=66,186 cfs) and medium (WY2010, Q_{peak}=76,705 cfs) flood hydrographs (Figure 3).

Low Peak Flood Simulation

The simulation of the low peak flow hydrograph (WY2008) indicates that the largest area of predicted bed elevation change occurs between the downstream end of the bank-attached bar (Cross Section 8) and the City of Chico outfall (Cross Section 10) (**Figure 17**). At the downstream end of the bar, the channel degrades by approximately 1.3 feet, indicating erosion of the previously mapped sand deposit. Approximately 23,900 yd^3 of sediment is eroded from this area and approximately 6,800 yd^3 of sediment is deposited in the area between Sta 1101+70 to Sta 1088+80 (a distance of approximately 1,370 feet); therefore, this area is net degradational. It is interesting to note that the sand deposit is transported from the center of the channel towards the left bank, and to the vicinity of the M&T pump intakes. Due to the inability of the SRH-2D model to predict the observed erosion along left bank that was observed during the 2011 flood simulation, it is uncertain if the model over-estimates the amount of deposition in the vicinity of the M&T pumps, or if the helical flow cell would form at this discharge with sufficient strength to prevent the predicted deposition. However, the predicted deposition pattern matches the measured patterns that occurred during previous low-flow events (Tetra Tech, 2011a). At the end of the simulation, the material deposited in the vicinity of the M&T pump intakes is primarily sand to small gravel. The sand deposition at the downstream end of the bar would likely continue during long low-flow periods, then be eroded during relatively low peak flow events and transported downstream into the vicinity of the M&T pumps. At the end of the simulation, the bed material in the vicinity of M&T pumps varies from 36mm adjacent to the bank, indicating that very little sediment transport has occurred in this area, to 1.5 mm towards the center of the channel. This sand-sized material was likely transported from the sand-deposit located at the downstream end of the bar.

Medium Peak Flood Simulation

The results of the medium peak flow hydrograph simulation (WY2010) are similar to the results from the low-flow peak hydrograph simulation, except that there is more deposition in the vicinity of the M&T pumps and in the area between the pumps and Sta 1080+00 (**Figure 18**). In addition, additional erosion and deposition occurs at the upstream end of the reach in the vicinity of River Road (Sta 1150+00).

At the downstream end of the bar, the channel degrades by approximately 1.5 feet, indicating erosion of the previously mapped sand deposit. Over the duration of the hydrograph, approximately 11,970 yd^3 of sediment is eroded from the downstream end of the sand bar and approximately 31,100 yd^3 of sediment is deposited between Sta 1101+70 and Sta 1088+80; thus, this area is net aggradational during this event. Also, similar to the low peak flow hydrograph simulation, the sand deposit is transported from the center of the channel towards the left bank and into the vicinity of the M&T pump intakes where about 0.40 feet of deposition of primarily medium sand (D_{50}=1.5 mm) occurs.

The repeat field surveys indicate that the area in the vicinity of the bar is net aggradational under the medium-sized flood hydrographs. The results of the modeling indicate that the reach in the vicinity of the M&T pumps is net aggradational, however, no deposition occurs between River Road and the downstream end of the bar.

3.3. Evaluation of the Eroded Right Bank Model

A significant concern of the Steering Committee is the continued erosion of the right bank opposite the M&T pumps. The concern is that if the right bank erodes further and the channel widens, the associated change in sediment-transport conditions will cause the bank-attached bar to migrate downstream, interfering with the hydraulics at the fish screen, and potentially burying the pump intake. To test this hypothesis, the 2-D model was modified in the area opposite the M&T pumps to represent erosion of the right bank and widening of the channel. This “eroded-bank” model was run with the 2011 flood hydrograph, and the results were used to evaluate the behavior of the gravel bar and aggradation/degradation patterns in the vicinity of the pump intakes.

3.3.1. Model Modifications

The eroded right bank model was developed by lowering the elevations of the mesh to represent approximately 250 feet of erosion along the right bank opposite the M&T pumps (**Figure 19**). The alignment of the eroded bank was based on bank erosion characteristics previously observed in the reach (MEI, 2005) and qualitative estimates of the most likely evolution of the bank if the erosion occurred. The bank erosion extends from the downstream end of the toe revetment to 430 feet downstream from the City of Chico outfall (Cross Section 10), a distance of 2,200 feet. It is assumed that the west bank toe revetment would remain fixed in place even if the right bank were to erode.

3.3.2. Model Results

The model output for the 2011 hydrograph simulation predicts that the area opposite the M&T pumps becomes significantly more depositional under eroded bank conditions (**Figure 20**) than under the baseline conditions (Figure 9). Under baseline conditions, the main channel was mostly erosional from the downstream end of the bank-attached bar (Cross Section 8) to just upstream of the City of Chico outfall (Cross Section 10). Under the eroded bank scenario, the main channel is mostly depositional, with a representative deposition depth of 0.5 feet, from the upstream end of the bank-attached bar (Sta 1125+00) to approximately 700 feet downstream from Cross Section 10, a distance of approximately one river mile.

The largest amounts of deposition of up to 5 feet occur in the area of the dredged bar. At the downstream end of the bar, the predicted deposition is about 1.5 feet. The deposition across the channel opposite the M&T pumps ranges from negligible to 0.7 feet, with a representative depth of 0.5 feet, in contrast to the measured degradation of 1 to 3 feet in this area under current conditions.

The sediment load at Cross Sections 9 and 10 is significantly lower than under baseline conditions due to the increase in channel width and the associated decrease in sediment-transport capacity (**Figure 21**), but the load at the downstream end of the bank-attached bar is similar to baseline conditions, with a total load of about 20,700 yd^3. The predicted sediment load from Cross Sections 1 to 7 is higher than under baseline conditions. The increase in channel width at the eroded bank steepens the energy gradient in the area upstream from the bank erosion, which in turn increases the upstream main channel velocity and sediment-transport capacity. The combination of increased in sediment-transport capacity upstream of M&T pumps and decrease in sediment-transport capacity downstream of the pumps will strongly favor deposition from the downstream end of the bank-attached bar to well downstream of the M&T pumps, effectively allowing the bar to continue to migrate downstream through the site.

At a steady-state discharge of 90,000 cfs, the model results indicate that the maximum main channel velocities decrease in the vicinity of the eroded bank compared to baseline conditions (**Figure 22**). For example, at Cross Section 8, the velocity is approximately 5 fps in the eroded bank model and approximately 6 fps under baseline conditions. Similarly, at Cross Section 9, the velocity is approximately 5 fps in the eroded bank model and approximately 7 fps under baseline conditions. At the M&T pump intakes, the velocity decreases from 4.8 fps under baseline conditions to 2.8 fps under the eroded bank scenario. In the area upstream from the eroded bank, the velocities increase slightly due to the decrease in water-surface elevation. At Cross Section 7, the velocities in the vicinity of the excavated bar are approximately 6.7 fps in the eroded bank model compared to 6.4 fps under baseline conditions, and at Cross Section 6, the velocities at the center of the channel are approximately 8.2 fps in the eroded bank model compared to 8.0 fps under baseline conditions. In the eroded bank model, the water-surface elevations at Cross Section 9 are the same as under baseline conditions, and approximately 0.3 feet lower between Cross Section 8 and at the upstream end of the model.

3.4. Evaluation of the 9-dike Model

To further evaluate the effectiveness of the proposed dikes, the geometry of the baseline model was modified to represent the 9-dike configuration (MEI, 2005). The model was run for the high, medium and low peak flow hydrographs and the model results were used to evaluate the behavior of the gravel bar and aggradation/degradation patterns in the vicinity of the pump intakes.

3.4.1. Model Modifications

The 9-dike model was developed by incorporating the 9-dike design from the Phase II analysis (MEI, 2005) into the baseline conditions model. An *n*-value of 0.20 was applied to the dikes to reflect the roughness and additional turbulence losses associated with the dikes. The other roughness values and model parameters in the 9-dike model remained the same as the baseline model for consistency. The sand deposit at the downstream end of the bank-attached bar was not included in the model because it was assumed that the additional turbulence caused by the dikes would quickly erode the deposit.

3.4.2. Model Results

The model results for the high peak flow hydrograph indicate that erosion along the upstream end of the gravel bar will increase significantly compare to baseline conditions due to the effective narrowing of the channel caused by the dikes (**Figure 23**). The baseline conditions model for this hydrograph predicted approximately 1.3 feet of erosion along the right (west) edge of the gravel bar and up to 1.0 feet of erosion on the upper surface of the bar. Under the 9-dike configuration, up to 5 feet of erosion occurs along the majority of the west edge of the gravel bar. In addition, from the downstream end of the gravel bar to downstream of the M&T pump intakes, a band of erosion occurs along the center of the main channel. In effect, a continuous band of erosion occurs from the upstream end of the bank-attached bar to downstream of the M&T pumps intakes, which should prevent the bar from expanding and migrating in the downstream direction towards the M&T pumps.

At the downstream end of the bar along the left side (east), the 9-dike model predicts up to 3 feet of aggradation, which is very similar to baseline conditions. At the M&T pump intakes, the model predicts little change in bed elevation under the 9-dike and baseline scenarios. However, it is important to remember that the model does not predict the scour along the riprap caused by the helical flow, as discussed above.

At the end of the simulation, the bed material in the vicinity of M&T pumps has a median size of about 1.5 mm, similar to baseline conditions. The results indicate sediment-transport loads at Cross Sections 6 through 9 are significantly higher than under baseline conditions due to the decrease in channel width and the associated increase in sediment-transport capacity caused by the dikes (**Figure 24**). The highest sediment loads of approximately 46,000 yd^3 occur midway along the bank-attached bar (Cross Section 7). At the downstream end of the bank-attached bar (Cross Section 9), the sediment load is about 36,500 yd^3 under the 9-dike condition compared to 21,450 yd^3 under the baseline conditions. At the M&T pumps, the sediment load is about 38,440 yd^3 under the 9-dike condition compared to 28,000 yd^3 under the baseline conditions.

At the steady-state discharge of 90,000 cfs, the model results indicate that the velocities over the bank-attached bar generally increase by 1.5 fps across the bar and by up to 3 fps at the upstream end of the bar over baseline conditions (**Figure 25**). At the downstream end of the bar and at the M&T pumps, the velocities increase by approximately 1.0 and 0.5 fps, respectively, compared to baseline conditions. Similar to baseline conditions, the flow orientation at the downstream end of the bar is directed slightly towards the left bank (east bank; **Figure 26**), which strongly suggests that the hypothesized helical flow and associated scour will occur along the left bank in the vicinity of the M&T pumps.

Under the 9-dike condition at 90,000 cfs, the water-surface elevations will increase over baseline conditions by a maximum of 0.5 feet at the most upstream dike (**Figure 27**). At River Road, the predicted increase in water-surface elevation is approximately 0.4 feet. The changes in water-surface elevation between the middle of the bank-attached bar and the downstream end of the model are relatively small.

The model results for the medium peak flow hydrograph indicate that very little change occurs throughout the reach, with some localized scour of up to 2-feet at the upstream end of the bank-attached bar and just downstream of the M&T pumps (Sta 1098+00, (**Figure 28**). Under the baseline conditions, the model predicts that the area in the vicinity of the M&T pumps is aggradational over the duration of the medium peak flow hydrograph. However, under the 9-dike condition, the model predicts no change in bed elevation in the vicinity of the M&T pumps.

The 9-dike model results for the low peak flow hydrograph indicates insignificant change in bed elevation over the duration of the hydrograph.

4. SUMMARY AND CONCLUSIONS

4.1. Summary

The specific objectives of this study were to:

1. Evaluate the hypothesis that the observed cyclic behavior of aggradation and degradation in the reach is tied to the peak flow events if the current bank alignment remains fixed.

2. Investigate the impacts, if any, of erosion of the right bank opposite the M&T pumps.
3. Further evaluate the effectiveness of the proposed 9-dike configuration.

To address the objectives of the investigation, the following 2-D models were developed for each specific objective:

1. The previously developed and calibrated Phase III model (Tetra Tech, 2011b) that was developed using the 2010 geometry was modified to perform sediment-transport simulations. Representative bed and surface material gradations were applied to the model and an inflow sediment-rating curve was developed. The sediment transport model was validated by simulating the 2011 peak flow hydrograph (Qpeak=102,538 cfs) and comparing the predicted bed geometry at the end of the simulation with the 2011 bed geometry. This baseline model was also run for low (Qpeak=66,186 cfs), medium (Qpeak=76,705 cfs) and high peak flow (Qpeak=102,538 cfs) hydrographs, and the model output 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 M&T pump.
2. The baseline model was modified to represent approximately 250 feet of lateral erosion along the right bank opposite the M&T pumps. This "eroded-bank" model was run with the 2011 flood hydrograph, and the results were used to evaluate the behavior of the gravel bar and aggradation/degradation patterns in the vicinity of the pump intake.
3. The baseline model was modified to represent the 9-dike configuration. The model was run using hydrographs with low, intermediate and high peak flows, and the model results were used to evaluate the behavior of the gravel bar and aggradation/degradation patterns in the vicinity of the pump intake.

4.2. Conclusions

This investigation led to the following conclusions:

1. The sediment-transport model was calibrated to the 2011 peak flow event. Results from this simulation indicate 1 to 4 feet of degradation in the area between River Road and the upstream end of the of the bank-attached bar, compared to the measured change of approximately 1 foot in this area. In the area of the gravel bar that was dredged in 2007, the SRH-2D model predicts aggradation of 1 to 4 feet over the gravel bar which matches the measured changes very well. In the area from the downstream end of the bank-attached bar to opposite the M&T pumps, the SRH-2D model predicts 1 to 3.5 feet of degradation near the center of the channel, which is very similar to the measured values. In general, the 2-D model predicts the aggradation and degradation patterns and magnitudes reasonably well in the area between River Road and the M&T pumps. The model does not, however, predict degradation measured along the rock toe or along the riprap in the vicinity of the M&T pump intake. The inability of the SRH-2D model to predict the degradation along the banks stems from its inability to accurately represent the 3-dimensional flow fields associated with the formation of the helical cells.
2. The baseline model simulation for the low peak flow hydrograph predicts that relatively low rates of sediment transport occur within the reach. The largest area of bed elevation change

of 1.5 feet occurs at the downstream end of the bank-attached bar, where a sand deposit is eroded and transported downstream and towards the M&T pumps. The model predicts very little change in bed elevation in the vicinity of the M&T pumps; however, due to the inability of the SRH-2D model to predict the erosion along left bank that was observed during the 2011 flood simulation, it is uncertain if the model is over estimating the amount of deposition in the vicinity of the M&T pumps, or if the helical flow cell would form at this discharge and prevent buildup of sediment. The predicted deposition patterns from the low-flow simulation match the measured observations that occurred during previous low-flow events (Tetra Tech, 2011a).

3. The baseline model simulation of the medium peak flow hydrograph (WY2010) indicates that the reach is depositional in the area from the downstream end of the bank-attached bar to below the City of Chico outfall. The model predicts some localized scour and associated deposition in the vicinity of River Road, but does not predict aggradation between River Road and the downstream end of the bank-attached bar.
4. The results of the baseline simulations support the hypothesis that the reach is aggradational during less-than-bankfull flows (~90,000 cfs) and degradational at flows greater than bankfull.
5. WIth the eroded bank model, the area opposite the M&T pumps becomes significantly more depositional than under baseline conditions during the 2011 peak flow hydrograph. Under baseline conditions, the main channel was mostly degradational from the downstream end of the bank-attached bar to just upstream of the City of Chico outfall (a distance of approximately one river mile). Under the eroded bank scenario, the main channel is mostly depositional along this reach.
6. Comparison of the model output for the proposed 9-dike configuration with the baseline conditions indicates that erosion along the upstream end of the gravel bar will increase significantly compared to baseline conditions due to the effective narrowing of the channel caused by the dikes. The baseline conditions model simulation of the 2011 peak flow hydrograph predicted approximately 1.3 feet of erosion along the right (west) edge of the gravel bar and up to 1.0 feet of erosion on the upper surface of the bar. Under the 9-dike configuration, the amount of erosion along the west edge of the gravel bar increases to a maximum of 5 feet and erosion is predicted to occur along the majority of the right edge of the bar. The increase in erosion potential along the right side of the bar should prevent the bar from expanding and migrating in the downstream direction towards the M&T pumps.
7. At the steady state discharge of 90,000 cfs, the 9-dike model predicts that the velocities over the bank-attached bar generally increase by 1.5 fps across the bar and by up to 3 fps at the upstream end of the bar compared to baseline conditions. At the downstream end of the bar and at the M&T pumps, the velocities increase by approximately 1.0 fps and 0.5 fps, respectively, over baseline conditions.
8. Under the 9-dike condition at 90,000 cfs, the water-surface elevations will increase by a maximum of 0.5 feet at the most upstream dike, and by about 0.4 feet at River Road.
9. Under the 9-dike scenario, the simulation for the medium peak flow event predicts only localized areas of bed degradation and significant bed-elevation changes do not occur within the reach. The 9-dike model does not predict the aggradation in the vicinity of the M&T pumps that occurs in the baseline simulation. Therefore, it appears that the 9-dike configuration would effectively prevent aggradation in the vicinity of the M&T pumps during medium-sized peak flow events.

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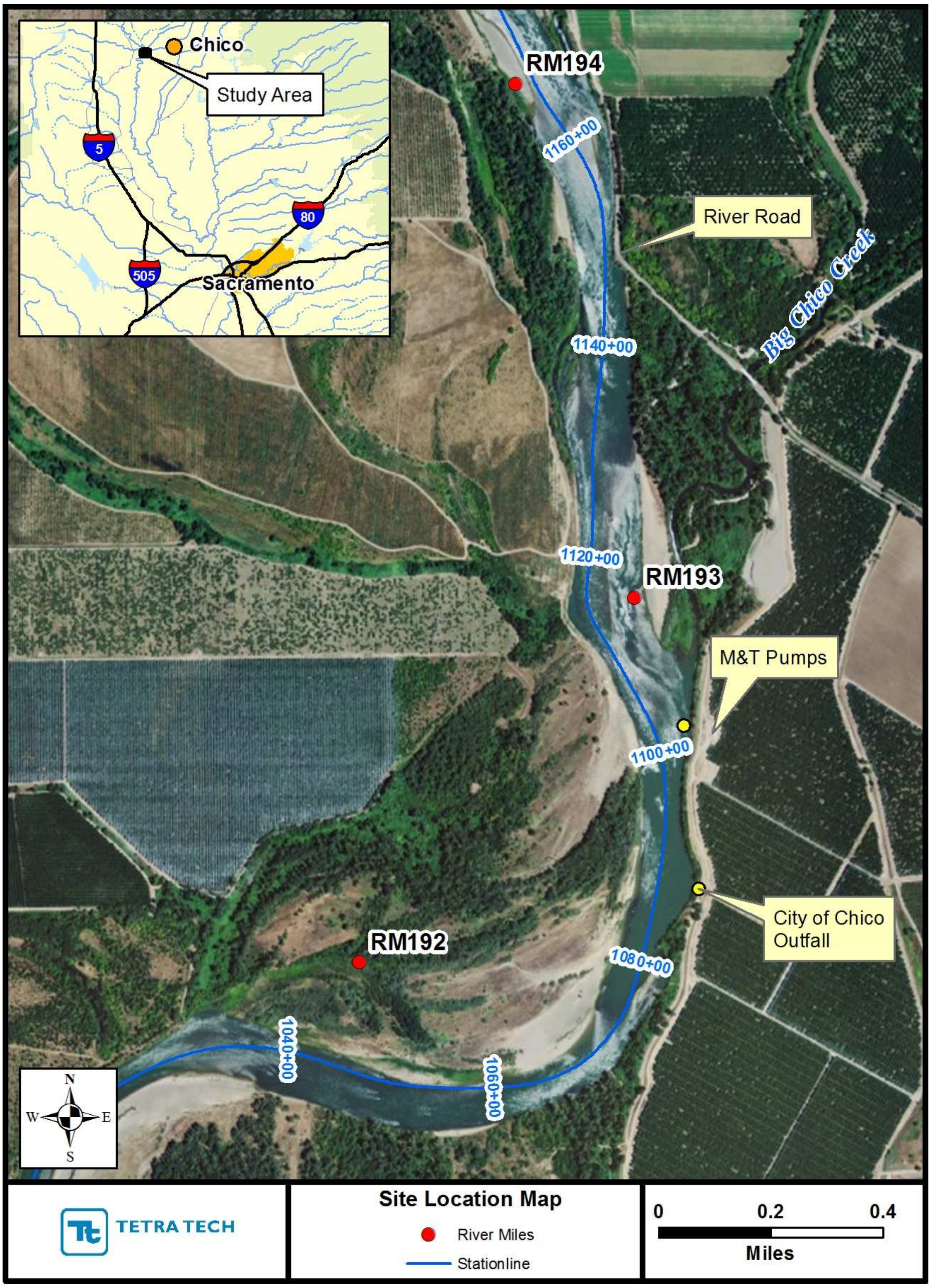

Figure 1. Site location map.

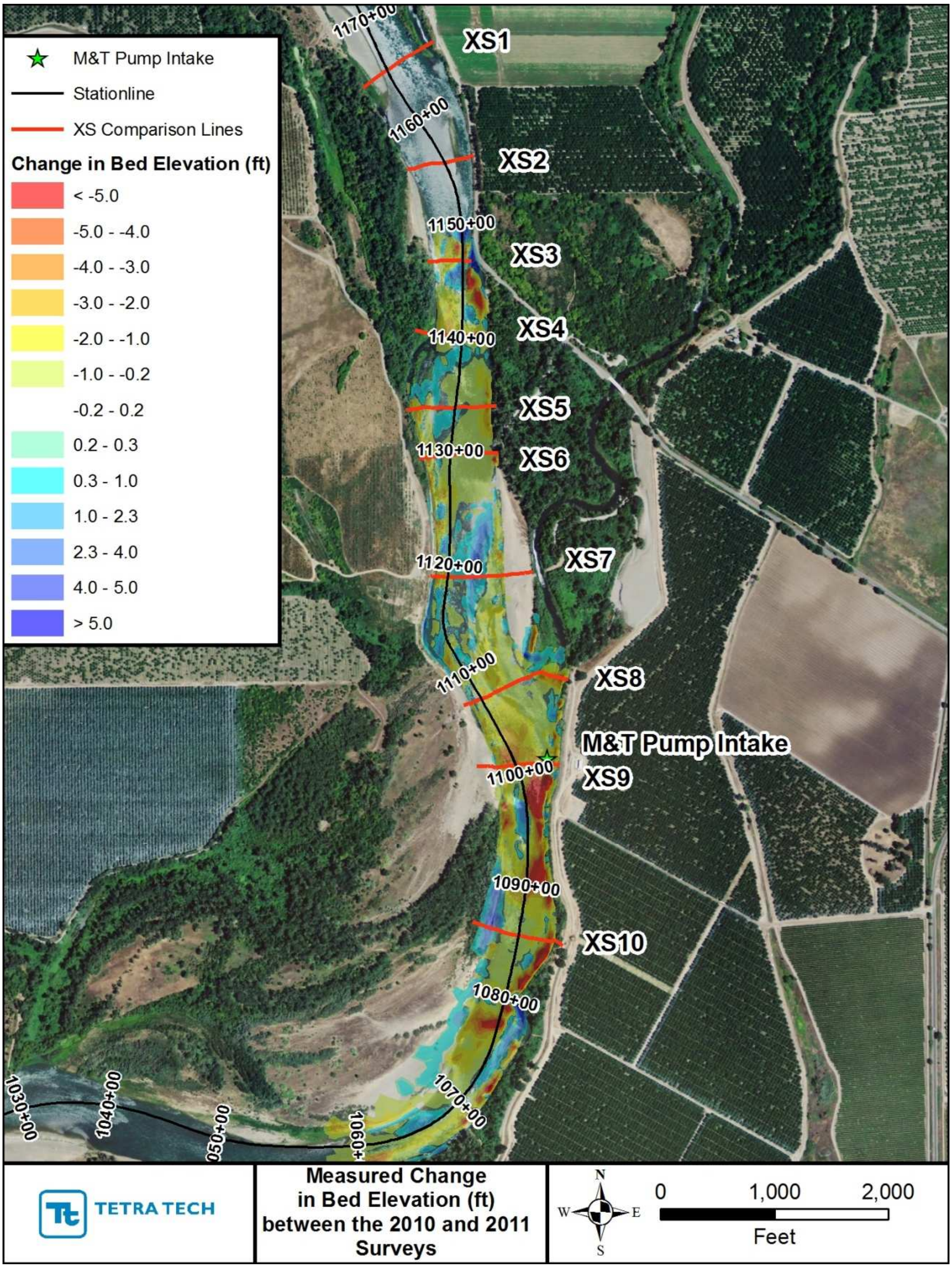

Figure 2. Changes in bed elevation between the January 2010 and June 2011 surveys.

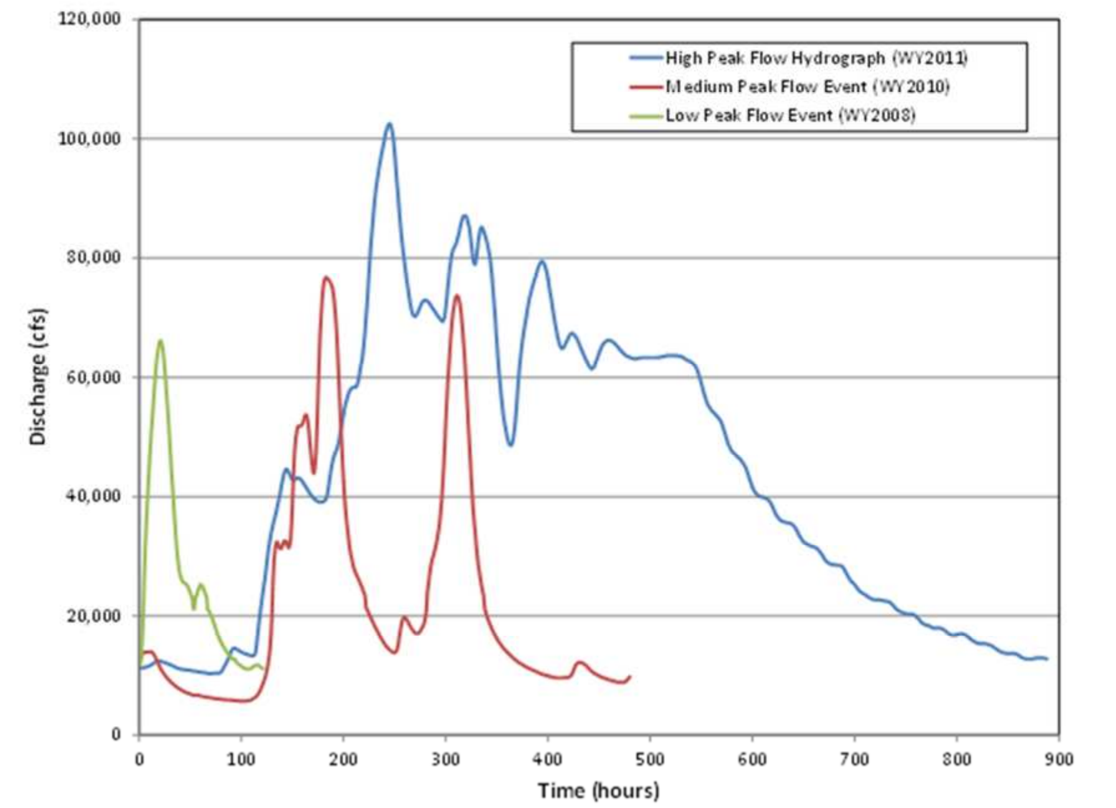

Figure 3. Representative hydrographs for the high, medium and low peak flow events.

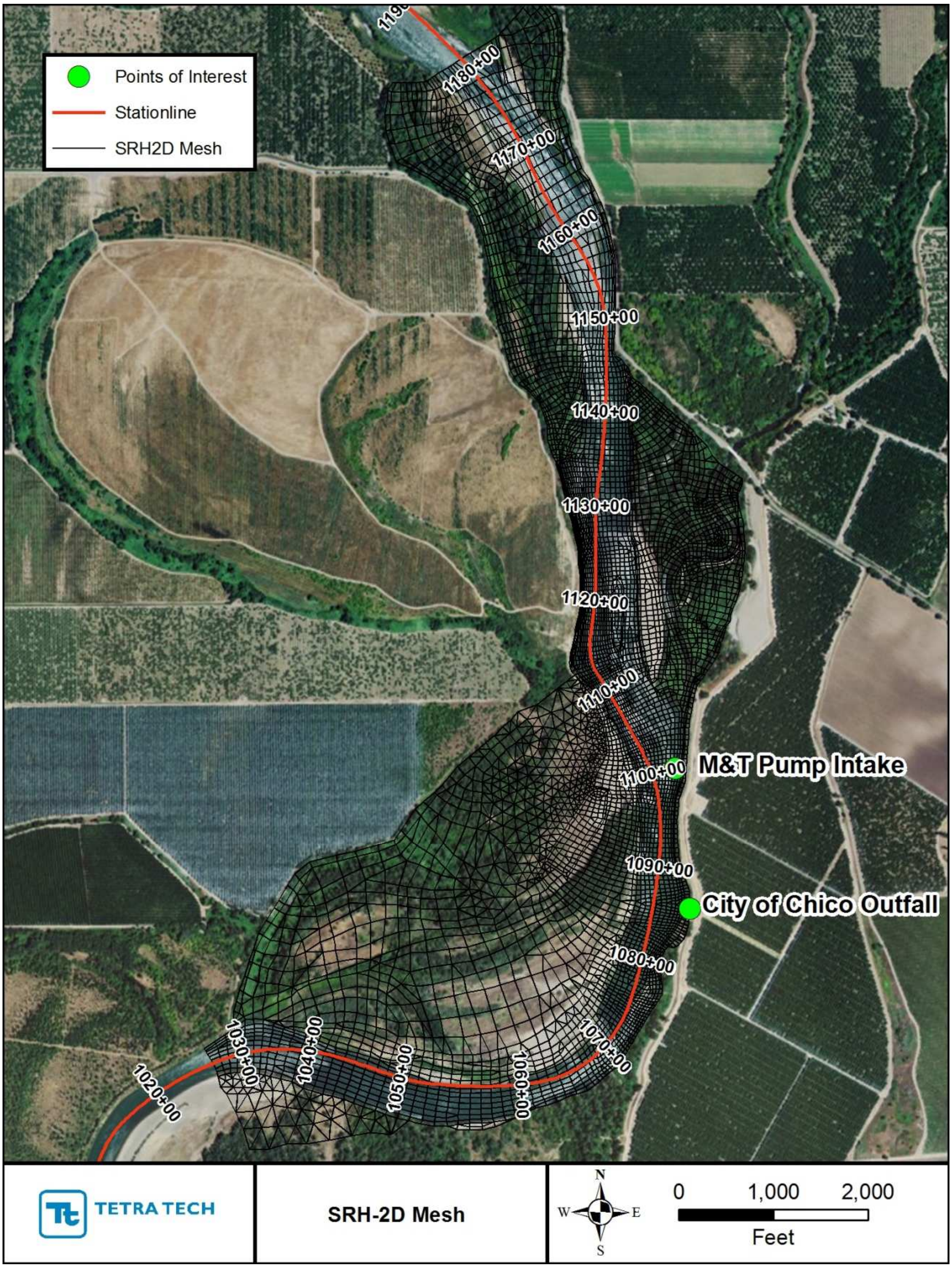

Figure 4. The SRH-2D mesh for the 2010 Phase III model of the M&T pumping plant reach.

Figure 5. Locations of the surface sediment samples that were collected by MEI using the pebble count method (Wolman, 1954) in conjunction with the December 2005 survey.

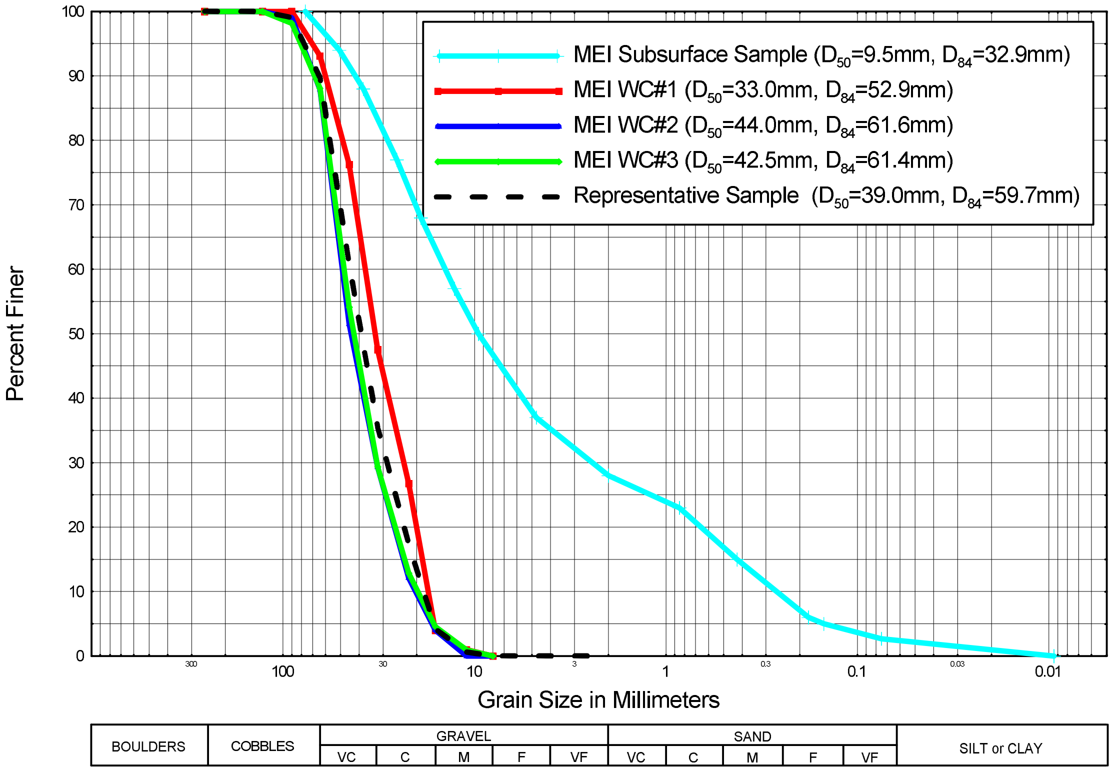

Figure 6. Bed-material gradation curves for samples collected by MEI in conjunction with the December 2005 surveys. Also shown is the representative surface-gradation curve that was used in the sediment-transport analysis.

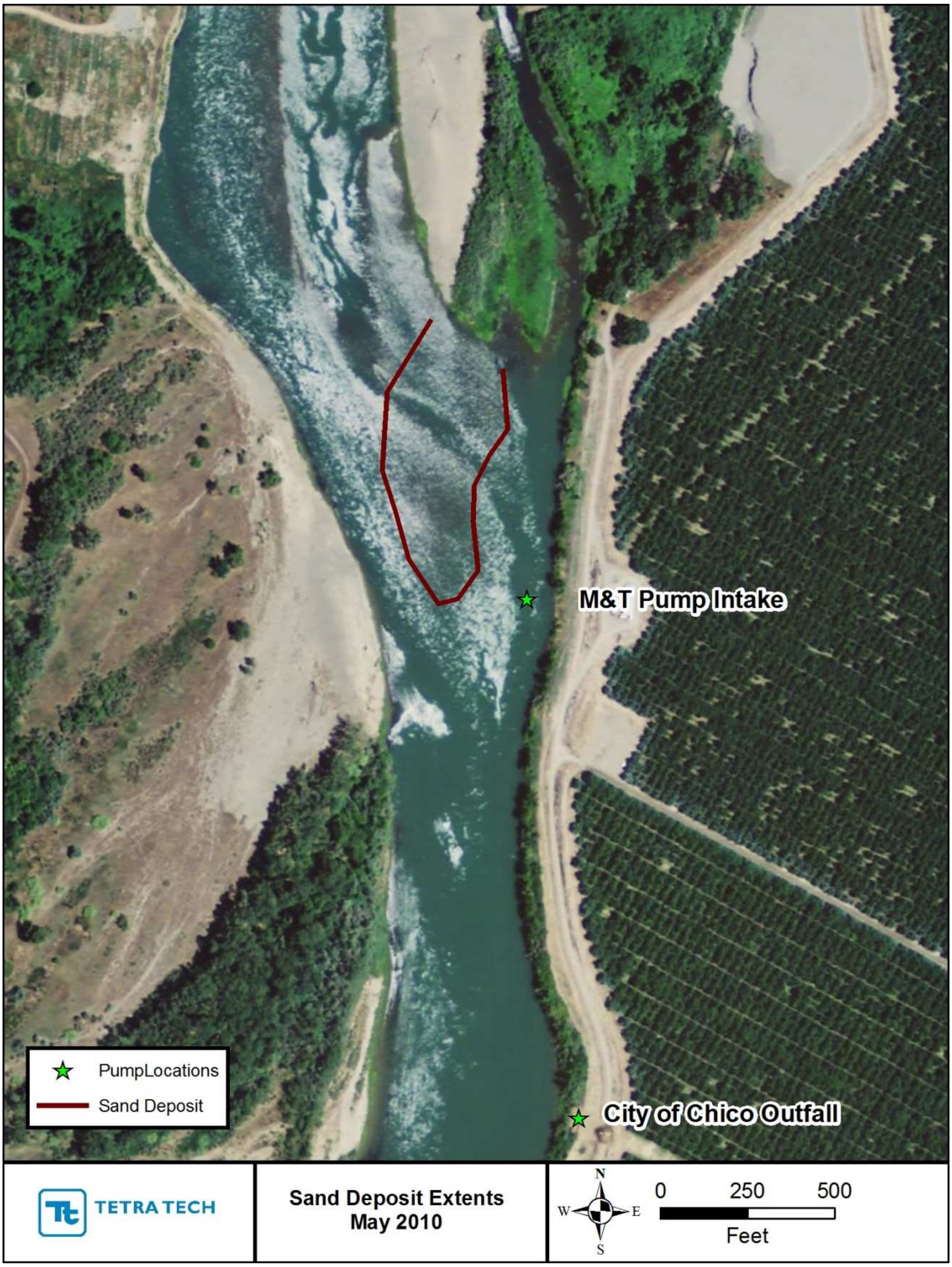

Figure 7. Extent of the sand deposit at the downstream end of the bank-attached bar in May 2010.

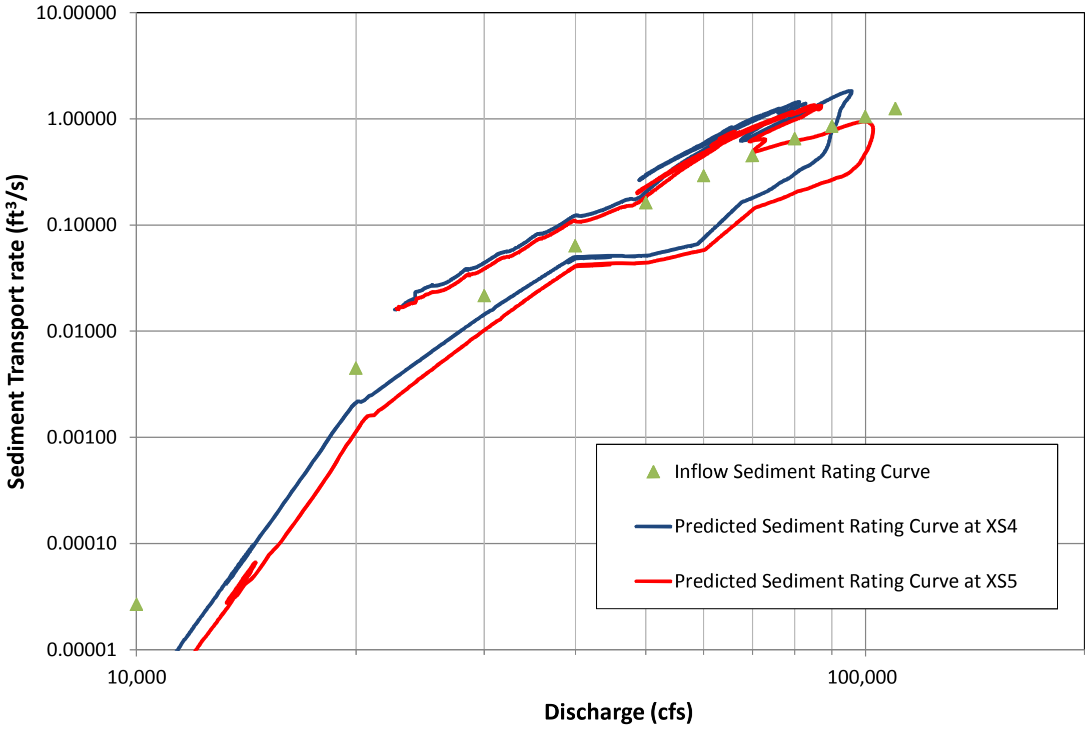

Figure 8. Comparison of the inflow sediment rating curve with the predicted sediment-rating curve at Cross Sections 4 and 5. The locations of Cross Sections 4 and 5 are shown in Figure 9.

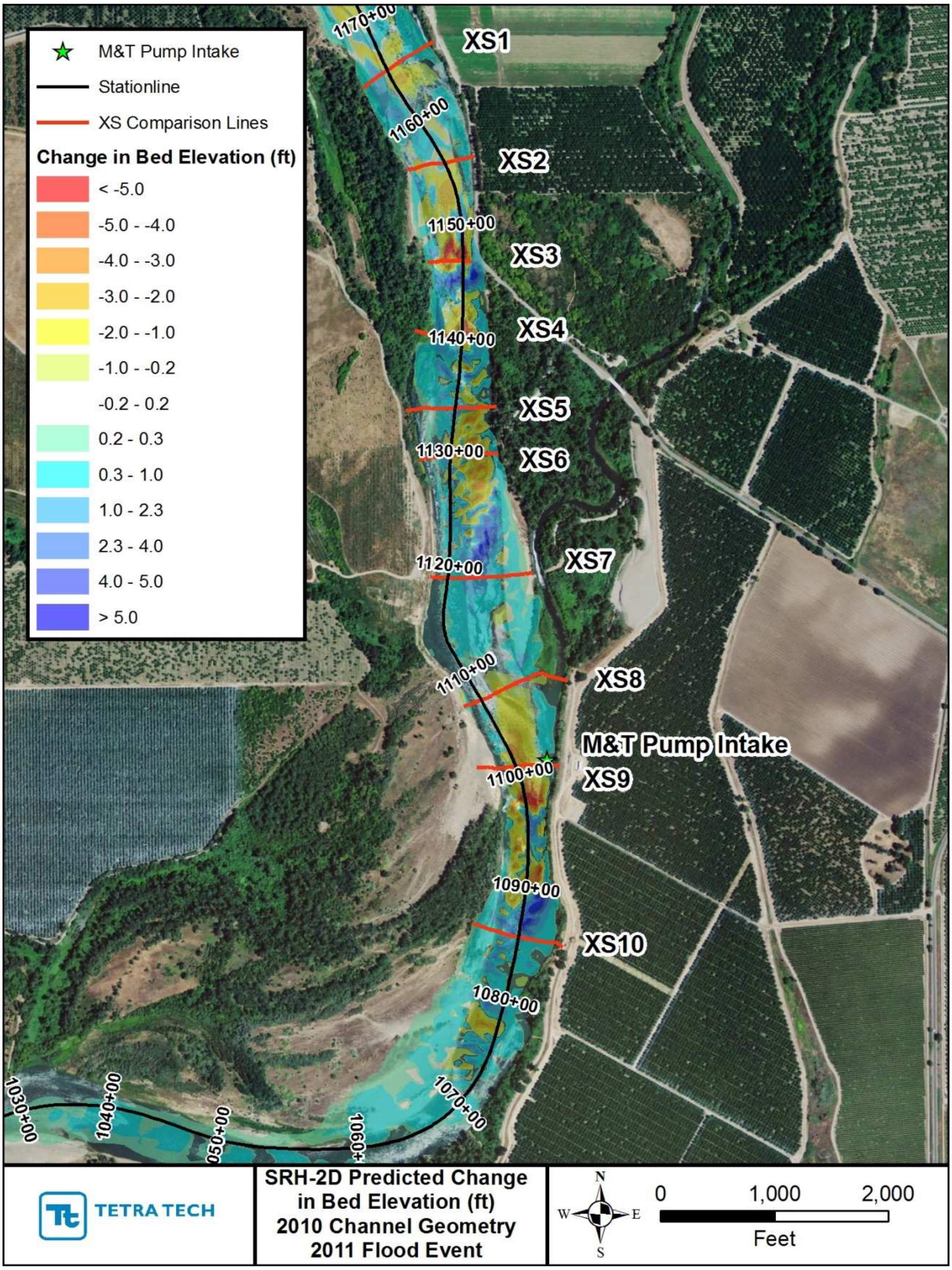

Figure 9. Comparison of the bed elevations at the end of the 2011 flood simulation with the 2010 and 2011 channel geometry.

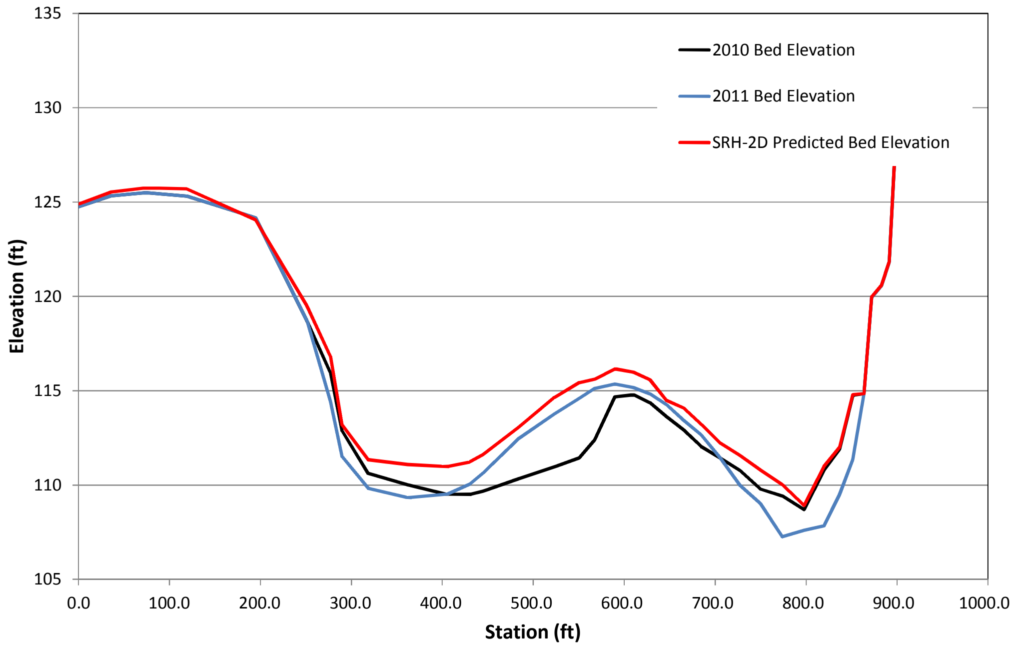

Figure 10. Comparison of the bed elevations at the end of the 2011 flood simulation with the 2010 and 2011 channel geometry across the bank-attached bar (Cross Section 7).

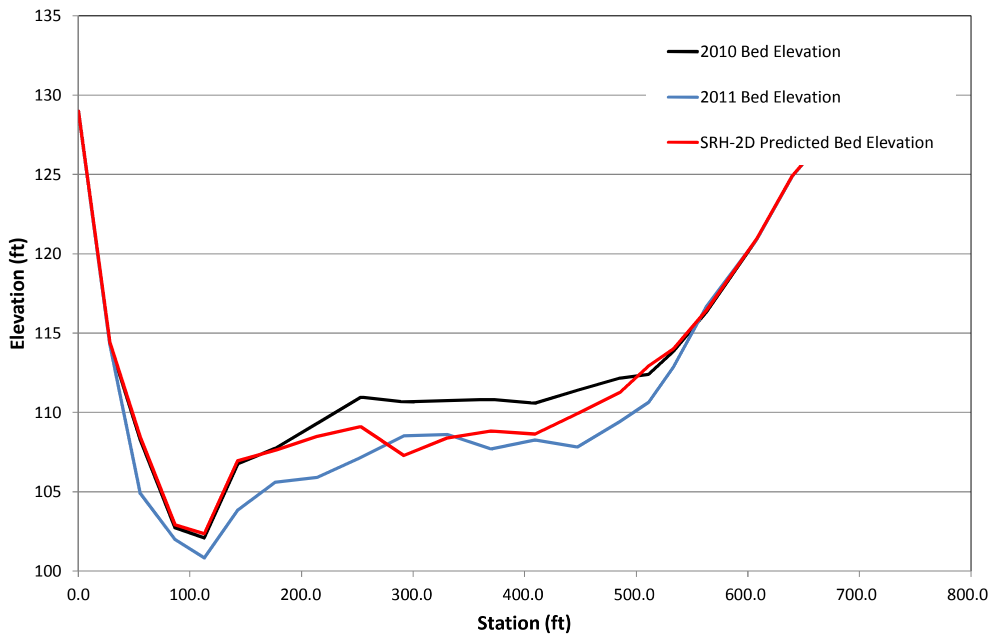

Figure 11. Comparison of the bed elevations at the end of the 2011 flood simulation with the 2010 and 2011 channel geometry at the M&T pumps (Cross Section 9).

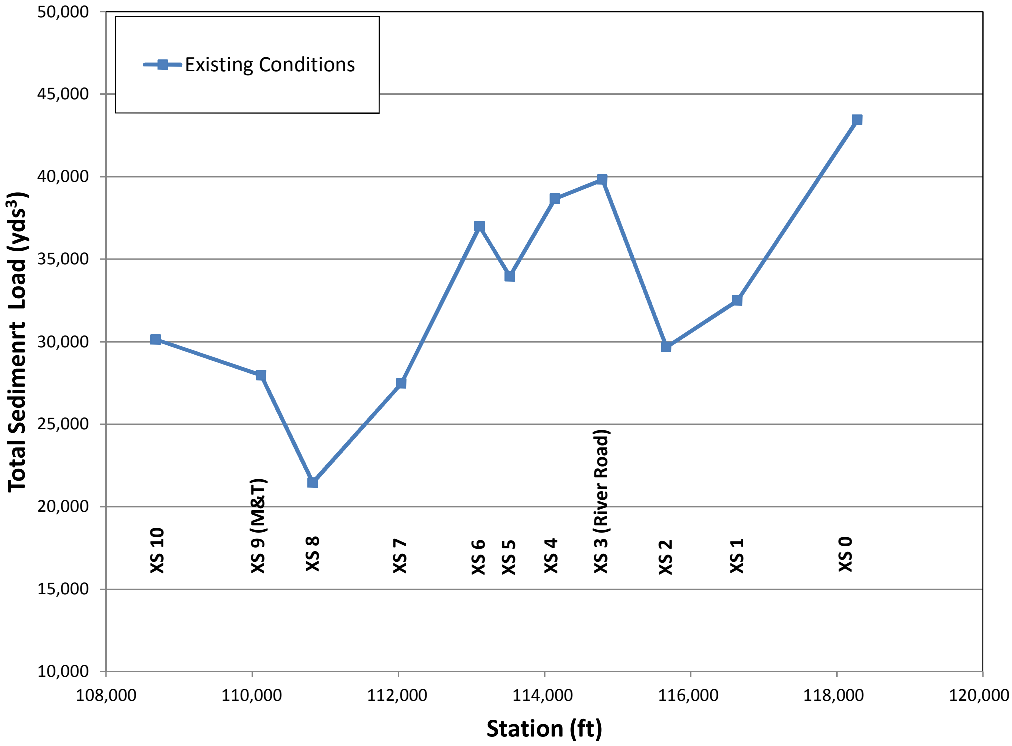

Figure 12. Predicted total sediment-transport loads over the duration of the 2011 hydrograph under baseline conditions. The locations of the cross sections are shown in Figure 9.

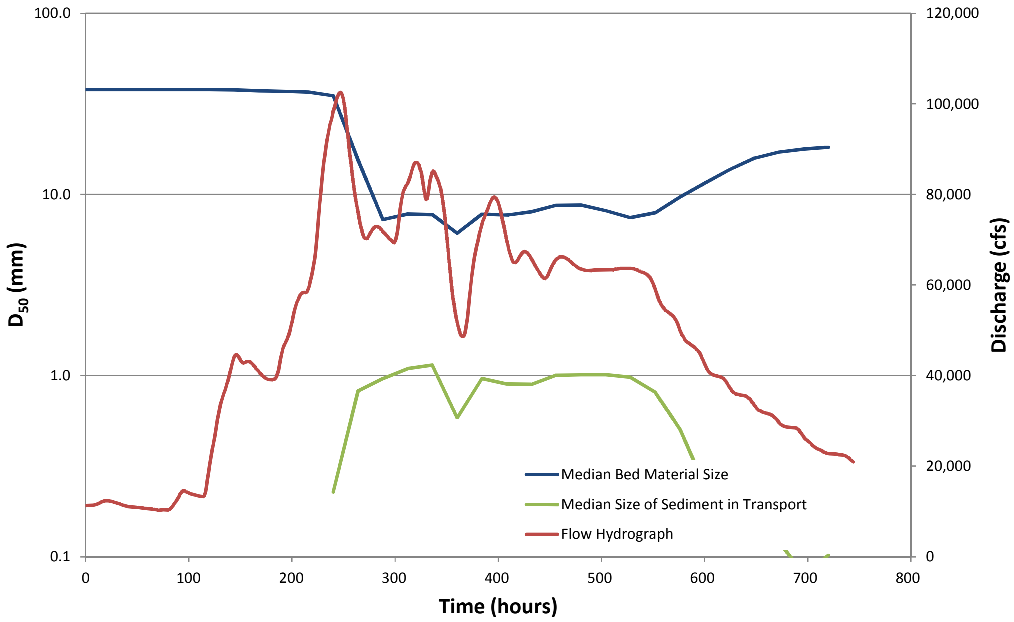

Figure 13. Predicted median (D_{50}) bed-material size over the duration of the 2011 hydrograph at the location of WC3.

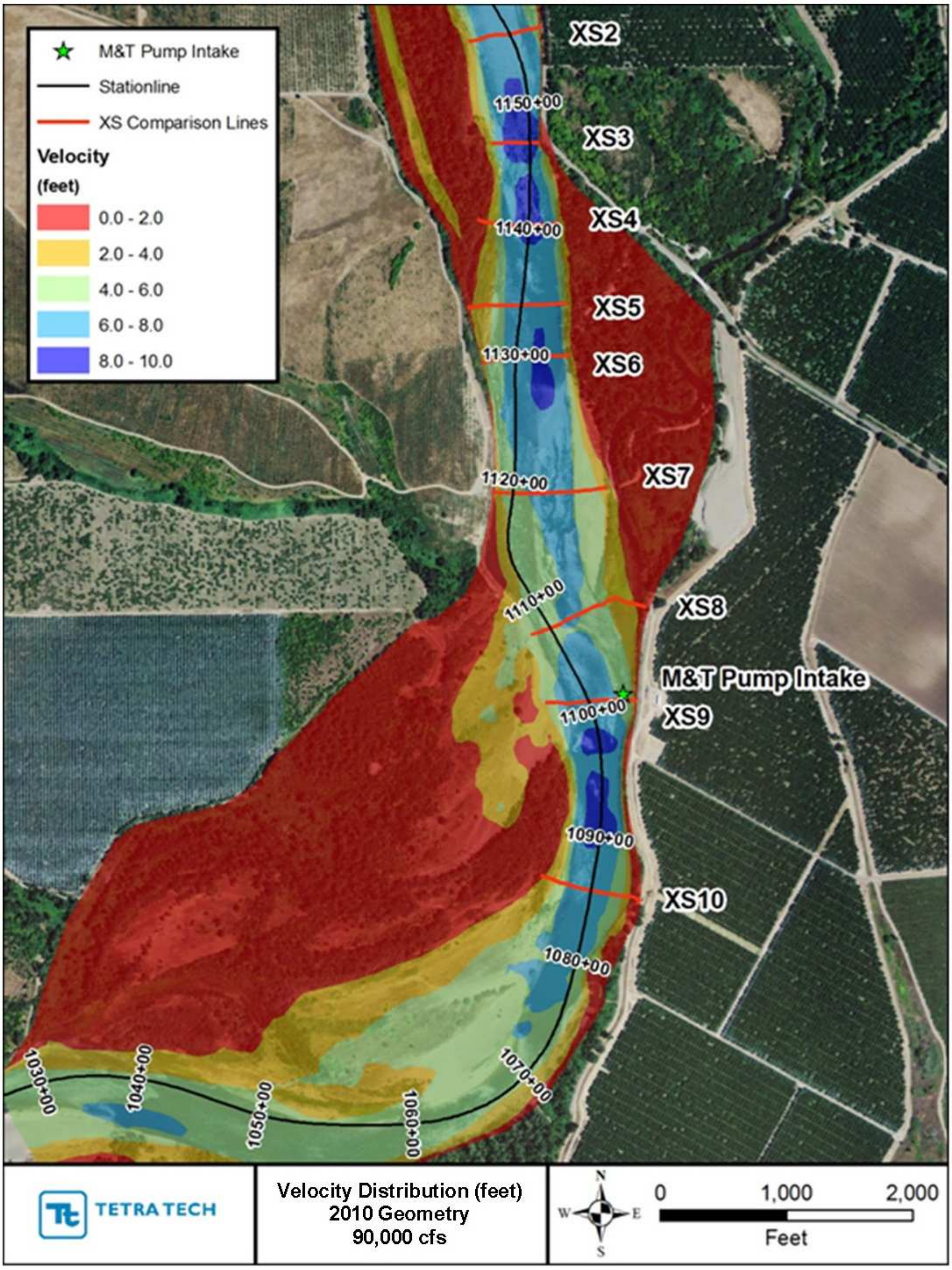

Figure 14. Velocity distribution predicted by the baseline conditions model at a discharge of 90,000 cfs.

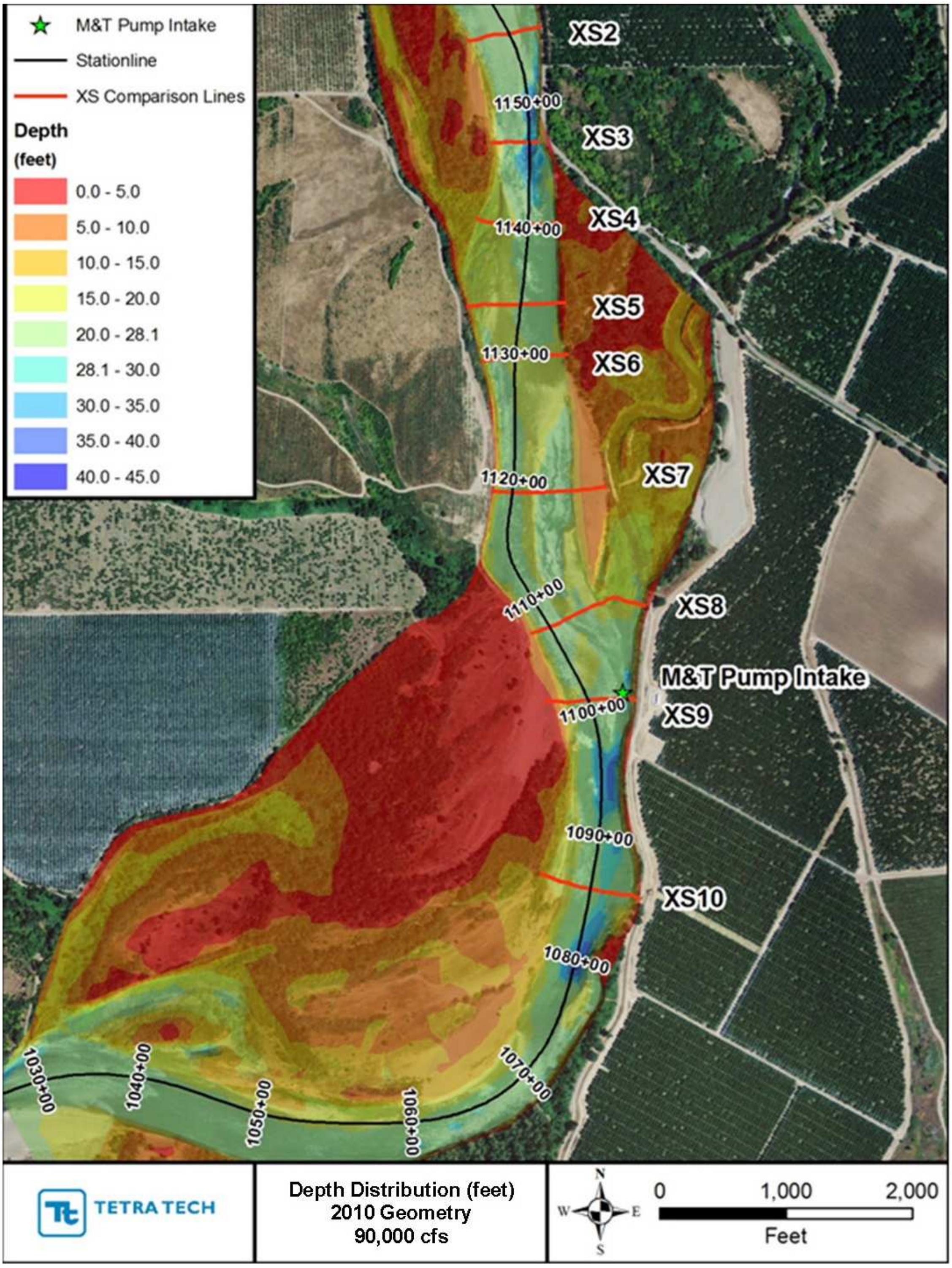

Figure 15. Depth distribution predicted by the baseline conditions model at a discharge of 90,000 cfs.

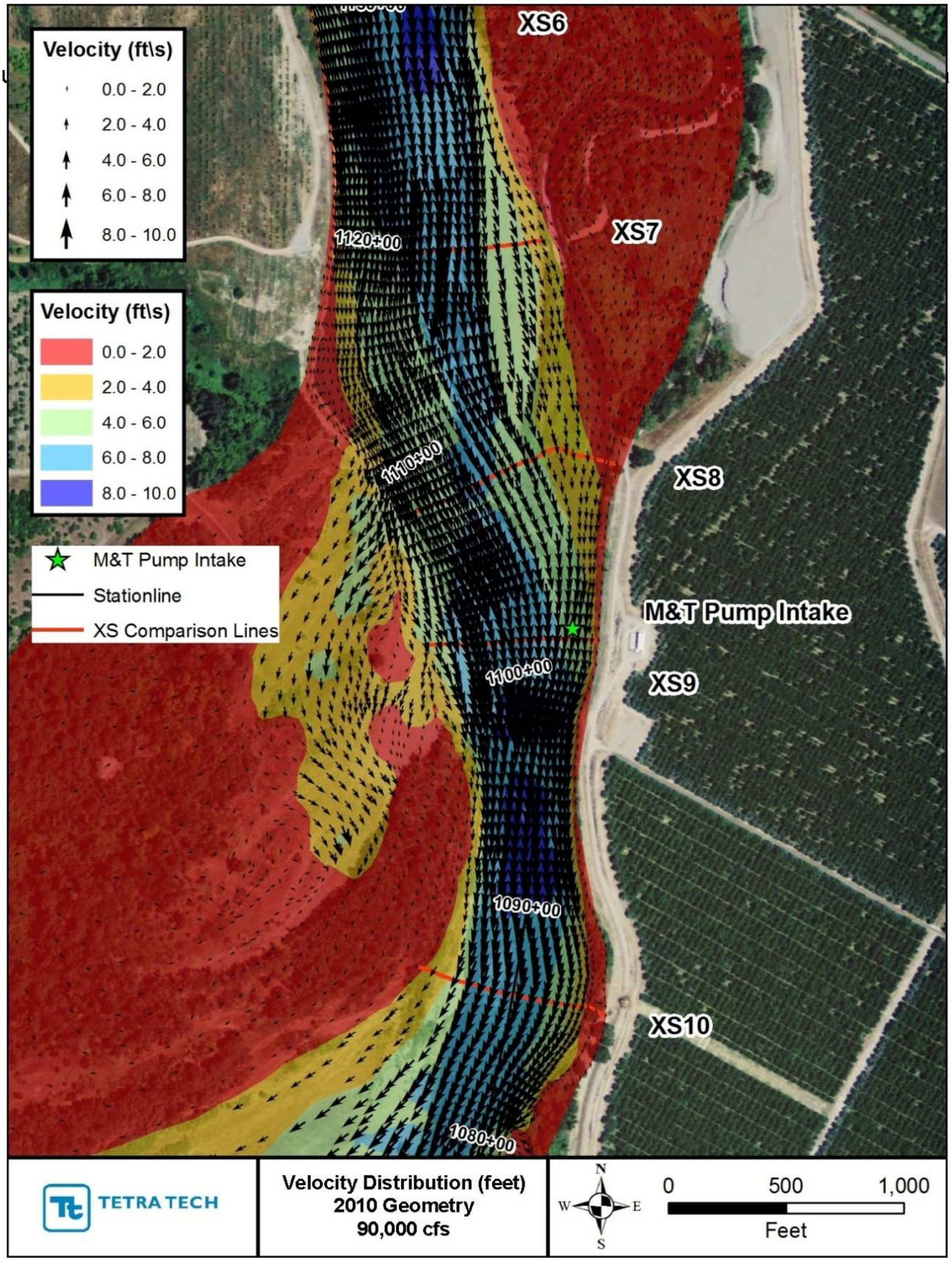

Figure 16. Predicted velocity orientation under the baseline conditions model at a discharge of 90,000 cfs.

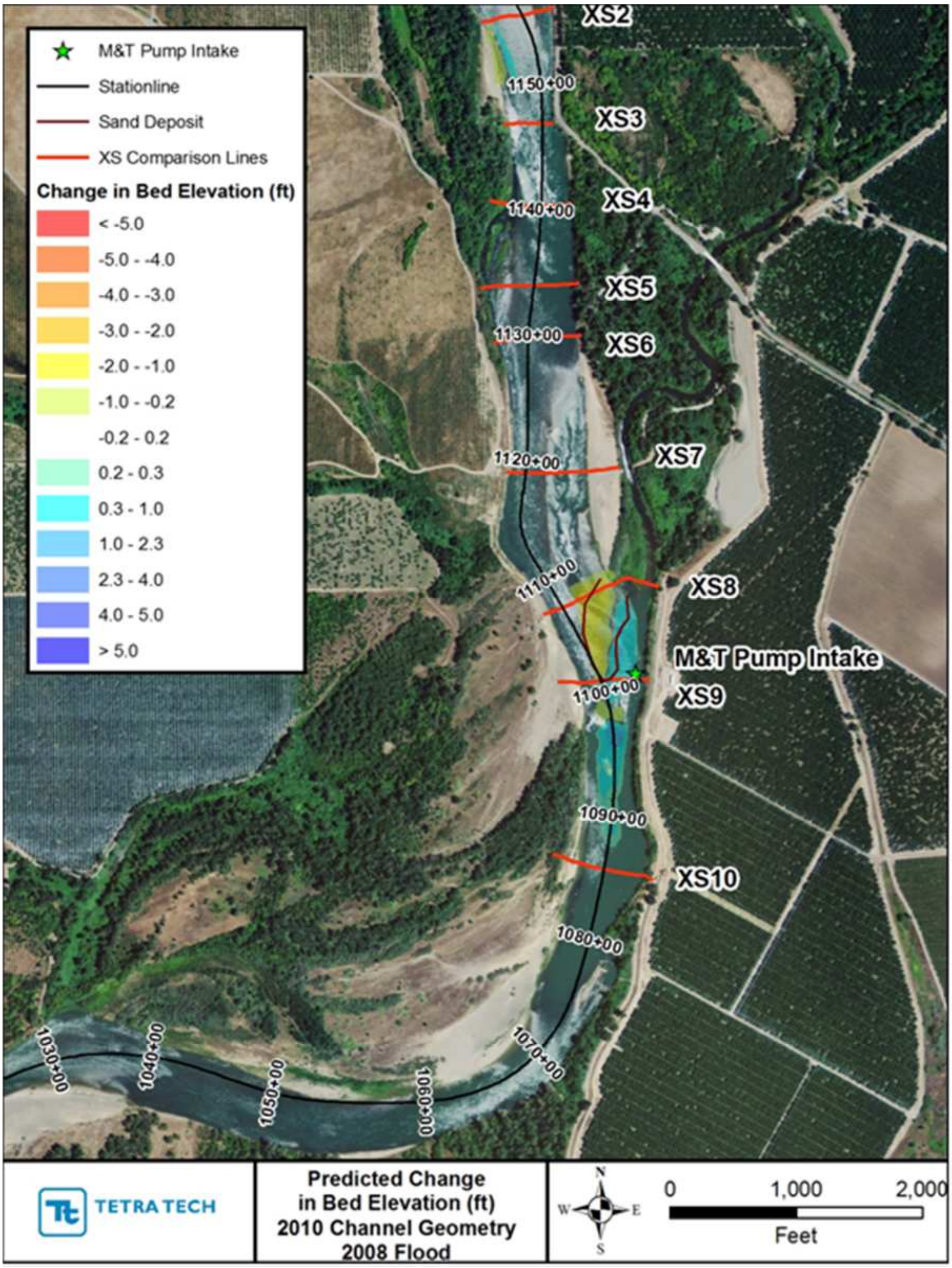

Figure 17. Change in bed elevation predicted by SRH-2D model simulation of the 2008 flood event.

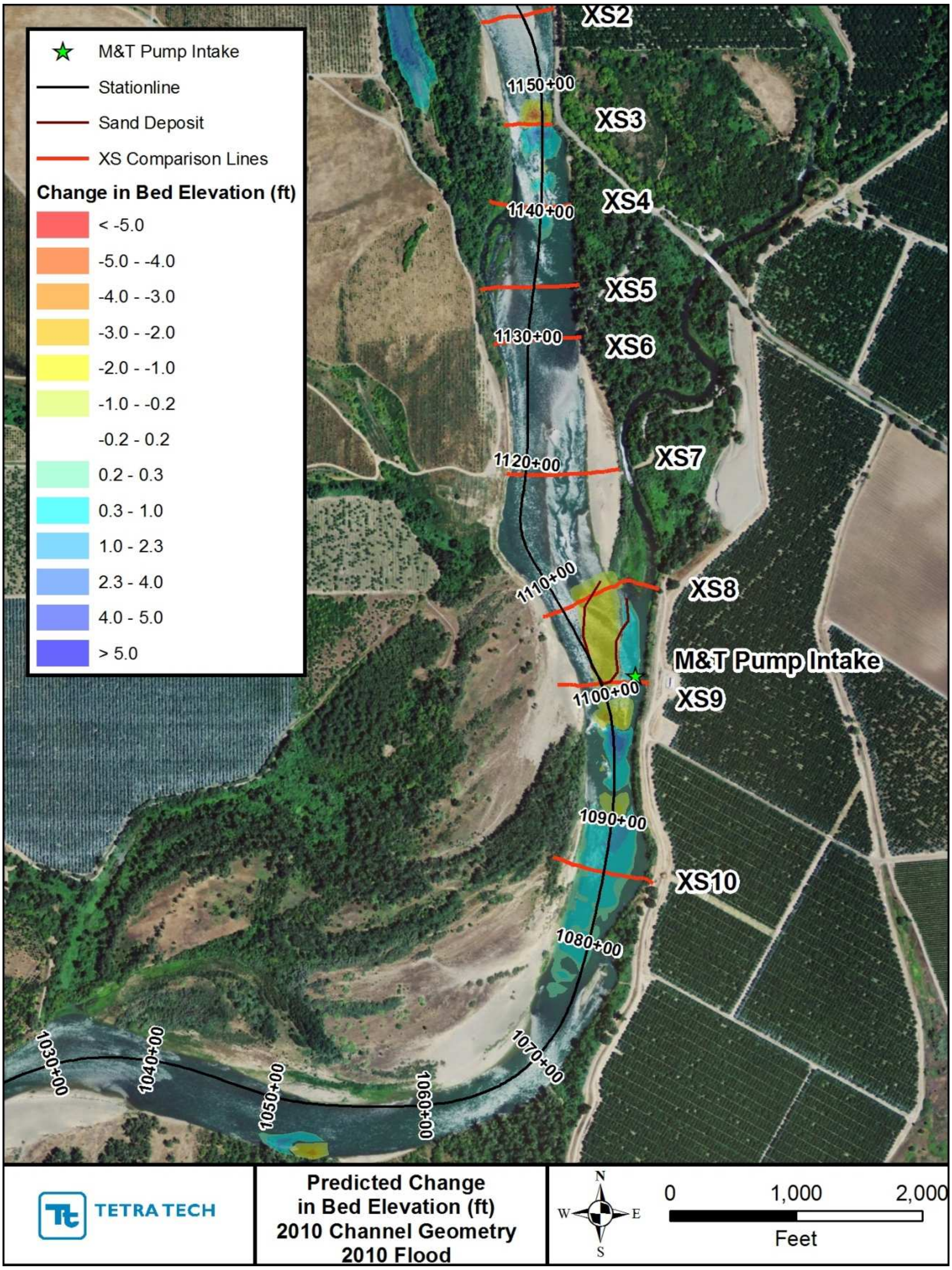

Figure 18. Change in bed elevation predicted by SRH-2D model simulation of the 2010 (medium peak flow) flood event.

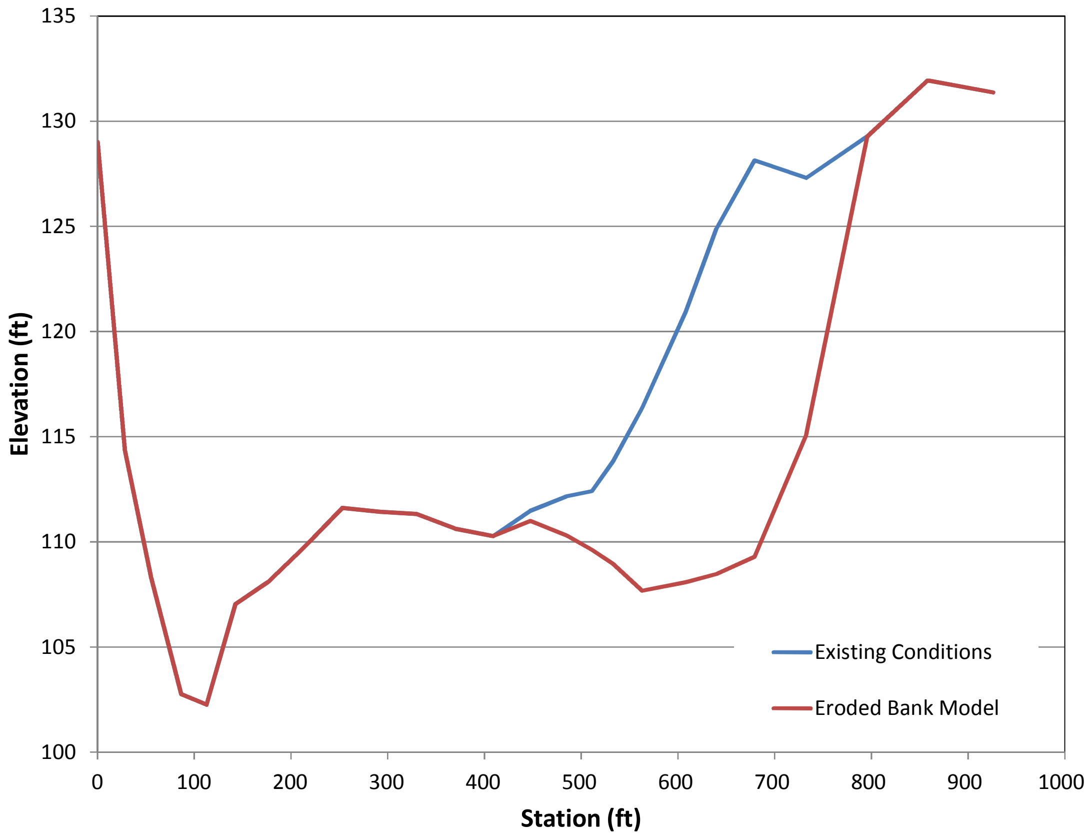

Figure 19. Comparison of the channel geometry for the baseline and eroded bank conditions at Cross Section 9.

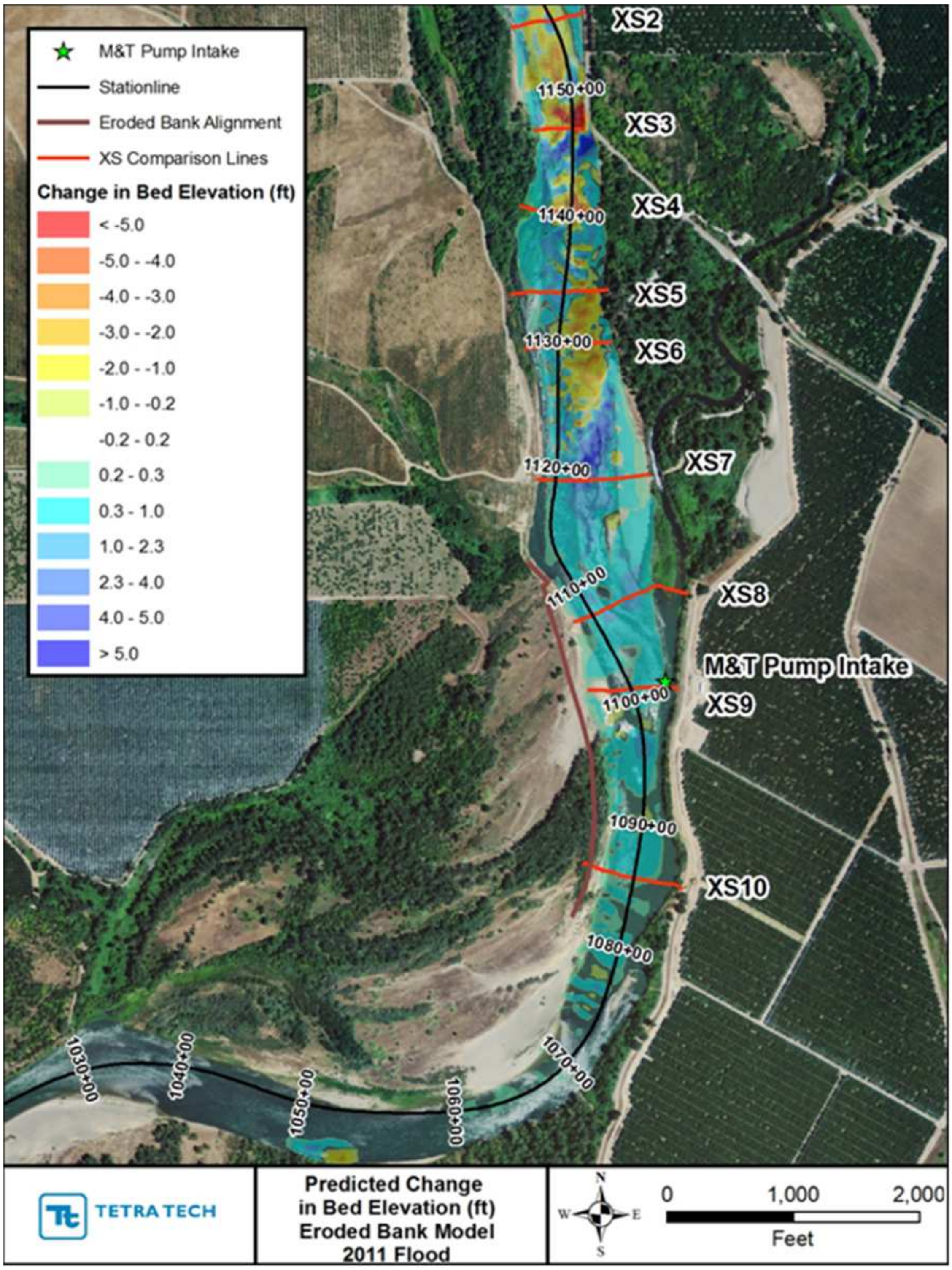

Figure 20. Change in bed elevation predicted by Eroded Bank Model for the 2011 flood event.

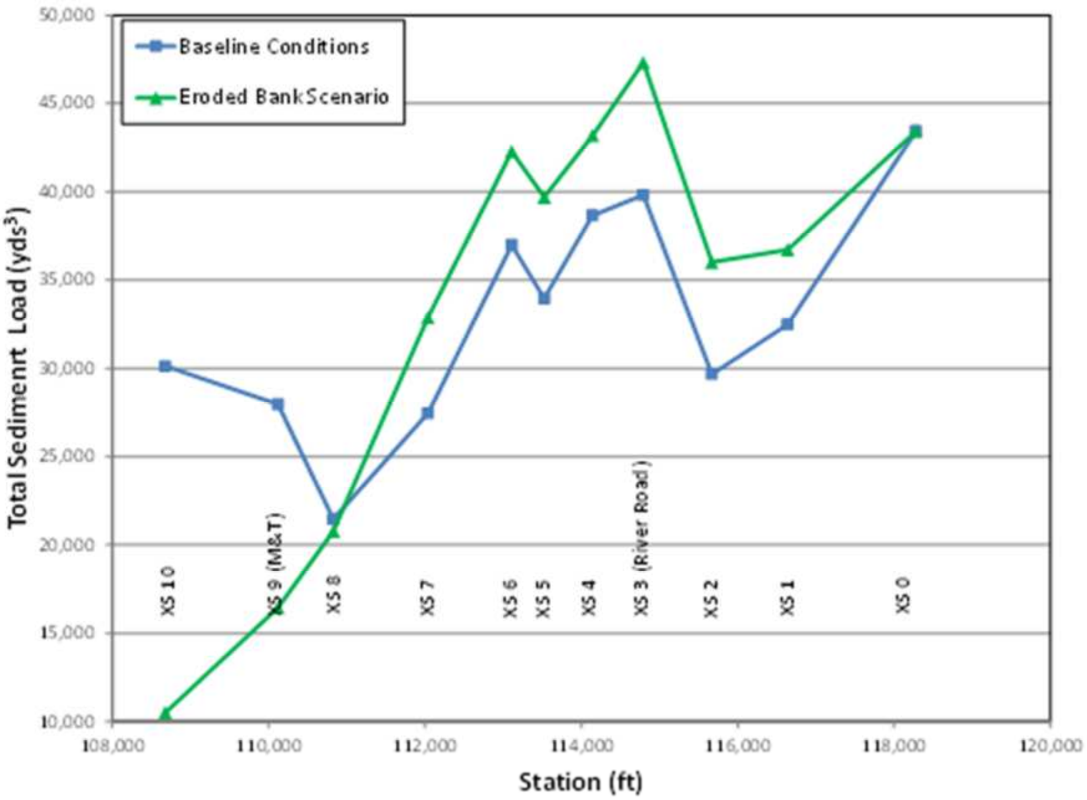

Figure 21. Predicted total sediment-transport loads over the duration of the 2011 hydrograph under baseline and eroded bank scenarios. The locations of the cross sections are shown in Figure 9.

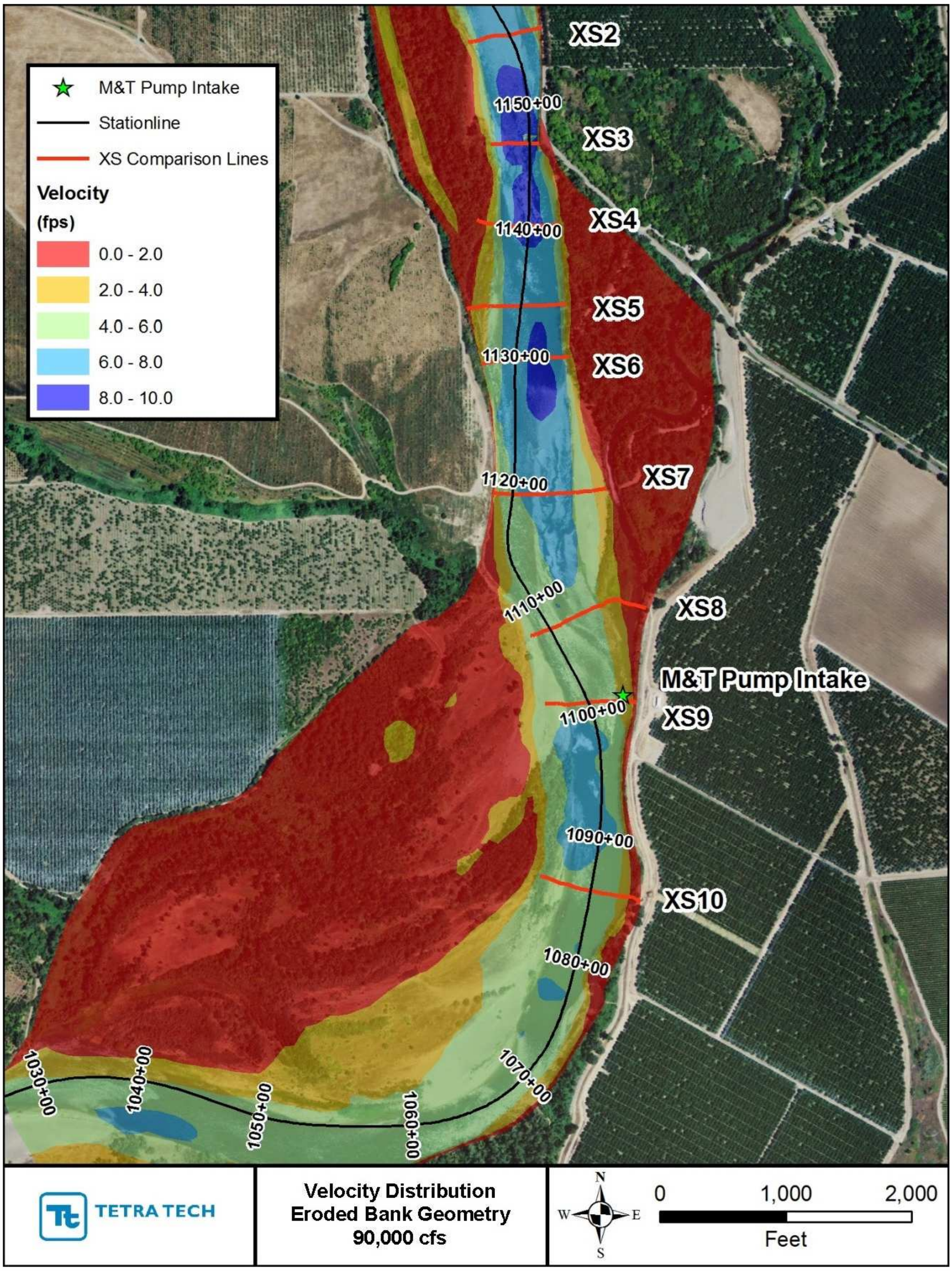

Figure 22. Velocity distribution predicted by the Eroded Bank Model at a discharge of 90,000 cfs.

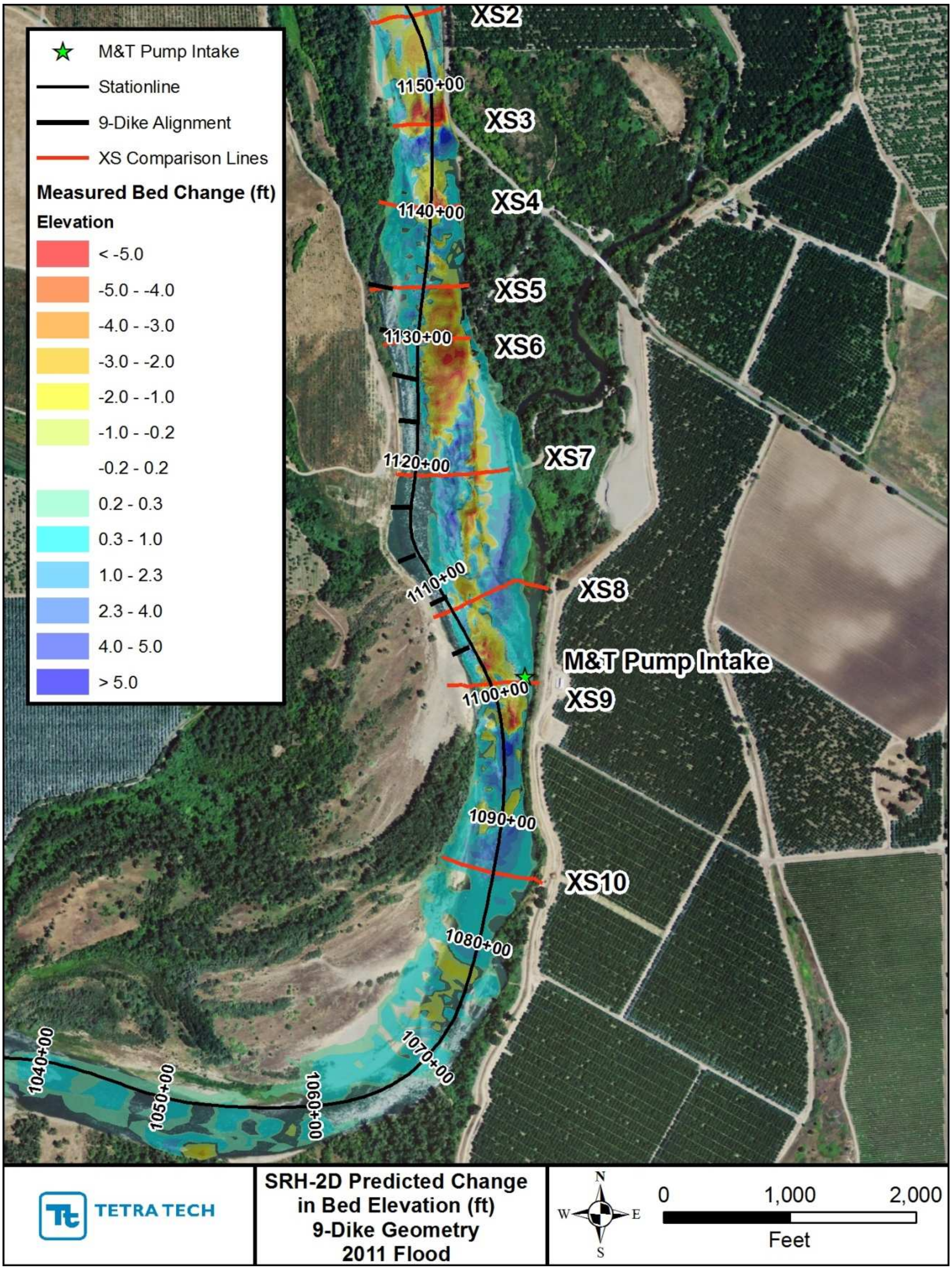

Figure 23. Change in bed elevation predicted by 9-dike model from the 2011 peak flow hydrograph simulation.

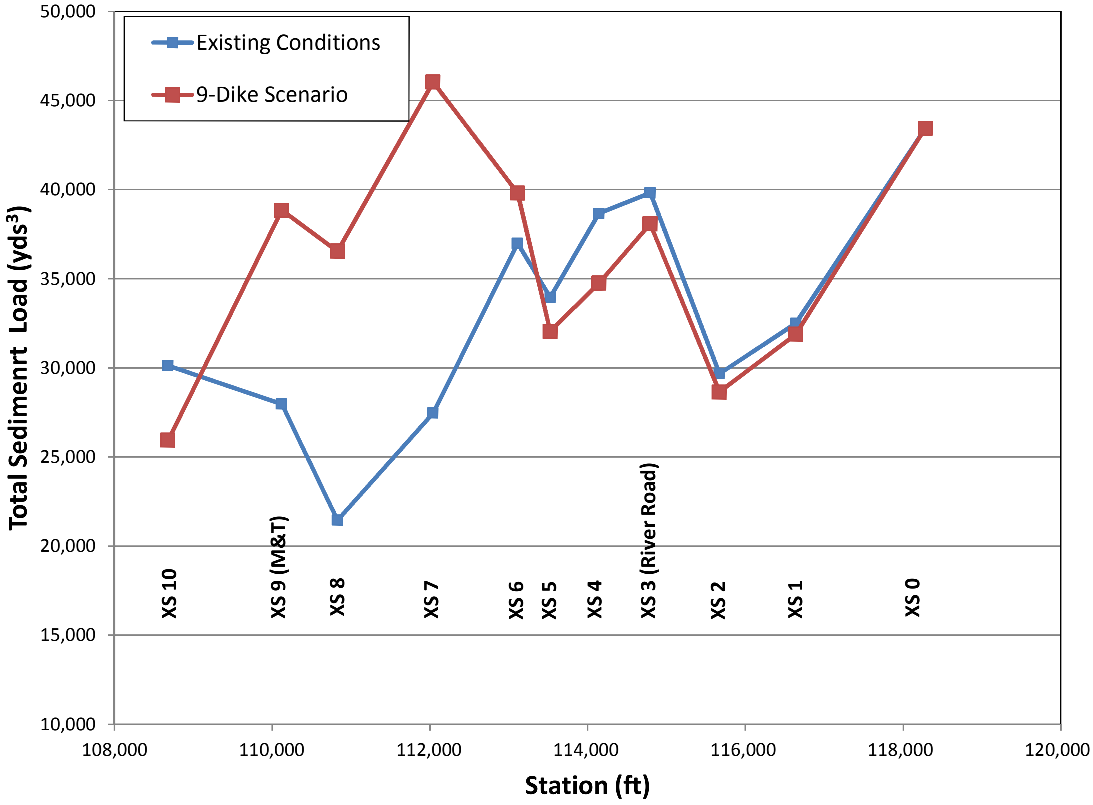

Figure 24 Predicted total sediment-transport loads over the duration of the 2011 hydrograph under baseline and 9-dike scenarios.

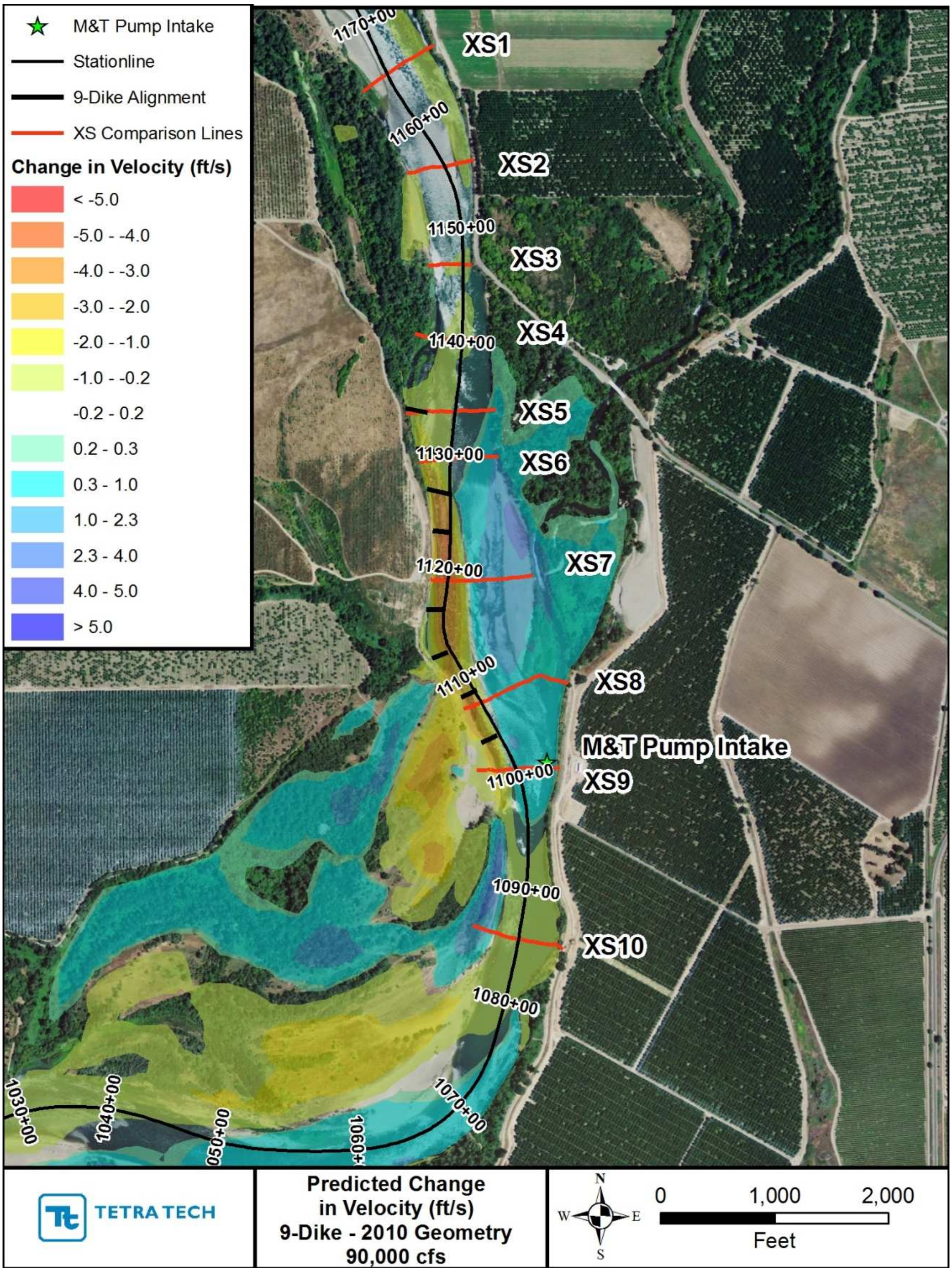

Figure 25. Predicted change in velocity at 90,000 cfs between the between the baseline and the 9-dike models.

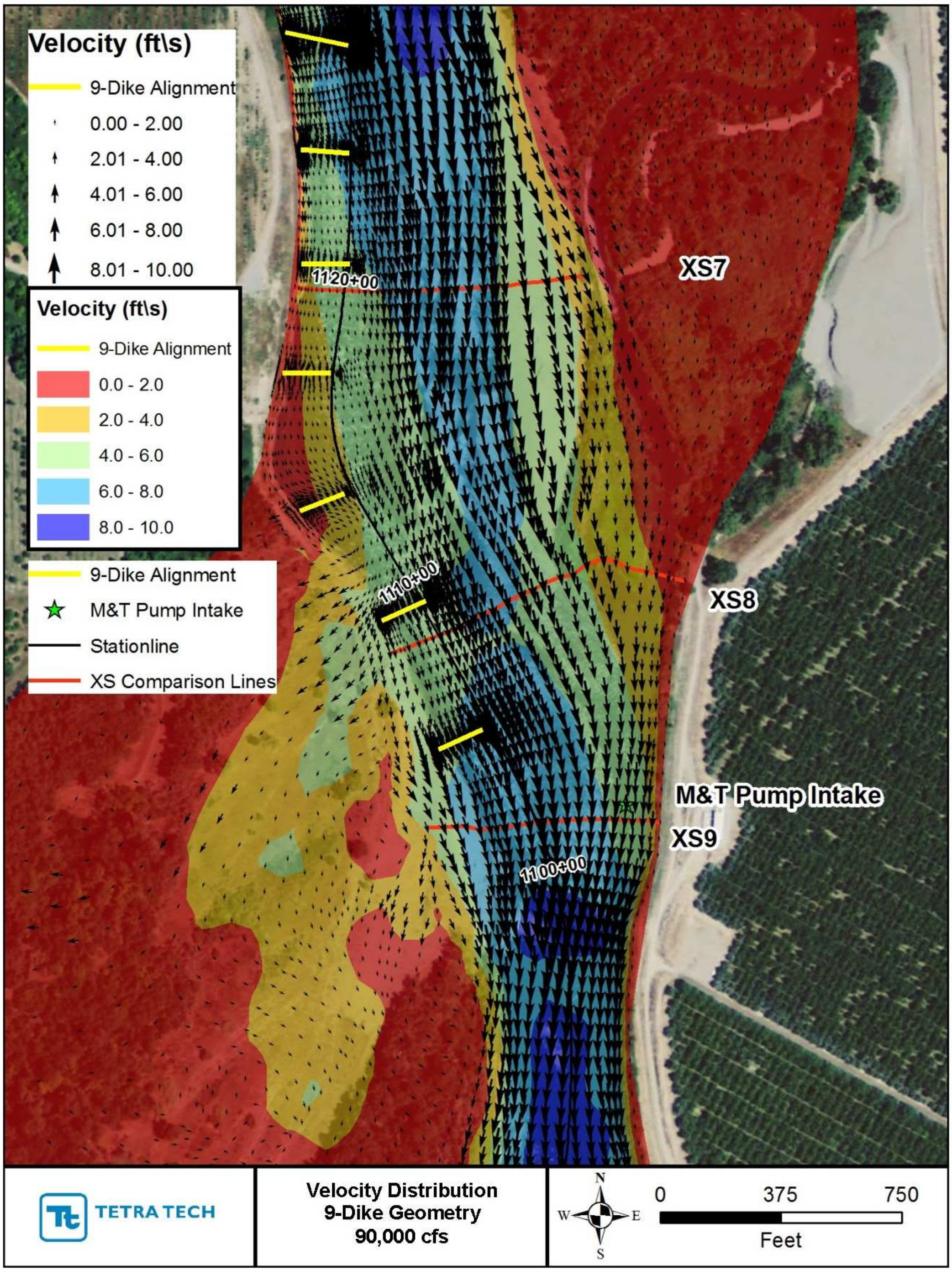

Figure 26. Predicted velocity orientation under the 9-dike conditions model at a discharge of 90,000 cfs.

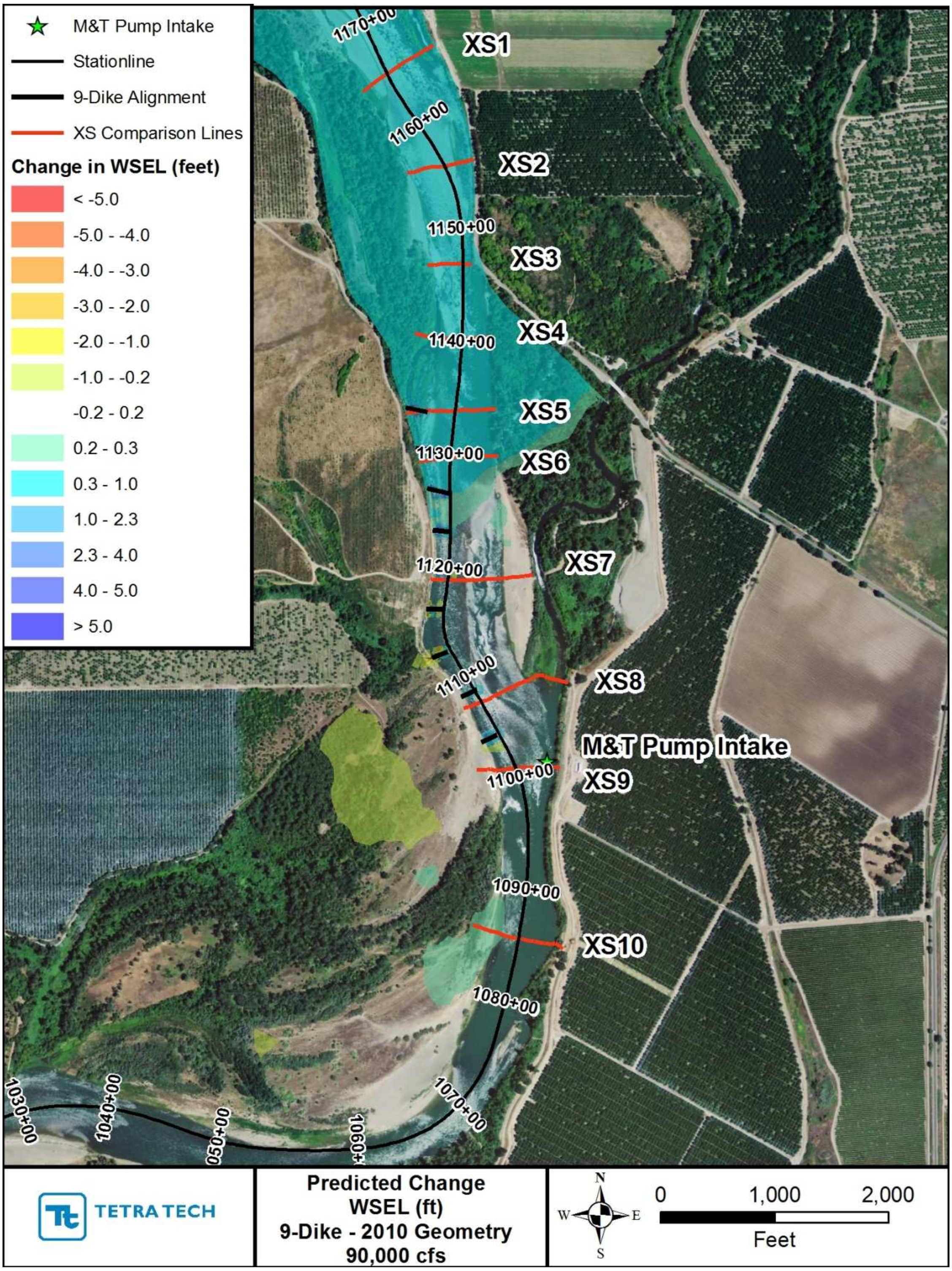

Figure 27. Predicted change in water-surface elevation at 90,000 cfs between the baseline and the 9-dike models.

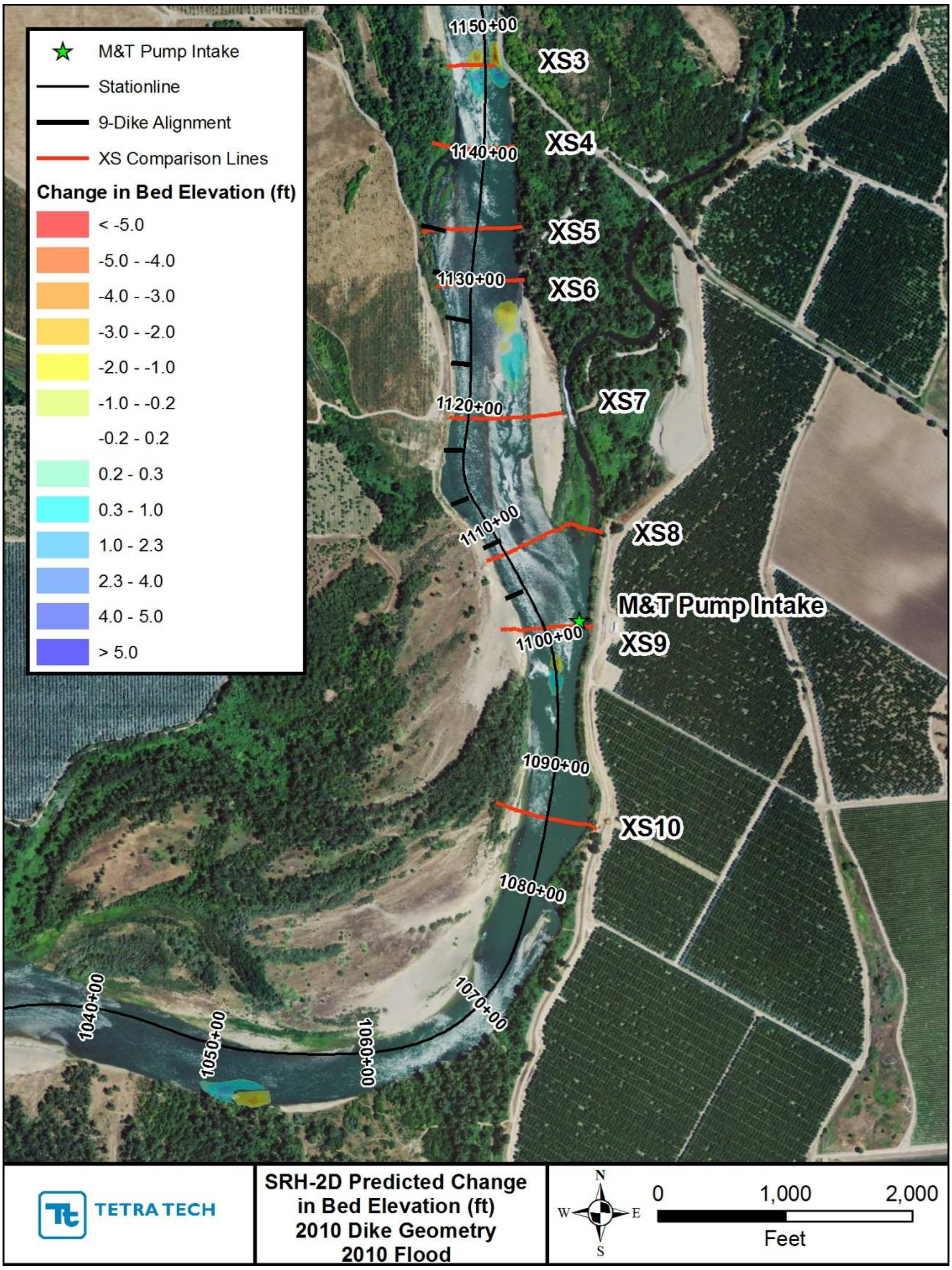

Figure 28. Change in bed elevation predicted by simulation of the 9-dike model during the 2010 flood event.