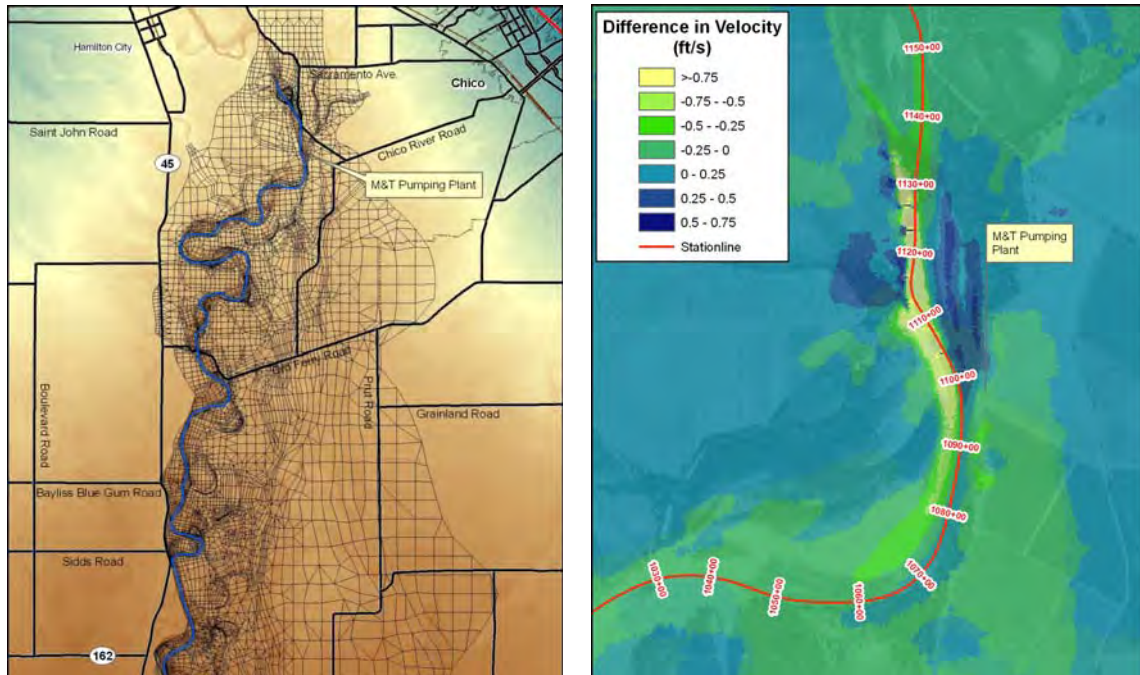


Phase III Two-Dimensional Modeling of M&T/Llano Seco Pumping Plant Reach, Sacramento River, RM192.5, California



Task Order 13.1: Hamilton City J-Levee Impacts
Task Order 13.4: Two-dimensional Modeling

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October 5, 2011

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EXECUTIVE SUMMARY

The specific objectives of this two-dimensional (2-D) hydraulic investigation of the M&T/Llano Seco Pumping Plant Reach of the Sacramento River (River Mile 192.5) were to:

1. Investigate the hydraulic impacts, if any, of the upstream Hamilton City Setback Levee project on the existing M&T Pumps and at the relocated City of Chico wastewater outfall,
2. Evaluate the hydrodynamic conditions over a range of flows at two potential alternative pumping sites located 2,200 and 3,500 feet downstream, respectively from the existing pumping site,
3. Investigate the hydraulic impacts, if any, of the Hamilton City Setback Levee project on the potential long-term solution alternatives at the M&T Pumps and the City of Chico outfall, and conversely, investigate the long-term solution alternatives impacts, if any, on the Hamilton City Setback Levee project, and
4. Investigate the hydrodynamic impacts of locating a gravel stockpile on the west overbank opposite the M&T Pumps and to investigate the mobility of the sediments in the stockpile.

To address the specific objectives of the project, individual SRH-2D computer models were developed for each objective:

1. The existing conditions Phase II model (MEI 2008) was updated with topographic and bathymetric survey data collected in January 2010; this model is referred to as the Phase III model (Table 1.1). The Phase III model was incorporated into the coarser scale Corps' (2008) J-Levee model that extends from RM 192 to RM 202; this model is referred to as the Phase III J-Levee model and it represents the existing conditions in the reach. The Phase III model was incorporated into the coarser scale Corps' (2008) Setback Levee model that extends from RM 192 to RM 202; this model is referred to as the Phase III Setback Levee model and it includes the geometry of the proposed Hamilton City Setback Levee. The Phase III J-Levee model output and the Phase III Setback Levee model output from the 50- and 100-year peak flow (237,829 cfs and 275,910 cfs, respectively) simulations were compared to evaluate the hydrodynamic effects of the Hamilton City Setback Levee project at the M&T Pumps and the City of Chico outfall.
2. The results of the Phase III and Phase III Setback Levee models were used to develop rating curves to evaluate the hydraulic conditions at the M&T Pumps, relocated City of Chico outfall and at the Alternative 1 pump relocation site (2,200-foot site). The Alternative 2 pump relocation Site (3,500-foot site) is located approximately 1,270 feet downstream from the end of the existing riprap along a section of the left (east) bank that has eroded approximately 200 feet back from the end of the riprap over time. As part of Alternative 2 site relocation (3,500-foot site), the left bank will be re-aligned to remove the spur created by the existing riprap. A Phase III Bank Realignment model was developed to represent the realignment of the left bank and the model results were used to develop rating curves at the Alternative 2 site and to evaluate any potential changes at the City of Chico outfall that is located upstream of the spur.
3. The 9-dike channel configuration alternative (MEI, 2005) was incorporated into the Phase III Setback Levee model. The Phase III Dike-Setback Levee model was run at the 50- and 100-year peak flow events and the model output was compared to Phase III Setback Levee model output to evaluate any potential hydrodynamic impacts of the dikes on the Hamilton City setback levee.

4. The Phase III Setback Levee model was modified to represent a conceptual gravel stockpile (100,000-ton) located on the right (west) overbank opposite the M&T Pumps. This Phase III Stockpile model was run at the 50- and 100-year peak flow events and the model output was compared to Phase III Setback Levee model output to evaluate the hydrodynamic impacts of the stockpile. An incipient motion analysis was performed to determine the mobility of the gravel located around the base of the stockpile. In addition, the Phase III Setback Levee model was modified to represent a gravel stockpile located immediately beside the river on the right (west) bank adjacent opposite the M&T Pumps in order to evaluate the potential of the river to entrain the bank stockpile and return the gravels to the river. This Phase III Bank-edge Stockpile model was run at flows of 90,000 and 145,000 cfs and an incipient motion analysis was performed to determine the mobility of the gravel in the bank-edge stockpile

Based on the results of the analyses the following were concluded:

1. The Hamilton City Setback Levee Project will not affect hydrodynamic conditions at the M&T Pumps, City of Chico outfall or at the two alternative pump sites
2. The influence of the nine spur dikes configuration is limited to the area in the vicinity of the dikes and the hydrodynamic effects of the dikes do not extend across to the Setback Levee or up to River Road.
3. The predicted depth-discharge rating curves showed that the flow depths are the largest at the existing M&T pumps compared to the other sites, over the range of modeled flows from 5,000 cfs to the 100-year peak flow event. The flow depths at the City of Chico outfall and the Alternative 1 site are approximately 1.0 feet less compared to the M&T Pumps site over the range of modeled flows. The flow depths at the Alternative 2 site are approximately 4 feet less than at the M&T pumps over the range of modeled flows. The predicted velocity-discharge curves indicate that the velocities at the M&T Pumps are highest in the range of flows from 5,000 to 109,322 cfs (December 2005 peak flow), compared to the other sites, and then remain fairly constant up to the 100-year peak flow event. The velocities at the City of Chico outfall and Alternative 1 sites are similar over the range of flows from 5,000 to 134,638 cfs. In general, the velocities at the Alternative 2 site are lower compared to the other sites over the entire range of modeled flows. The shear stress-discharge rating curves at all four sites show similar trends to the velocity-discharge rating curves. The shear stress values at the M&T Pumps steadily increase up to 90,000 cfs and then flatten out at higher discharges. The shear stress values at the City of Chico outfall and the Alternative 1 site steadily increase up to 134,638 cfs and then flatten out slightly at higher discharges. The shear stresses at the Alternative 2 site steadily increases up to 50,000 cfs and then flatten out slightly at higher discharges. Comparison of the depth-discharge, velocity-discharge and shear-stress discharge rating curves at the City of Chico outfall for the Phase III Setback Levee and Phase III Realignment scenarios shows that there will be very little change in hydrodynamic conditions at the City of Chico outfall. The design criteria for the City of Chico outfall were not available to evaluate the hydrodynamic effects of the bank realignment on the outfall, however, the very small changes in hydrodynamic conditions indicates that the bank realignment will not likely affect the City of Chico outfall.
4. The hydrodynamic effects of the stockpile located on the west overbank do not extend to the City of Chico outfall or to the two Alternative pump sites. The incipient motion analysis indicated that the stockpile is stable at the 50-year peak flow event. The incipient motion analysis of the bank-edge stockpile indicated that the toe of the stockpile will erode at the downstream end at 90,000 cfs (2-year return interval) and 145,000 cfs (approximately 10-year return interval); however, relatively little sediment would be eroded from the stockpile

compared to the total volume of the stockpile during a significant flood event, and that significant erosion would only take place during relatively infrequent flood events.

1 INTRODUCTION

In 1997, the M&T/Llano Seco Pumping and Fish Screen Facility was moved downstream from Big Chico Creek to the east bank of the Sacramento River just downstream from the mouth of the creek at RM 192.75 (**Figure 1.1**). The pumps had previously been located in the creek about 0.5 miles upstream from the confluence. Since 1997, geomorphic changes have occurred in the Sacramento River channel that pose a significant risk to the continued operation of the facility, including erosion and lateral migration of the west bank of the river and growth of the large gravel bar that is located on the east side of the river at the mouth of Big Chico Creek, just upstream from the intake.

Based on the aerial photographic comparisons and survey measurements, the west bank of the Sacramento River eroded by up to 330 feet just upstream from the intake between 1996 and 2006. In 2000 and 2008, approximately 189,000 and 84,500 yd³ of material, respectively, were dredged from the gravel bar as a short-term solution to limit sedimentation at the pump inlet and a longitudinal stone toe was emplaced along approximately 1,200 feet of the west bank in October 2007. Previous work detailed the historic migration of the river in this area and identified the hydraulic factors that are responsible for creation and continued development of the gravel bar and the resulting sedimentation problems at the M&T Pump intake (Harvey et al., 2004). The Steering Committee report (2006) suggested that a possible solution to the problem could include a series of eight or nine spur dikes along about 2,500 feet of the west bank opposite and upstream from the pump intake that would force the flows back to the east, preventing further lateral erosion and potentially increasing flow velocities sufficiently to prevent, or at least limit, deposition in the vicinity of the intake.

As part of Phase I of the project, Mussetter Engineering, Inc. (MEI, 2006) conducted two-dimensional (2-D) hydrodynamic and sediment-transport modeling to determine if spur dikes installed along the west bank of the river upstream of the M&T Ranch pumping plant inlets and fish screens (RM 192.75) could create hydrodynamic conditions that would permit sustainable operation of the pumps for the next 40 years (MEI, 2006).

Three specific questions addressed by the MEI (2006) study were:

1. Will the spur dikes prevent further erosion of the west bank of the river that has retreated over 330 feet between 1996 and 2006, which is the primary cause of the problems at the M&T Pumps,
2. Will the spur dikes stabilize the bank-attached bar on the east bank that has migrated downstream towards the pump inlets as the west bank has retreated, and
3. Will the spur dikes create sufficiently high velocities and shear stresses in the vicinity of the pumps during the range of flows when pumping generally takes place (4,000 to 14,000 cfs) to prevent sand accumulation around the fish screens and pump inlets?

Two-dimensional (2-D) hydrodynamic models (RMA2) (MEI, 2006) were developed to represent the December 2005 bathymetry and topography of the site. Models were developed and run for a range of flows from 5,000 to 90,000 cfs for the following scenarios:

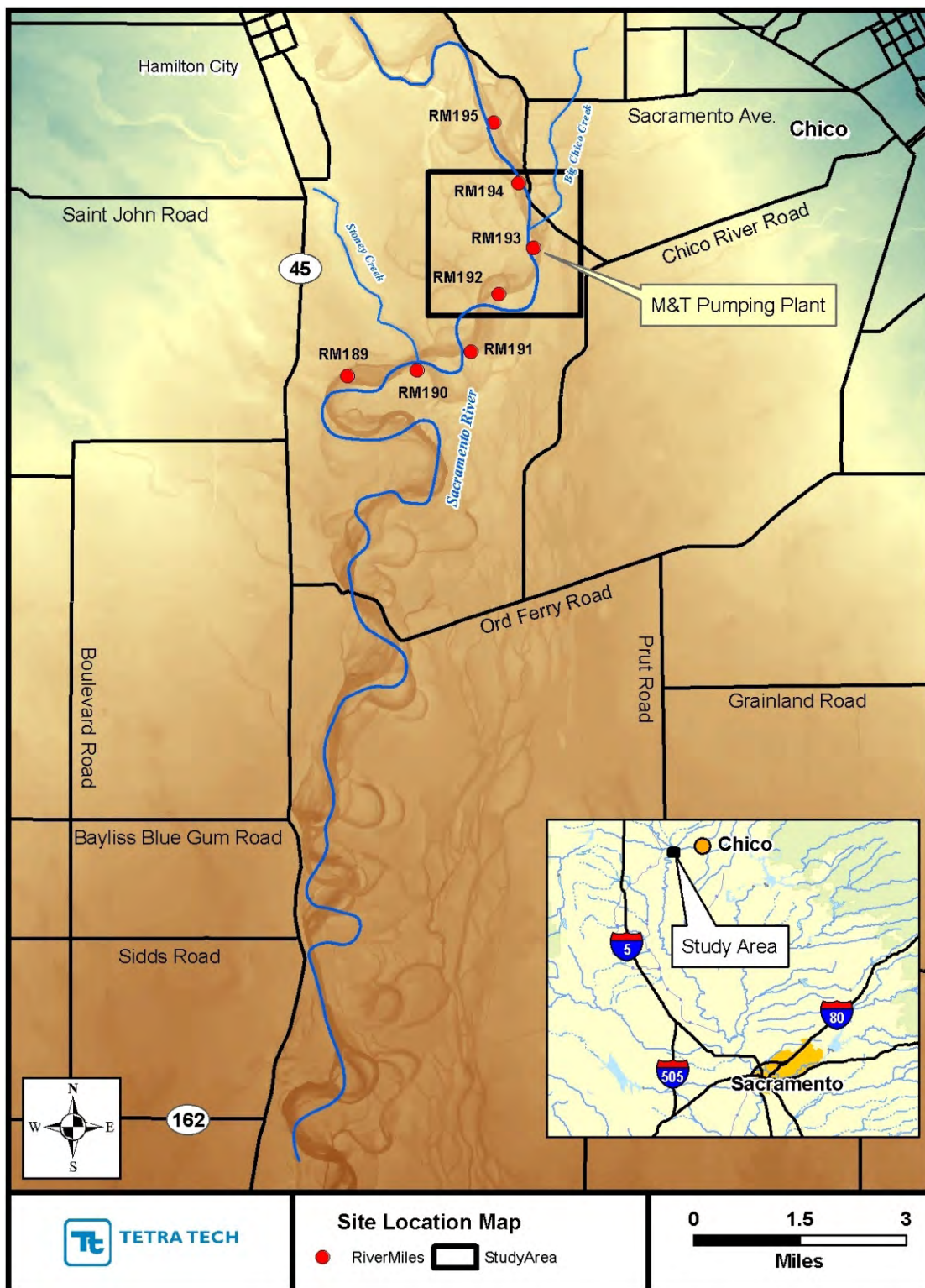


Figure 1.1. Location of the Sacramento River and the study site.

1. 2005 channel alignment and geometry for a baseline condition (Scenario 1),
2. An 8-dike configuration with dike height at two-thirds bank height (Scenario 2),
3. A 9-dike configuration with dike height at two-thirds bank height (Scenario 3), and
4. An extended 9-dike configuration with the lower three dikes raised to full bank height (Scenario 4).

Incipient motion and sediment-transport analyses were conducted with output from the 2-D models and an average bar sediment gradation with a median (D_{50}) size of 39 mm and a D_{84} size of 60 mm that were developed from three pebble counts conducted on the bank-attached bar in December 2005. A sand size of 1 mm was used in the analysis of deposition potential around the fish screens and pump inlets.

Based on the results of the analyses the following were concluded:

1. All of the spur dike configurations will prevent further erosion of the west bank,
2. All of the spur dike configurations will prevent further downstream migration of the bank-attached bar located on the east bank upstream of the M&T Pumps, and
3. Only the extended and raised 9-dike configuration (Scenario 4) will prevent sand accumulation at the pump inlets during the range of river flows when pumping typically occurs.

Based on the review of the MEI (2006) report, the Steering Committee were concerned that the downstream boundary of the Phase I 2-D model was not located sufficiently far downstream to ensure that the downstream boundary effects were not influencing the hydrodynamic results in the vicinity of the M&T Pump. The Steering Committee recommended that additional Phase II 2-D modeling be performed, to evaluate the hydrodynamic conditions at the downstream boundary of the Phase I 2-D models. The Steering Committee also recommended that the 8- and 9-dike configurations be incorporated into a coarser scale 2-D model of the Butte Basin that extends from RM174 to RM212 (USACE, 1997) to evaluate the hydrodynamic impacts of the proposed spur dikes on flood-flow distributions between the river and the Butte Basin. In addition, the Steering Committee recommended that the Butte Basin model be modified to incorporate the likely future channel conditions (50 years) following removal of identified revetments as predicted by meander modeling (Larsen, 2008) in order to evaluate the effects of river meandering on the stage-discharge relations at the M&T overflow weir.

Based on the results of the Phase II analyses, the following were concluded:

1. Comparisons of the hydraulic results of the Extended models with the Original models for the without dikes, and 8- and 9-dike conditions, indicated that the boundary effects of the Original models were negligible and that the downstream boundaries of the Original models were suitably located to minimize boundary effects; therefore, the results of the Original models were not affected by the length of the models.
2. Comparison of the hydraulic results between the without-dikes (baseline) conditions and 9-dike configuration at the 100-year peak flow event (370,000 cfs) indicated that the 9-dike conditions will increase the velocities over the attached bar by approximately 0.5 ft/s

and increase the water-surface elevations upstream of the dikes by approximately 0.15 feet for a distance of 3,200 feet.

3. Comparison of the hydraulic results of the three channel migration scenarios with the existing condition indicated that removal of the revetments at both Phelan Island and Golden State Island (Scenario 3) will have the largest effect on stage and discharge at the M&T weir. The stage at the weir is predicted to increase by 0.2 feet for both the 50- and 100-year peak flow events, and the discharge will increase by 5,800 (4-percent increase) and 8,300 cfs (5-percent increase) for the 50- and 100-year peak flow events, respectively.

Based on review of the Phase II analysis, MEI (2008) report, the Steering Committee further recommended that additional Phase III 2-D modeling be performed to evaluate the following:

1. Relocating the existing pumping plant at two alternative sites located 2,200 feet (Alternative Site 1) and 3,500 feet (Alternative Site 2) downstream from the existing site.
2. Investigate the impacts of the Hamilton City Setback Levee project (J-Levee) on the long-term solution at the M&T Pump alternatives and the City of Chico outfall, and conversely, investigate the long-term solution impacts of the M&T Pump alternatives on the Hamilton City Setback Levee project.
3. Determine hydraulic conditions in the reach to provide boundary conditions for the physical modeling performed by Colorado State University (CSU).

1.1 Scope of Work

The work performed for this Phase III study included the following tasks:

1. A topographic and bathymetric survey of the M&T reach between RM 192 and RM 193.5 was conducted in January 2010 and the data were used to update the Phase II 2-D hydraulic models (MEI 2008) and to provide geometry data to develop the physical model (CSU, 2011). [Note: the updated model is referred to as the Phase III model. (**Table 1.1**)]
2. The calibrated Phase III existing conditions (“without-dikes”) 2-D hydraulic model was incorporated into the coarser Hamilton City J-Levee model that extends from RM 192 to RM 202 (USACE, 2008); this model is referred to as the Phase III J-Levee model and it was calibrated to gage records and high-water marks measured during the December 2005 (109,322 cfs) and January 2006 (134,638 cfs) floods. The Phase III J-Levee models were run at the 50- and 100-year peak flow event and the model output was used to evaluate the hydrodynamic effects of the Hamilton City Setback Levee project on the M&T reach.
3. The calibrated existing conditions Phase III (“without-dikes”) 2-D hydraulic model was then incorporated into the coarser Hamilton City Setback Levee model that extends from RM192 to RM202 (USACE, 2008). The Phase III model was run for a range of discharges from 5,000 to 15,000 cfs and the Phase III Setback Levee model was run for a range of discharges from 50,000 cfs to the 100-year peak flow event. The Phase III Setback Levee model output at the 50- and 100-year peak flow events were compared to the Phase III J-Levee model output to evaluate the hydrodynamic impacts of the proposed Hamilton City Setback Levee on the M&T reach. The Phase III model and the Phase III Setback Levee models were used as the baseline models to compare with the other scenarios and they were used to evaluate the hydraulic conditions at the Alternative 1 site (2,200-foot site) and

at the City of Chico relocated outfall. The Phase III model and the Phase III Setback Levee models were then modified to represent the realignment of the east bank downstream of the City of Chico outfall that would be necessary for relocating the pump to the Alternative 2 site. The Phase III Bank Realignment models were used to evaluate the hydraulic conditions at the Alternative 2 site (3,500 feet) and to evaluate the changes in hydraulic conditions at the City of Chico relocated outfall resulting from the bank realignment.

4. The proposed 9-dike configuration was incorporated in the Phase III Setback Levee model and was run at the 50- and 100-year peak flow events. The output from the Phase III Dike-Setback Levee model was compared to the output from the Phase III Setback Levee model to evaluate the hydrodynamic impacts of the proposed spur dikes on the Setback Levee project, and conversely, the hydrodynamic effects of the Setback Levee project on the M&T spur dikes.
5. The Phase III Setback Levee model was also modified to represent the inclusion of a 100,000 ton gravel stockpile on the west bank of the river opposite the M&T Pump. The Phase III Stockpile model and the Phase III Setback Levee model output were compared to evaluate any change in hydrodynamic conditions in the vicinity of the M&T Pump. An incipient motion analysis was conducted to determine the mobility of the gravel in the stockpile. In addition, the Phase III Setback Levee model was also modified to represent the inclusion of a gravel stockpile located on the edge of the west bank of the river opposite the M&T Pump to determine if the hydrodynamic conditions are sufficient to mobilize the bank-edge stockpile.

1.2 Authorization

This study was carried out by Tetra Tech (dba Mussetter Engineering, Inc.) (Tt-MEI) under a contract agreement with Ducks Unlimited. Tt-MEI staff who contributed to the work included:

- Dr. Robert A. Mussetter, P.E., Principal Engineer
- Dr. Michael D. Harvey, P.G., Principal Geomorphologist
- Mr. Dai B. Thomas, P.E. (CO), Staff Engineer
- Mr. Erich Schwaller, Field Technician
- Mr. Matt Iman, GIS Specialist

Stage	Initial Model	Action	Final Model	Comment on Final Model
Model development	"MEI Phase II (No Dikes) Model" (2006)	Updated with 2010 geometry	"Phase III Model"	"Phase III Model" calibrated to measured WSEL's during 2010 survey. Model used to provide baseline conditions at low flows (5,000 to 15,000 cfs).
	"Phase III Model"	Incorporated into Corps "50- and 100-year J-Levee Models"	"Phase III J-Levee Model"	Both the 50- and 100-year "J-Levee Setback Levee models" were developed. To avoid confusion, both models are referred to as the "Phase III J-Levee Model". Model calibrated to Dec2005/Jan2006 HWMs and gage recordings.
	"Phase III Model"	Incorporated into Corps "50- and 100-year Setback Levee Models"	"Phase III Setback Levee Model"	Used as baseline model to compare the other model results with. Both the 50- and 100-year "J-Levee Setback-Levee models" were developed. To avoid confusion, both models are referred to as the "Phase III Setback Levee Model". The 50-year model is used to model flows from 50,000 cfs to the 50-year event. The 100-year model is used to model just the 100-year event.
Evaluate alternative pumping plant locations			"Phase III Model"	Used to model flows from 5,000 to 15,000 cfs. Model results used develop rating curves at M&T Pump, City of Chico and Alternative Site 1.
			"Phase III Setback Model"	Used to model flows from 50,000 cfs to 100-year peak flow event. Model results used develop rating curves at M&T Pump, City of Chico and Alternative Site 1.
	"Phase III Model"	Re-aligned left bank downstream from City of Chico Outfall	"Phase III Bank Realignment Model"	Used to model flows from 5,000 to 15,000 cfs. Model results used develop rating curves at City of Chico and Alternative Site 2.
	"Phase III Setback Model"	Re-aligned left bank downstream from City of Chico Outfall	"Phase III Bank Realignment Setback Model"	Used to model flows from 50,000 cfs to 100-year peak flow event. Model results used develop rating curves at City of Chico and Alternative Site 2.
Evaluate impacts of 9-dike geometry	"Phase III Setback Model"	Incorporated 9-dike geometry	"Phase III Dike-Setback Model"	Used to simulate 50- and 100-year peak flow events. Model results compared to baseline conditions.
Evaluate stockpile	"Phase III Setback Model"	Incorporated stockpile geometry	"Phase III Stockpile Model"	Used to simulate 50- and 100-year peak flow events. Model results compared to baseline conditions and used to evaluate the mobility of the stockpile.
	"Phase III Setback Model"	Incorporated bank-edge stockpile geometry	"Phase III Bank-edge Stockpile Model"	Model results used to evaluate mobility of the bank-edge stockpile at 90,00 and 145,000 cfs.

2 HYDROLOGY

The analysis of the discharge regime in the study reach that was performed by MEI (2008) was updated for this study by adding data collected since 2008 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.

Data used to update flood-frequency curve for the post-Shasta Dam (1946-2011) period at the Hamilton City gage (**Figure 2.1, Table 2.1**) included the provisional peak discharge of 102,528 cfs that was recorded on March 21, 2011. As noted in the previous analysis, 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 2.1 indicates that the 1.5- and 2-year recurrence interval peak discharges are about 70,900 and 90,000 cfs, respectively. (The previous analysis indicated peak discharges for these recurrence intervals of 71,000 and 90,000 cfs, respectively). In addition, the 50- and 100-year peak flow events were revised to 237,800 and 275,900 cfs, respectively, based on the reported values in the Hamilton City J-Levee analysis (UASCE, 2008). [The MEI (2008) analysis indicated peak discharges for these recurrence intervals of 157,800 and 374,100 cfs, respectively]. The 50- and 100-year peak flows from the Hamilton City J-Levee analysis were used for consistency with USACE analyses (USACE, 2008). The selection of the 50- and 100-year peak flow values used in the analysis will not affect the conclusion of the analyses, because the comparisons between the different scenarios are conducted using the same discharges for each peak flow event.

Table 2.1. Peak discharges and associated recurrence intervals derived from the flood-frequency curve (Figure 2.1) at the Hamilton City gage.	
Peak Discharge (cfs)	Return Period (years)
59,100	1.2
70,900	1.5
90,000	2
133,800	5
150,400	10
162,900	20
157,800	50
237,800 ^a	50
275,900 ^a	100

^aUSACE (2008)

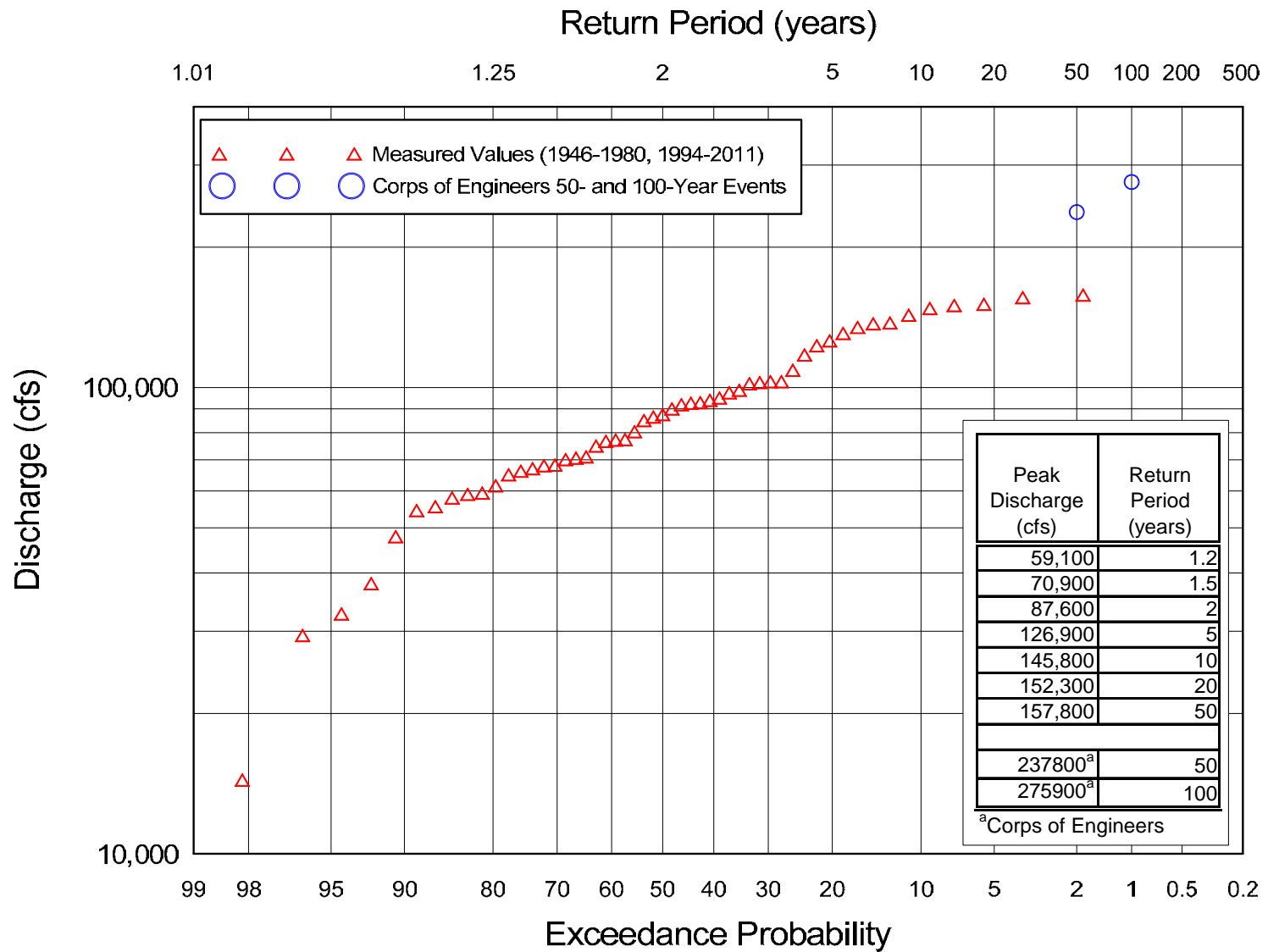


Figure 2.1. Post-Shasta Dam flood-frequency curve for Hamilton City gaging station.

The mean daily flow-duration curve for the Hamilton City gage was also updated using the available record of complete water years, WY1946 through WY1955, WY1957 through WY1980, from the USGS data, WY1997 through WY2000 and WY2010 from the CDEC data (**Figure 2.2**). The resulting curve indicates that the median flow (flow that is equaled or exceeded 50 percent of the time) at the gage is about 8,560 cfs, and the 10- and 90-percent exceedence flows are 22,430 and 5,425 cfs, respectively. Annual runoff past the Hamilton City gage during the 29-year period of complete water years varied from about 4.3M ac-ft in 1977 to about 18.5M ac-ft in 1974, and it averaged about 9.9M ac-ft per year (**Figure 2.3**).

The annual peak flows for the period of record are also shown in Figure 2.3 for comparison with the annual runoff volumes. The bar opposite Bidwell State Park, located at RM 193, likely first formed during the 1964 flood (Stillwater Sciences, 2001). The bar has continued to grow since 1964, and between 1995 and 2001, the bar migrated approximately 1,700 feet downstream to its current location. Relatively high-magnitude flood peaks and large flow volumes occurred in 1974, 1997, 1998 and 2006. The formation and migration of the bar is very likely related to the occurrence of these high-magnitude flows.

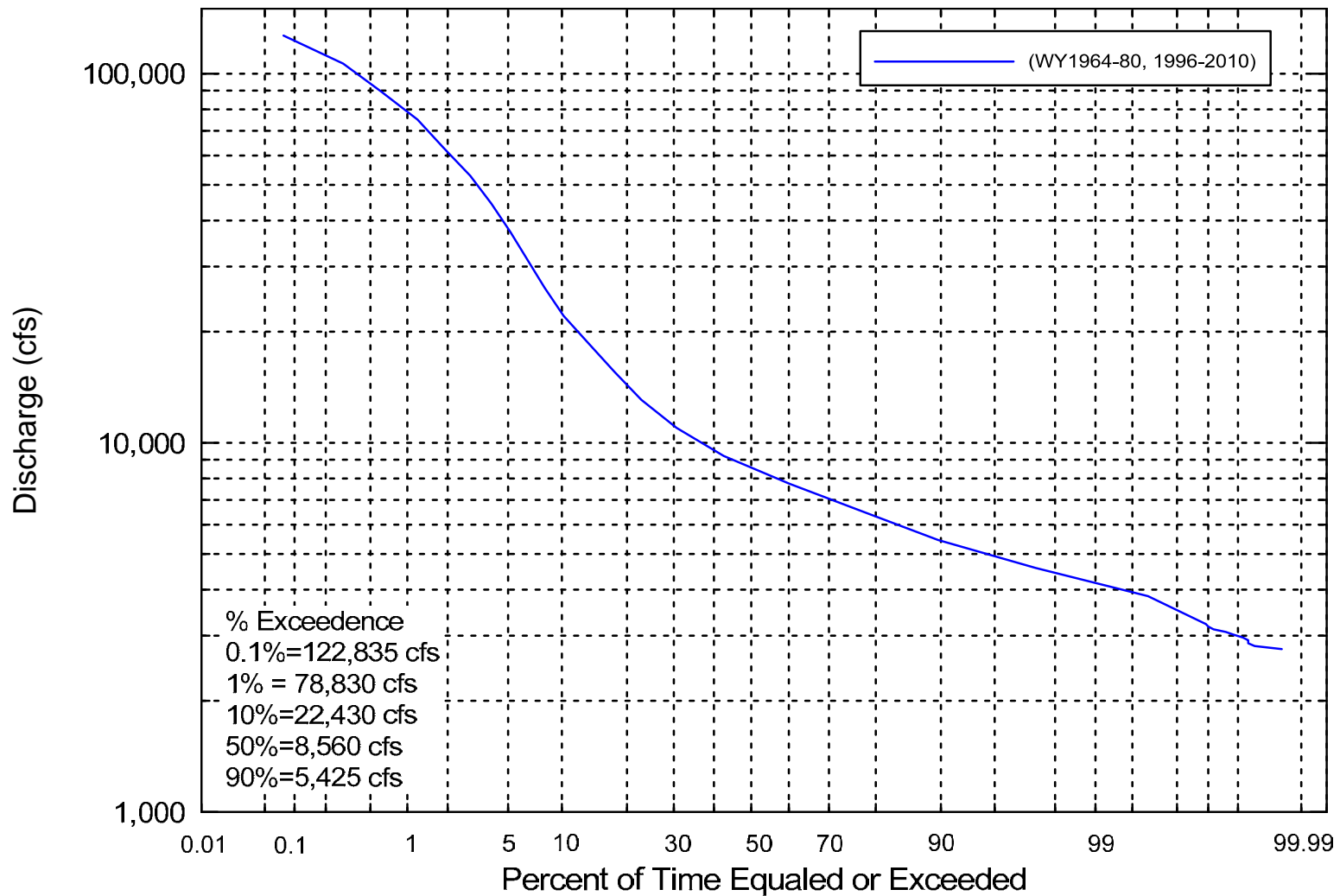


Figure 2.2. Post-Shasta flow-duration curve for Hamilton City gaging station.

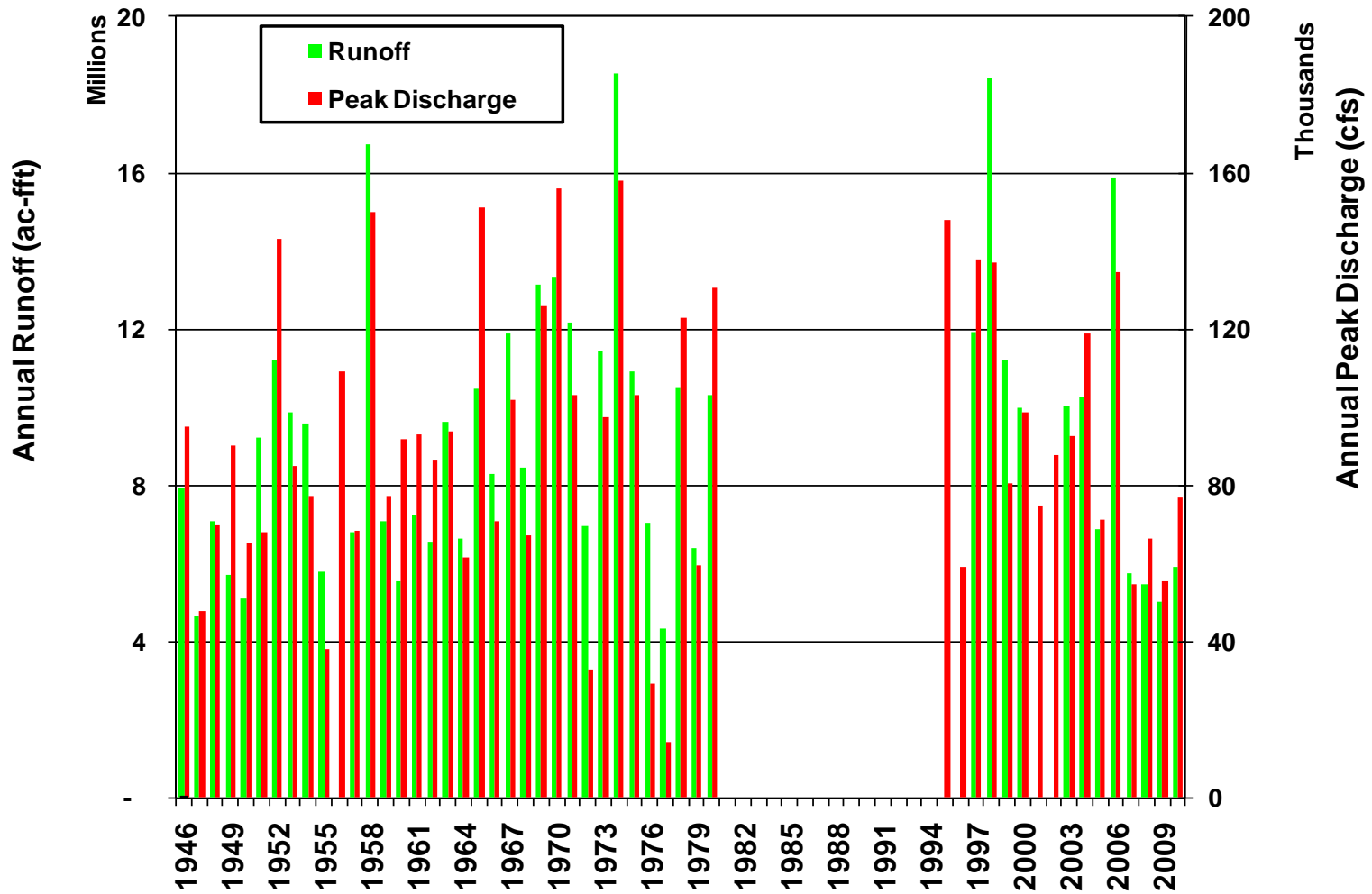


Figure 2.3. Annual runoff volumes and peak discharges at the Hamilton City gaging station from 1946 to 2010. No data are available for the 1981 to 1984 period and complete water years were not available for 1995, 1996, 2001 and 2002.

3 DATA COLLECTION

Tt-MEI conducted a hydrographic and topographic survey of the study reach in January 2010 for the purpose of obtaining topographic data that was used to: (1) to evaluate topographic changes within the study reach between 2010 and the previous survey that was conducted in May 2006, (2) develop the 2-D hydrodynamic models, (3) develop the physical model built at Colorado State University (CSU, 2011).

3.1 Hydrographic and topographic survey

The bathymetric and topographic survey was conducted between January 11 and 15, 2010, when the discharge in the river ranged from 4,500 to 13,500 cfs (**Figure 3.1**). The hydrographic survey was conducted on January 13, when the discharge at the Hamilton City gage was steadily increasing to a peak value of 13,962 cfs (January 13, 2010, 10pm) and on January 15 when the discharge was steadily receding (Figure 3.1). The flows on Big Chico Creek during the time of the survey were likely less than 450 cfs based on the flow measurements at the Chico Creek gage (California Data Exchange Gage ID. BIC).

Soon after completion of the survey, two flood events were recorded at the Hamilton City gage which had peak discharges of approximately 76,700 cfs (January 21, 2010) and 73,700 cfs (January 26, 2010). Both of these flood events had recurrence intervals of approximately 1.6 years. For context, the bankfull discharge in the reach is approximately 90,000 cfs.

Survey control for the project was obtained from the MEI survey that was conducted in May 2006. The May 2006 survey established two control points (**Table 3.1**) on the river side of the levee adjacent to the M&T Pump based on a GPS traverse from the National Geodetic Survey (NGS) control point "Wildlife". The horizontal datum is referenced to State Plane Coordinate System, North American Datum of 1983 (NAD83) (California, Zone 2) and the vertical datum is based on the North American Vertical Datum of 1929 (NAVD29).

Control Point	Easting (ft)	Northing (ft)	Elevation (ft) (NGVD 29)
Wildlife	6,571,599.36	2,385,582.95	135.0
MEI CP5	6,578,277.186	2379285.221	139.54
MEI CP6	6578277.365	2379309.401	139.46

To facilitate development of the model and interpretation of model results, a station line that represents the distance along the approximate centroid of the flow was developed, with the downstream end (Sta 0+00) located at the downstream boundary of the USACE Butte Basin 2-D model. (Note: the station line used in the MEI (2006) report had the downstream boundary located at Stony Creek.) Along this station line, the up- and downstream ends of the J-Levee model are located at Sta 1298+37 and Sta 479+33, respectively (**Figure 3.2**). The M&T pumping station is located at Sta 1101+20 on the left (east) bank of the river. **Table 3.2** summarizes the location of key points of interest along the stationline.

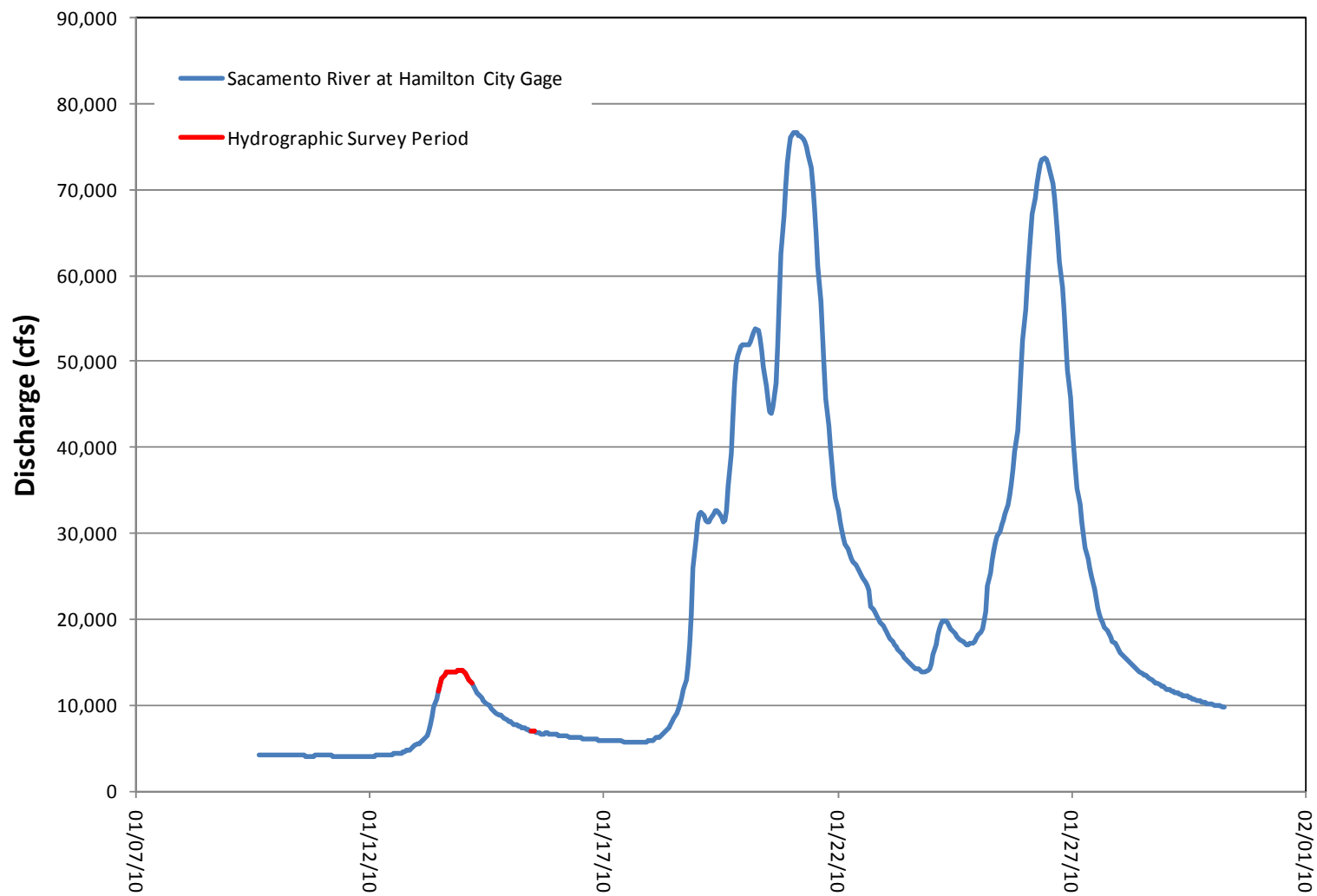


Figure 3.1. Flow hydrograph measured at the Sacramento River at Hamilton City gage. The discharge during the hydrographic survey period from ranged from approximately 13,000 cfs on January 13, to approximately 7,000 cfs on January 15, 2010.

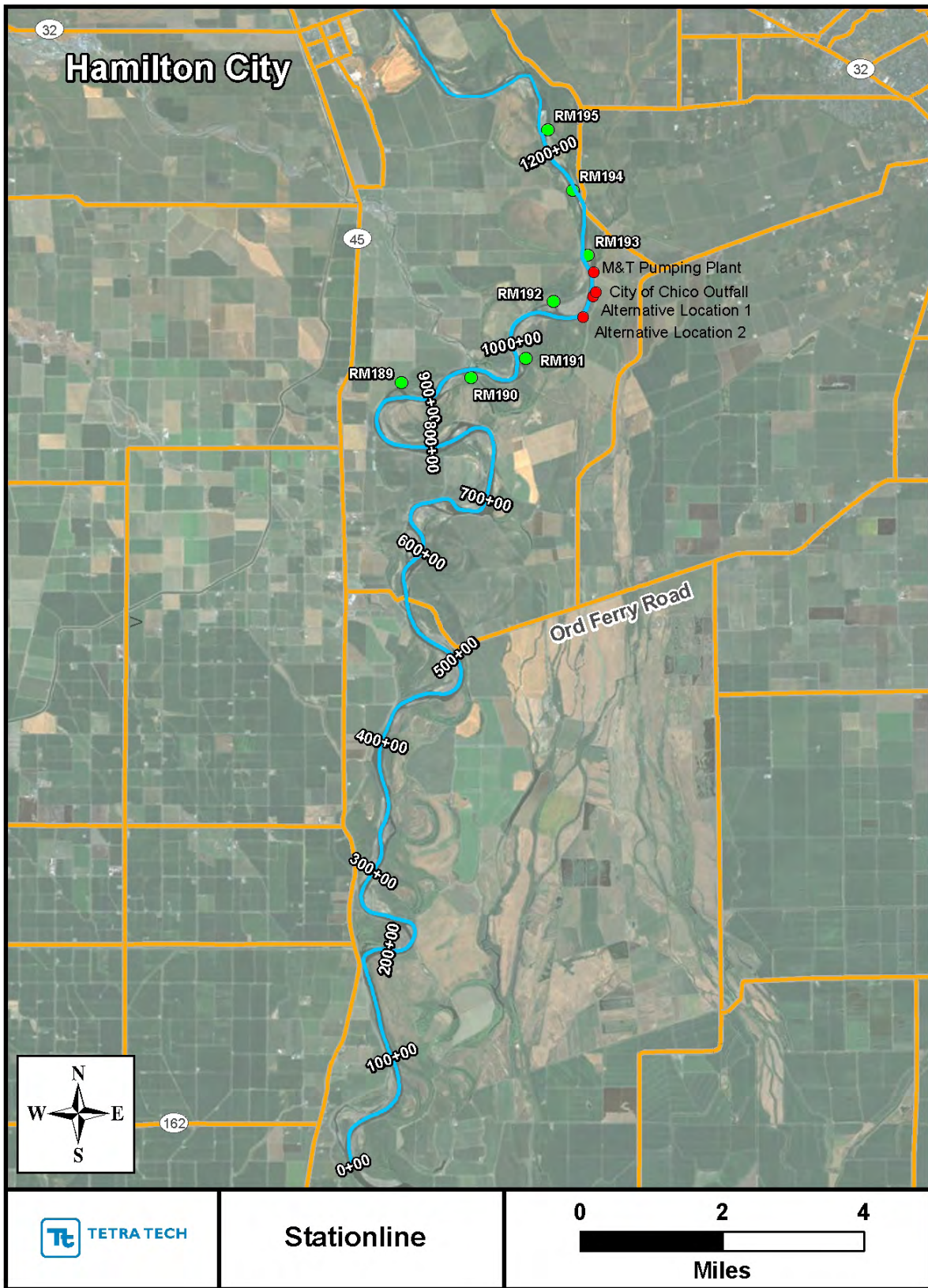


Figure 3.2. Project stationline.

3.3

Station	Description
479+33	Downstream end of Setback and J-Levee models
934+37	Stony Creek
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 Pump
1109+51	Downstream end of bank attached bar
1130+01	Upstream end of bank attached bar
1147+86	River Road
1298+44	Upstream end of Setback and J-Levee models

The hydrographic survey was conducted using conventional surveying methods in accordance with guidelines that meet the requirements of Class 2 Hydrographic surveys (USACE, 2002). Depth soundings were measured using an Innerspace 445 survey-grade fathometer with an 8-degree sonar transducer (± 0.1 -foot resolution) mounted on the side of the boat. Horizontal and vertical positioning data were measured using a Trimble 4800 series RTK survey-grade GPS. The positioning accuracy of the GPS data is approximately 0.05 feet both vertically and horizontally. The depth and positioning data were recorded using Coastal Oceanographics Hypack Lite software on a portable computer. The hydrographic survey was conducted by surveying transects across the river at approximately 150-foot spacing throughout the reach. The topographic survey of the above-water portion of the site, including the attached gravel bar upstream of the M&T Pump, top of river banks, portions of the excavated mid-channel bar and limited portions of the overbanks, was also conducted using the Trimble 4800 series rover unit. A digital terrain model (DTM) and a 2-foot contour map of the channel bed were developed from the hydrographic topographic survey data using ArcGIS software.

3.2 Comparison of Bed Change

The DTMs developed from the 2005/2006 and 2010 bathymetric surveys were compared in the ArcGIS software and an isopach map of the differences was developed (**Figure 3.3**). Between Sta 1037+00 and about Sta 1060+00 the left bank of the river eroded and retreated and is represented on Figure 3.3 as about 20 to 25 feet of degradation. For the bulk of the reach between Sta 1020+00 and Sta 1150+00 there was little change in the bed topography (-5 to +5 feet). However, degradation of between 5 and 10 feet is shown on Figure 3.3 between Sta 1110+00 and Sta 1124+00 and this represents the bar dredging that was completed in Fall 2007. Lack of high flows since the 2007 dredging has resulted in little or no deposition on the gravel bar, and in fact at the time of the survey, the constructed containment berm was still in place. In contrast, between Sta 1097+00 and about Sta 1105+00 there had been between 5 and 15 feet of aggradation. This region of aggradation is downstream of the gravel bar and is in the center-left portion of the channel opposite the M&T Pumps. Some aggradation also occurred along the left (east) side of the channel in the vicinity of the relocated City of Chico wastewater outfall (about Sta 1085+00).

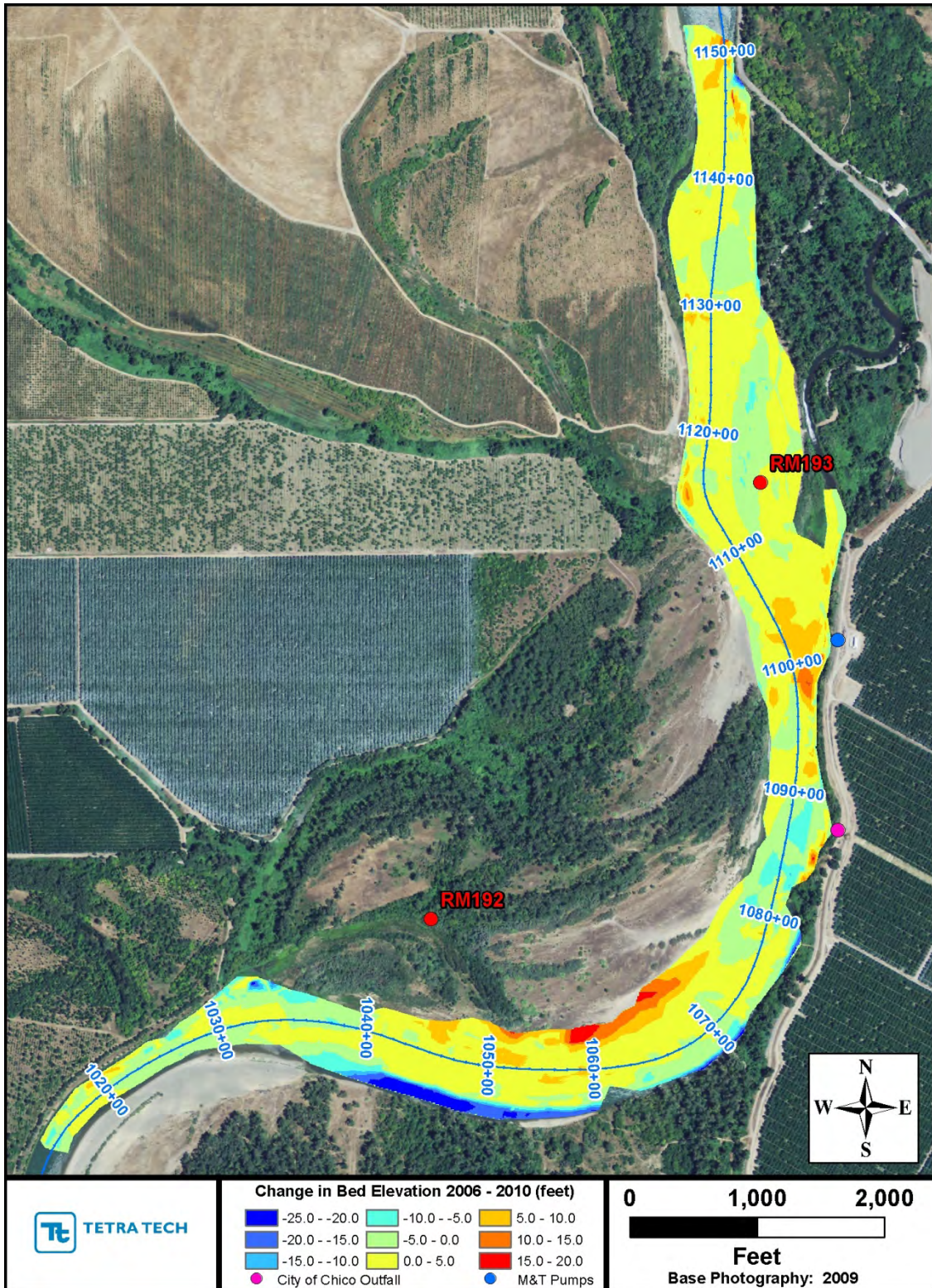


Figure 3.3. Change in bed elevation between the 2006 and 2010 surveys.

4 TWO-DIMENSIONAL HYDRODYNAMIC MODEL ANALYSIS

Two-dimensional (2-D) hydrodynamic models of the Sacramento River in the vicinity of the M&T pumping plant were developed to:

1. evaluate the hydrodynamic impacts, if any, of Hamilton City Setback Levee project on the M&T Reach,
2. evaluate the hydraulic conditions at the alternative pump sites located 2,200 (Alternative Site 1) and 3,500 feet (Alternative Site 2) downstream of the existing site,
3. evaluate the hydrodynamic impacts, if any, of the proposed spur dikes on Hamilton City Setback Levee project, and conversely, the hydrodynamic effects, if any, of the Setback Levee project on the M&T spur dikes, and
4. evaluate the hydrodynamic impacts of locating a proposed dredge stockpile on the west bank of the river, just downstream of the M&T Pump.

The Corps of Engineers developed the Hamilton City J-Levee models (the USACE developed separate meshes to represent the 50- and 100-year peak flow events) to represent the existing conditions of the reach (RM 192 to RM 202) (USACE, 2008). The J-Levee models were developed based on 1995-1997 topographic data and they were calibrated to gage measurements at the Ord Ferry gage [California Data Exchange (CDEC) gage ORD] and Hamilton City gage (CDEC gage HMC) and to high-water marks measured during the 2005-2006 flood. Following calibration of the J-Levee models, they were modified to incorporate the proposed Setback Levee and training dike that is located near Hamilton City (**Figure 4.1**). The Setback Levee is approximately 12,000 feet long and is set at the elevation of the 320-year flood profile. Downstream from the Setback Levee, there is a training levee that is separated into two sections. The first section is approximately 3,000 feet long and is set at the 100-year flood profile elevation and the second section is approximately 3,400 feet long and the elevation is set at the 20-year flood profile elevation (the elevation of the second section of the training levee is approximately 2.1 feet lower than the first section). In this report, the Setback Levee and Training Levee were considered as one levee and are referred to as the Setback Levee. The existing J-Levee will be removed following construction of the Setback Levee and this has been represented in the Corp's Setback model (USACE, 2008) and the Phase III Setback Levee models.

The USACE conducted their analyses using the RMA-2V hydrodynamic model. A significant effort was made to run the original USACE models using the RMA-2V software; however, numerical instabilities were encountered and it was not possible to obtain a model solution. The USACE was not able to provide the hotstart or the solution files for the RMA-2V models. The hotstart files are used in the hydraulic model to re-create the solution file and avoid the time consuming process of "spinning down" the model. In addition, the incorporation of the more detailed Phase III models into the J-Levee and Setback Levee models, significantly increased the model complexity, and it was apparent that the RMA-2V model would result in continued numerical instabilities that would be too time consuming to overcome. Therefore, the RMA-2V models were converted to SRH-2D models.

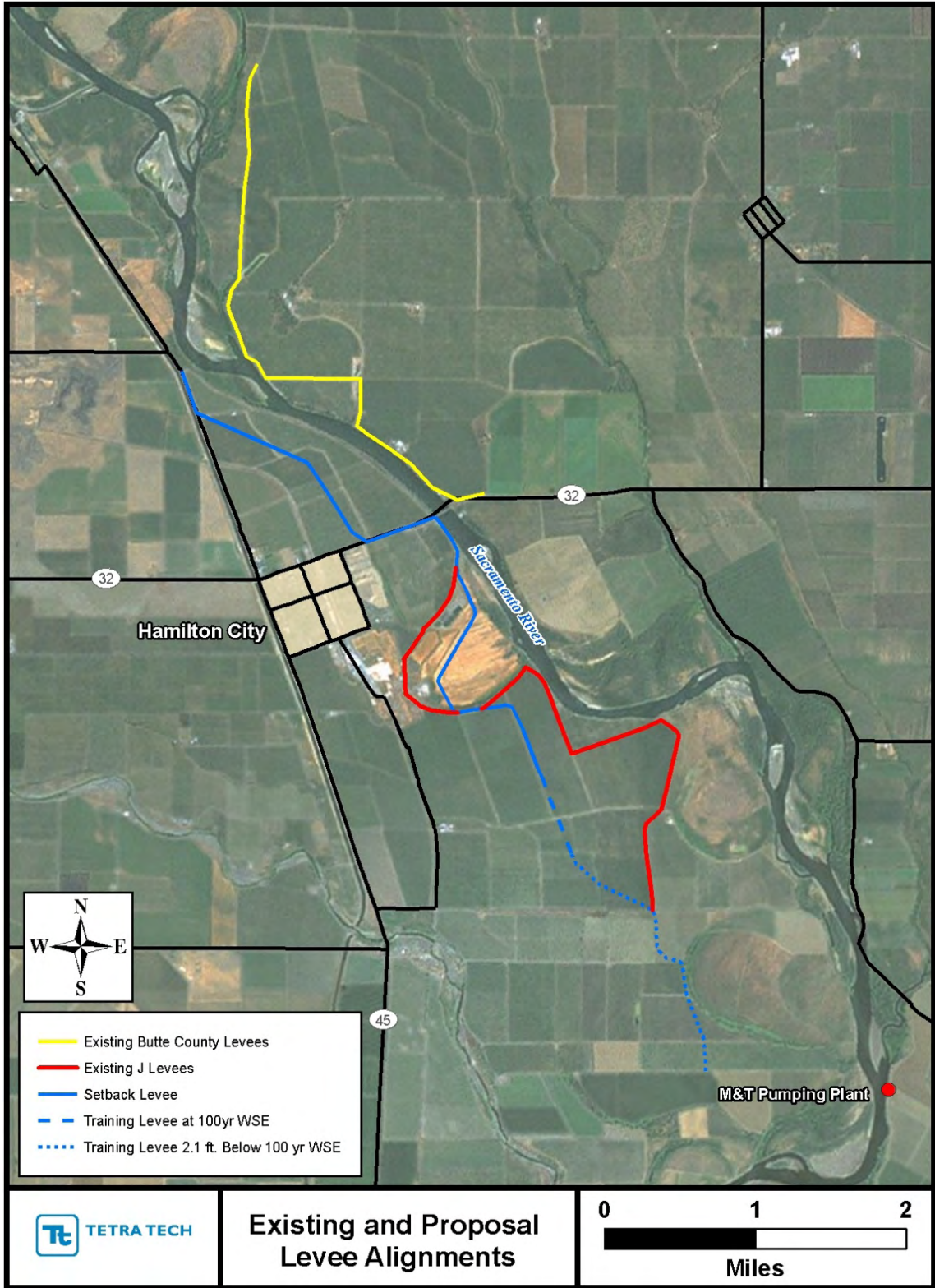


Figure 4.1. Location of the existing J-Levee and proposed Setback Levee.

The subsequent modeling of the alternatives was conducted using SRH-2D, Version 2.0 (BOR, 2008) with Version 10.1 of the Aquaveo Surface Water Modeling System (SMS) graphical user interface (Aquaveo, 2010). SRH-2D is a depth-averaged, finite volume, hydrodynamic model that computes water-surface elevations and horizontal velocity components for subcritical, free-surface flow in 2-D flow fields.

SRH-2D was designed for far-field problems in which vertical accelerations are negligible and velocity vectors generally point in the same direction over the entire depth of the water column at any instant in time. SRH-2D was chosen for this project because it is a generally accepted 2-D model that provides more accurate prediction of the flow patterns in the vicinity of the dikes and pump intake, it is numerically robust, and because it has been successfully applied on other similar river systems.

4.1 Model Development

To evaluate the hydraulic conditions in the reach for the various scenarios, 2-D models were developed to represent the existing conditions (J-Levee model), the proposed Setback Levee conditions, and the other scenarios (Table 1.1). Although the Setback Levee has not been constructed yet, it is likely that it will be constructed before a permanent, long term solution is emplaced for the M&T/Llano Seco pumps and fish screens, and therefore, the Setback Levee model was used as the baseline model for comparison purposes with the other scenarios. The 2-D models used in the analyses were developed in the following order:

1. The existing conditions Phase II model (MEI 2008) that was based on 2006 topography was updated with the 2010 survey data and was validated using water-surface elevations measured during the 2010 survey. This model is referred to as the existing conditions Phase III model.
2. The existing conditions Phase III model was incorporated into the USACE J-Levee model and was validated with high-water marks measured during the 2005-2006 floods. This model is referred to as the Phase III J-Levee model.
3. The existing conditions Phase III model was incorporated into the USACE Setback Levee model. This model is referred to as the Phase III Setback Levee model.
4. The Phase III Setback Levee model was modified to represent the proposed 9-dike scenario. This model is referred to as the Phase III Dike-Setback Levee model.
5. The Phase III Setback Levee model was modified to represent the gravel stockpile scenario. This model is referred to as the Phase III Stockpile model.

4.1.1 Topographic Data

The existing conditions Phase II hydraulic model (MEI, 2008) and the COE J-Levee and Setback Levee models were converted from RMA-2V to SRH-2D. The meshes for the RMA-2V and SRH-2D models were developed using the Aquaveo Surface Water Modeling System (SMS) graphical user interface (Aquaveo, 2010). Both the SRH-2D and the RMA-2V meshes are composed of triangular and quadrilateral elements with corner nodes (the RMA-2V model also includes side nodes) that represent the planform geometry and channel topography of the modeled reach. The SMS software was used to convert the RMA-2V models to SRH-2D models.

The existing conditions 2-D hydraulic model developed for Phase II analyses (MEI, 2008) was developed using topographic data that were derived from the 1996 mapping by Ayres Associates and from the May 2006 bathymetric and topographic survey (MEI, 2008). The Phase II model was updated for the Phase III analyses by replacing the in-channel topography with the 2010 data and re-contouring the banks and near-river overbanks to reflect the 2010 channel alignment. The Phase III mesh was then incorporated into the coarser 50- and 100-year J-Levee model.

The USACE J-Levee analysis used separate hydraulic models for the 50- and 100-year events. The major difference between the two models is the extent of the models. The 100-year model has a larger model domain than the 50-year model in order to represent the greater amount of flow inundation at higher discharges. The mesh for the 50-year model contains approximately 32,200 elements and 26,607 nodes, and the mesh for the 100-year model contains approximately 33,000 elements and 27,200 nodes. To calibrate the J-Levee model to the December 2005 and January 2006 floods, the USACE applied lower Manning's n -values in certain areas of the overbank (namely in the areas delineated as orchard) in order to represent flow around the base of the trees; for the 50- and 100-year models, the USACE used higher n -values to represent flow through the tree canopy.

The original USACE J-Levee and Setback Levee models have a very coarse mesh resolution compared to the Phase III model and do not appear to adequately predict hydraulic conditions in the M&T reach at flow less than 50,000 cfs; therefore, the Phase III model was used to simulate flows up to 50,000 cfs and the J-Levee and Setback Levee models were used to simulate flows greater than 50,000 cfs.

4.1.2 Downstream Boundary Conditions

The downstream boundary condition for the 2-D model requires a specified water-surface elevation for each particular discharge that is being modeled. For the purposes of this analysis, rating curves at the downstream boundary of the Phase III and Setback Levee models were developed. The rating curve developed for the Phase II model (MEI, 2006) was applied to the Phase III model. For the J-Levee and Setback Levee models, the water-surface elevations for the 50- and 100-year peak flow events were obtained from the original Corp models. The rating-curve values for flows from 50,000 cfs to the December 2005 event (109,322 cfs) were estimated using a normal depth assumption and the geometry from the USACE 2-D models (**Figure 4.2**). The downstream boundary of the model is located approximately 12 miles downstream from the M&T Pumps; thus, any error associated with the assumed boundary conditions will have no effect on model results in the area of interest.

4.1.3 Material Properties and Model Validation

The SRH-2D model uses Manning's n -values to define boundary friction losses and a parabolic turbulence model was used to compute energy loss due to internal turbulence.

The Manning's n -values used in the original USACE 2-D models were applied to all the models. The USACE models used 24 different roughness zones (**Figure 4.3**) to represent the main channel, side channels and various overbank roughness zones. Several roughness zones used the same Manning's n -values and there are only 10 unique Manning's n -values that range from 0.025 to 0.16. A main-channel Manning's n -value of 0.035 was used for the entire channel and for the entire range of discharges. Manning's n -values for the overbanks ranged from 0.025

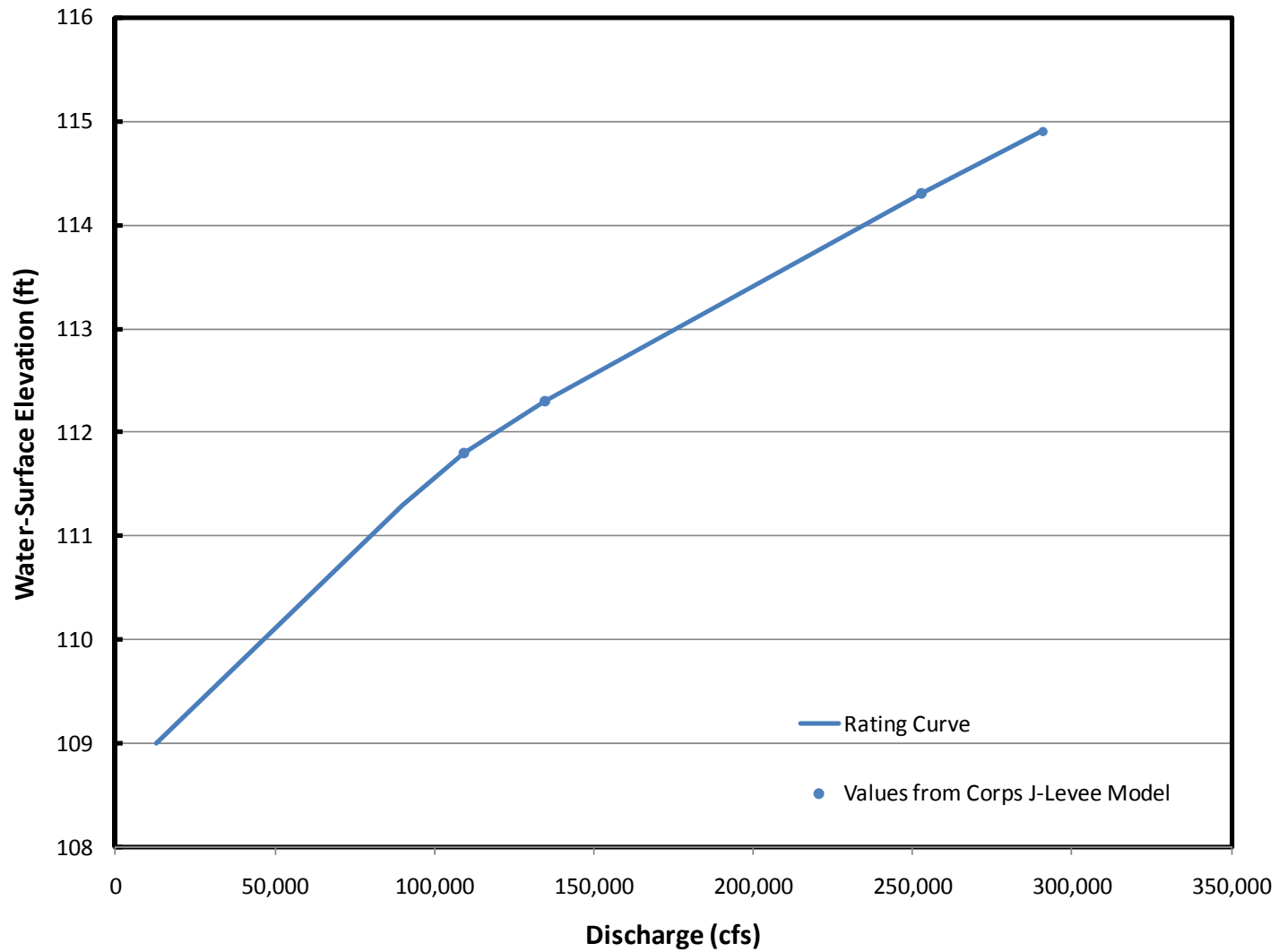


Figure 4.2. Stage-discharge rating curve develop for the downstream boundary of the Phase III J-Levee and Setback Levee models. Note: the stage-discharge rating curve values from the original USACE J-Levee were used to develop rating curve.

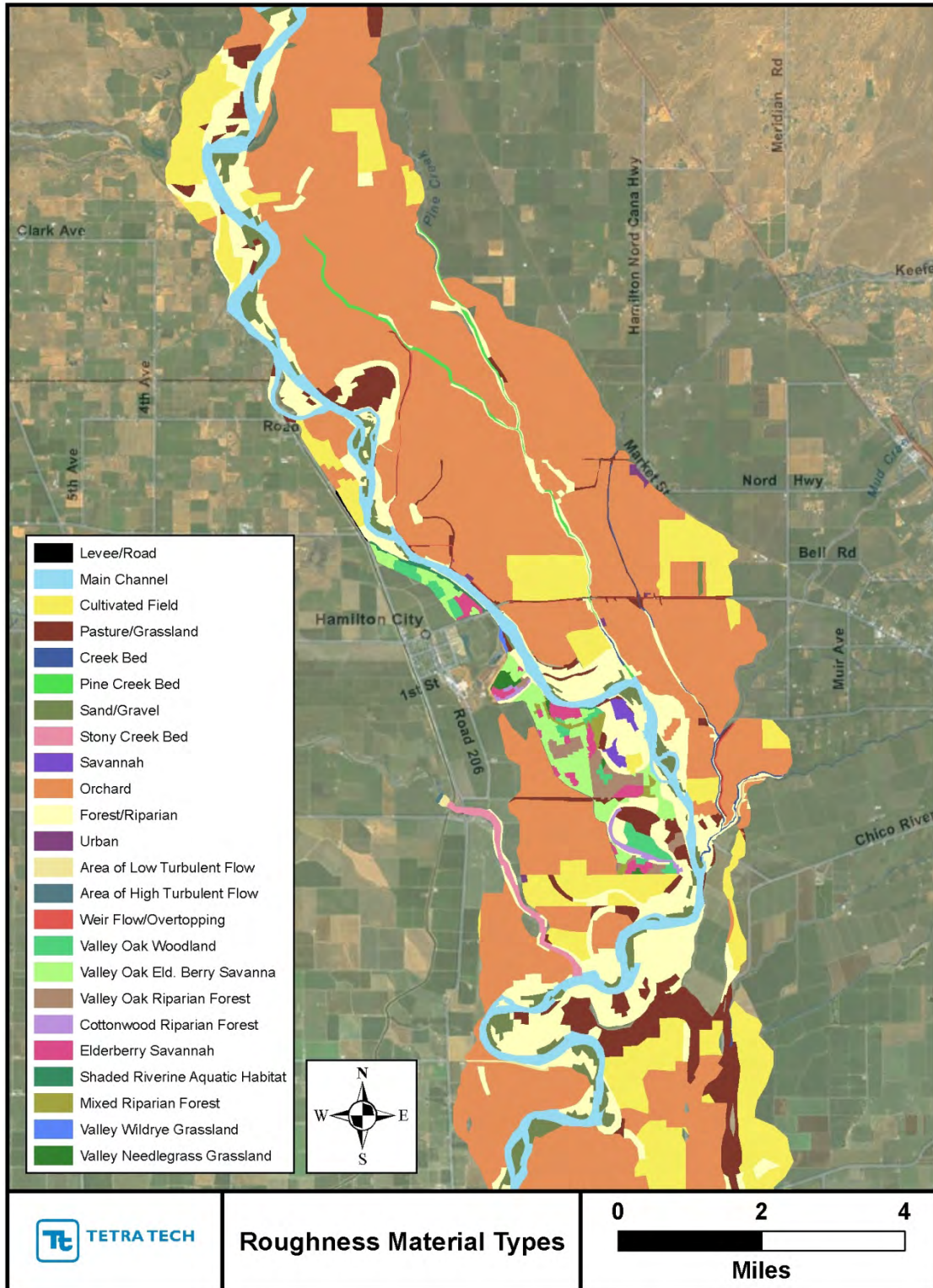


Figure 4.3. Distribution of the material types used to define the Manning's n -values in the modeled reach (USACE, 2008).

to 0.16 to reflect the roughness of the vegetation in these areas and the roughness regions were verified based on aerial photographs and field observations.

The agreement between the computed and measured water-surface elevations during the January 2010 survey when the discharge was approximately 13,000 cfs using these parameters is very good (**Figure 4.4**). The maximum difference between the measured and predicted water-surface elevation is 0.3 feet and it occurs in the area located downstream of the M&T Pump. At the M&T Pump, the predicted water-surface elevation is 0.1 feet higher than the measured values. There is an average difference of 0.04 feet between the measured and predicted values along the surveyed reach.

The J-Levee model was simulated for the December 2005 (109,322 cfs) and January 2006 (134,638 cfs) peak flood events and the predicted water-surface elevations were compared to the measured high-water marks reported by the USACE (2008) (**Figure 4.5, Table 4.1**). In the vicinity of the J-Levee, there was one high-water mark (HWM B) measured during the December 2005 flood and five high-water marks (1000-1004) measured in close proximity to each other during the January 2006 flood. In addition, peak water-surface elevations were reported at the Ord Ferry Gage (ORD) and the Hamilton City Gage (HMC).

In general, the predicted water-surface elevations are slightly higher than measured values during both the December 2005 and January 2006 peak flood events. However, in comparison to the original calibration values presented in the USACE report, the water-surface elevations predicted by the Phase III J-Levee model match the measured high-water marks closer than the original USACE calibration (Table 4.1). At the Hamilton City gage, the predicted water-surface elevation is approximately 0.7 feet higher than the gage record during the 2005 flood and 1.1 feet lower during the 2006 flood. At the Ord Ferry gage, the predicted water-surface elevation is approximately 1.09 feet higher than the gage record during the 2005 flood and 0.6 feet lower during the 2006 flood. In the vicinity of the J-Levee, the predicted water-surface elevations are 0.3 feet higher compared to measured values during the December 2005 flood, and are on average, approximately 0.2 feet higher during the January 2006 flood.

In summary, the Phase III model calibrates well to the water-surface elevations measured during the January 2010 survey and the Phase III J-Levee model calibrates to the high-water marks measured during the 2005-2006 flood. Because of the good calibration of the existing conditions models, it is assumed that the Phase III Setback Levee model can be used to evaluate the hydraulic conditions in the reach and for the various scenarios.

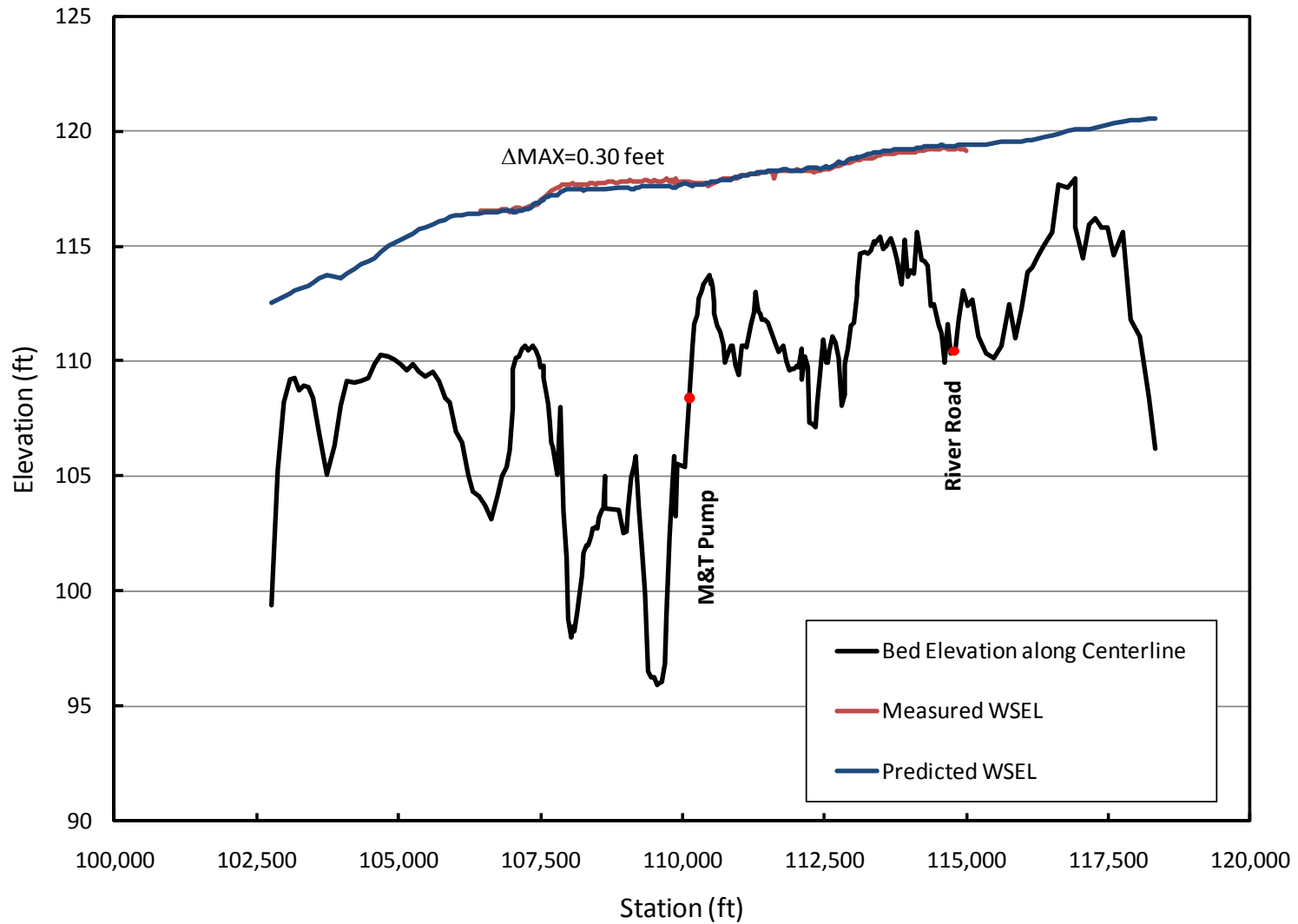


Figure 4.4. Comparison of the predicted and measured water-surface profiles from the January 2010 survey when the discharge was approximately 13,000 cfs.

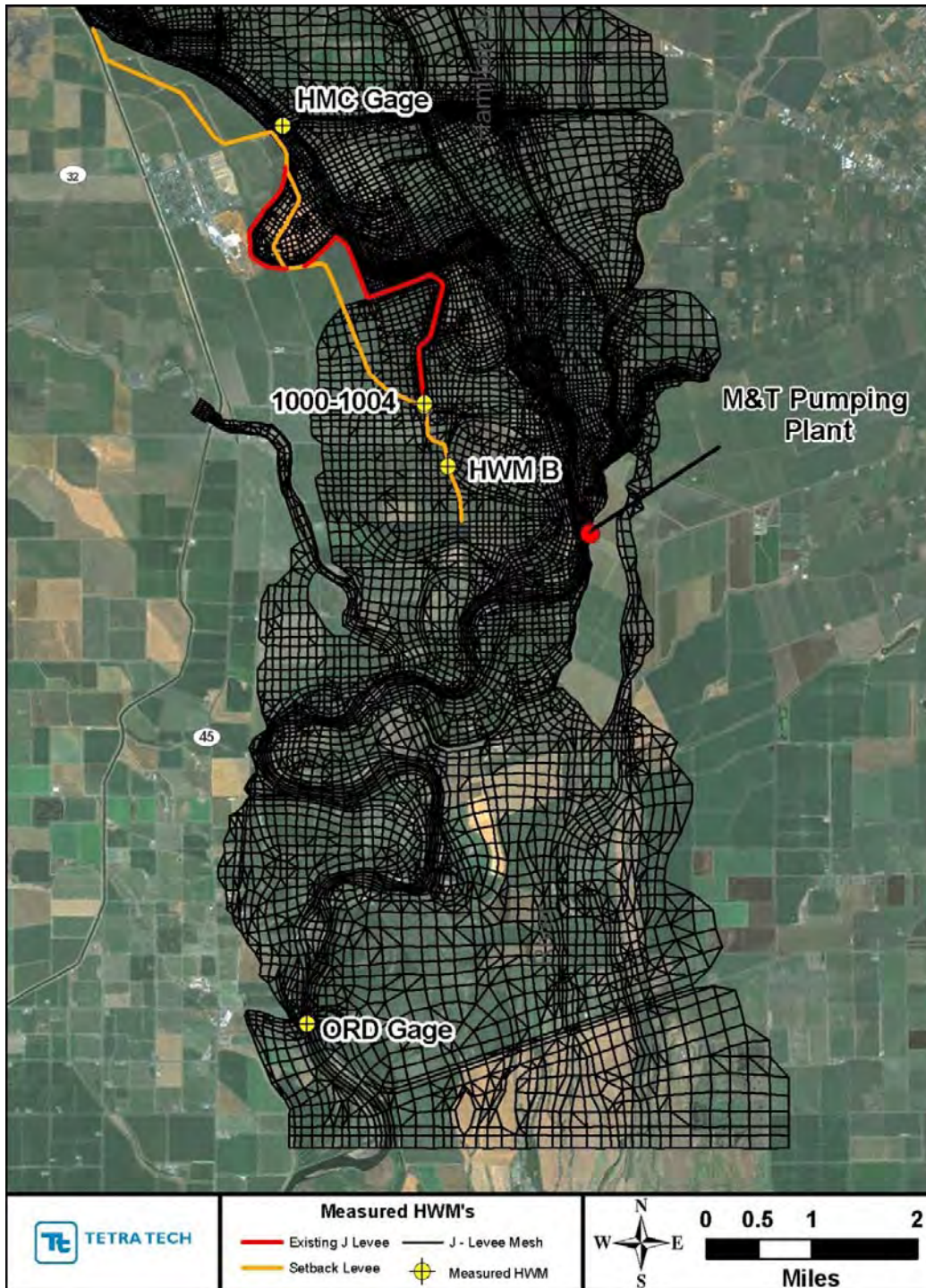


Figure 4.5. Locations of the Ord Ferry gage (ORD), Hamilton City Gage (HMC) and measured high-water marks from December 2005 and January 2006 peak flow events. The mesh is the RMA-2V J-Levee mesh for the 100-year existing conditions. The high-water mark elevations are listed in Table 4.1.

Table 4.1. Two-dimensional model calibration points.*					
January 1, 2006 flow event (134,638 cfs)					
Location	Measured	USACE J-Levee Model (RMA-2V)		Phase III J-Levee Model (SRH-2D)	
		Model	Difference	Model	Difference
HMC Gage	146.84	146.27	-0.57	145.78	-1.09
ORD Gage	115.24	116.54	1.3	115.84	0.60
1000	134.4	134.6	0.2	134.25	-0.2
1001	134.2	134.6	0.4	134.25	0.1
1002	134.2	134.7	0.5	134.25	0.1
1003	134.2	134.7	0.5	134.46	0.3
1004	134.2	134.8	0.6	134.46	0.3
December 29, 2005 flow event (109,322 cfs)					
HMC Gage	144.05	145.1	1.1	144.75	0.7
ORD Gage	113.19	115.6	2.4	115.84	1.9
HWM B	131.95	132.5	0.6	132.12	0.3

*Modified from J-Levee Report (USACE, 2008)

4.2 Evaluation of the Proposed Hamilton City Setback Levee Project on the M&T/Llano Seco Reach

Based on review of the Phase II report (MEI, 2008), the Steering Committee recommended an evaluation of the hydrodynamic impacts of the Hamilton City Setback Levee project on the M&T/Llano Seco reach.

4.2.1 Model Results

The Phase III Setback Levee models were run at the 50- and 100-year peak flow events and the hydraulic results were compared to the Phase III J-Levee model by comparing the difference in predicted water-surface elevations and velocities between the two conditions.

The results of the analysis indicate that at the 50-year peak flow event, the predicted water-surface elevations are a maximum of 3.2 feet higher in the area to the east of the setback levee, and 2.5 feet lower in the area located to the west under the Phase III Setback Levee condition compared to Phase III J-Levee model conditions (**Figure 4.6**). The inclusion of the Setback Levee decreases the width of the floodplain, and as the result, the water-surface elevations increase in area to the east of the setback levee, and decrease in the area behind (to the west) of the training levee. The effect of the Setback Levee, as shown by the area with the increase in water-surface elevations, extends downstream along the floodplain to approximately opposite the M&T Pumps (Figure 4.8). In addition, the Setback Levee increases the water-surface by up to 0.15 feet in the vicinity of River Road and along the river to approximately 1,500 feet downstream of River Road. It is important to note that there are no changes in water-surface elevations at the M&T Pumps, the City of Chico relocated outfall or at the two alternative pump sites.

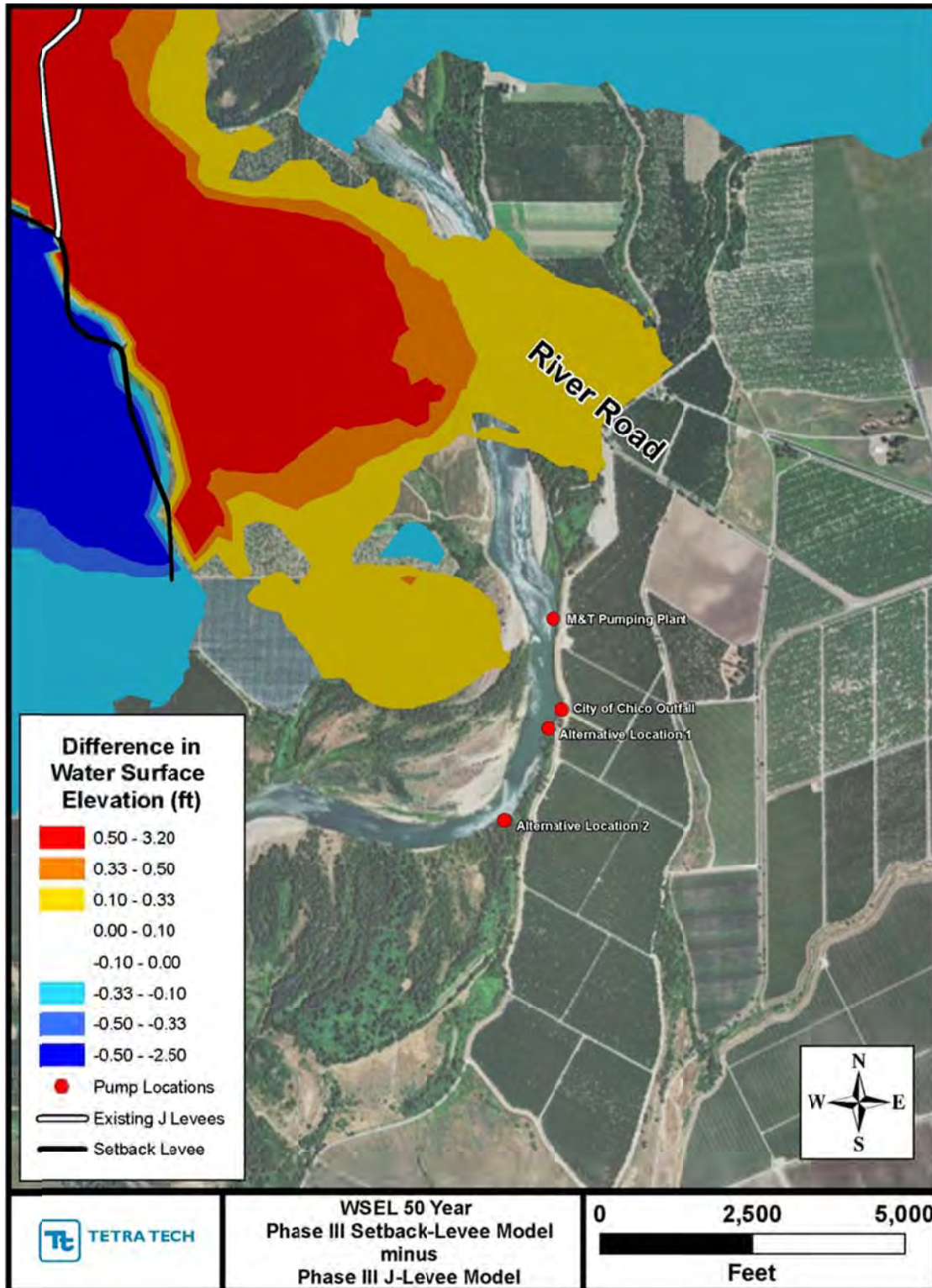


Figure 4.6. Difference in water-surface elevation at the 50-year peak flow event between the Phase III Setback Levee model and Phase III J-Levee model conditions.

At the 50-year event, the velocity increases by a maximum of 2.5 fps, under the Setback model conditions compared to the J-Levee conditions. These increases occur at the downstream end of the J-Levee (the J-Levee is removed in the Phase III Setback Levee models) and at the downstream end of the Setback Levee (**Figure 4.7**). The maximum velocity decrease of approximately 2.1 fps occurs in the main channel approximately 1 mile upstream from River Road and velocity decrease of approximately 1 fps occur on the floodplain opposite the upstream end of the bank-attached bar.

At the 100-year peak flow event, the predicted maximum increase in water-surface elevation is 3.2 feet under the Setback Levee conditions compared to the J-Levee conditions (**Figure 4.8**). Similar to the 50-year event, the maximum increases occur on the east side of the Setback Levee and the maximum decreases of up to 1.7 feet occur to the west of the Setback Levee. Similar to the 50-year peak flow event, the effect of the Setback Levee extends downstream along the floodplain to approximately opposite the M&T Pumps. The largest increase in water-surface elevation opposite M&T Pumps is 0.2 feet and it occurs approximately 1,500 feet to the west of the main channel. The water-surface elevations in the vicinity of River Road increase by approximately 0.1 feet under the Setback Levee conditions. There are no predicted changes in water-surface elevations at the M&T Pumps, the City of Chico relocated outfall or at the two Alternative pump sites.

The maximum velocity increase at the 100-year event is approximately 2.6 fps and it occurs at the downstream end of the J-Levee and at the downstream end of the Setback Levee (**Figure 4.9**). The velocities on the floodplain opposite the M&T Pumps increase by 0.15 fps in an area located approximately 650 feet to the west of the channel. At River Road, the main channel velocities increase by up to 0.6 fps and the velocities increase by up to 0.2 fps at the head of the attached bar. There are no predicted changes in velocity at the M&T Pumps, the City of Chico outfall or at the two alternative pump sites.

In summary, the predicted differences in water-surface elevations and velocities between the Phase III Setback Levee condition and Phase III J-Levee conditions for both the 50- and 100-year peak flow events, indicate that the maximum influences of the Setback Levee occurs in the vicinity of the Setback Levee and the effects of the Setback Levee do not extend across to the M&T Pumps, the City of Chico relocated outfall, or across to the Alternative 1 and 2 sites.

4.3 Evaluation of the Alternative 1 (2,200-foot) and Alternative 2 (3,500-foot) Pumping Sites

Based on review of the Phase II report (MEI, 2008), the Steering Committee recommended that additional 2-D modeling be performed to evaluate two alternative pump sites that are located 2,200 feet (Alternative Site 1) and 3,500 feet (Alternative Site 2) downstream from the existing site (**Figure 4.10**).

The Phase III model was run for a series of flows from 5,000 to 15,000 cfs and the Phase III Setback Levee models were run for a series of flows from 50,000 to 275,910 cfs (100-year peak flow event). The model output was used to evaluate the hydraulic conditions at the 2,200-foot site. The 3,500-foot site is located approximately 1,850 feet downstream from the City of Chico outfall and approximately 1,270 feet downstream from the end of the riprap located along the left (east) bank. Immediately downstream of the riprap, the bank has eroded back approximately 200 feet from the end of the riprap creating a condition where the bankline projects out into the

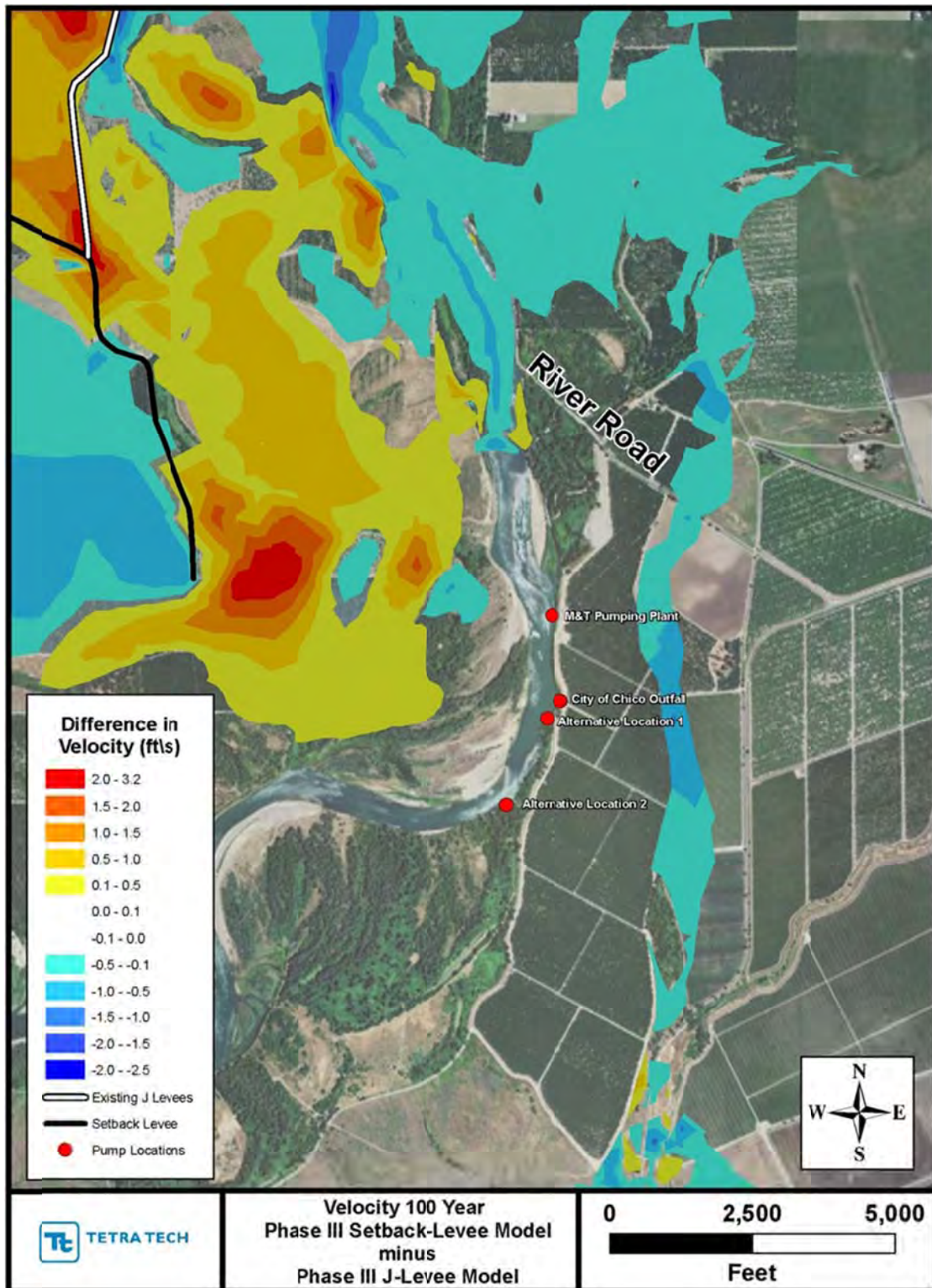


Figure 4.7. Difference in velocity at the 50-year peak flow event between the Phase III Setback Levee model and Phase III J-Levee model conditions.

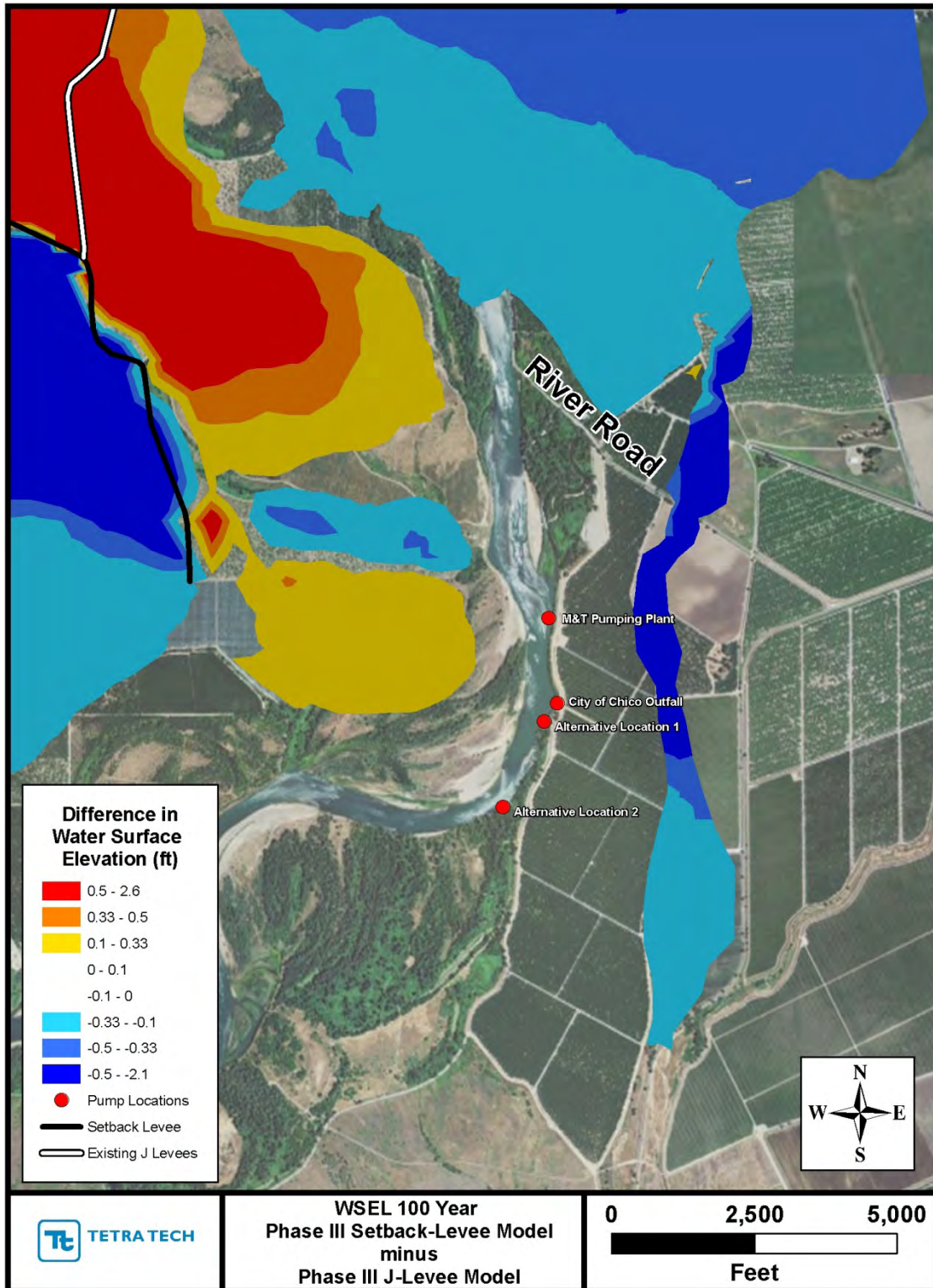


Figure 4.8. Difference in water-surface elevation at the 100-year peak flow event between the Phase III Setback Levee model and Phase III J-Levee model conditions.

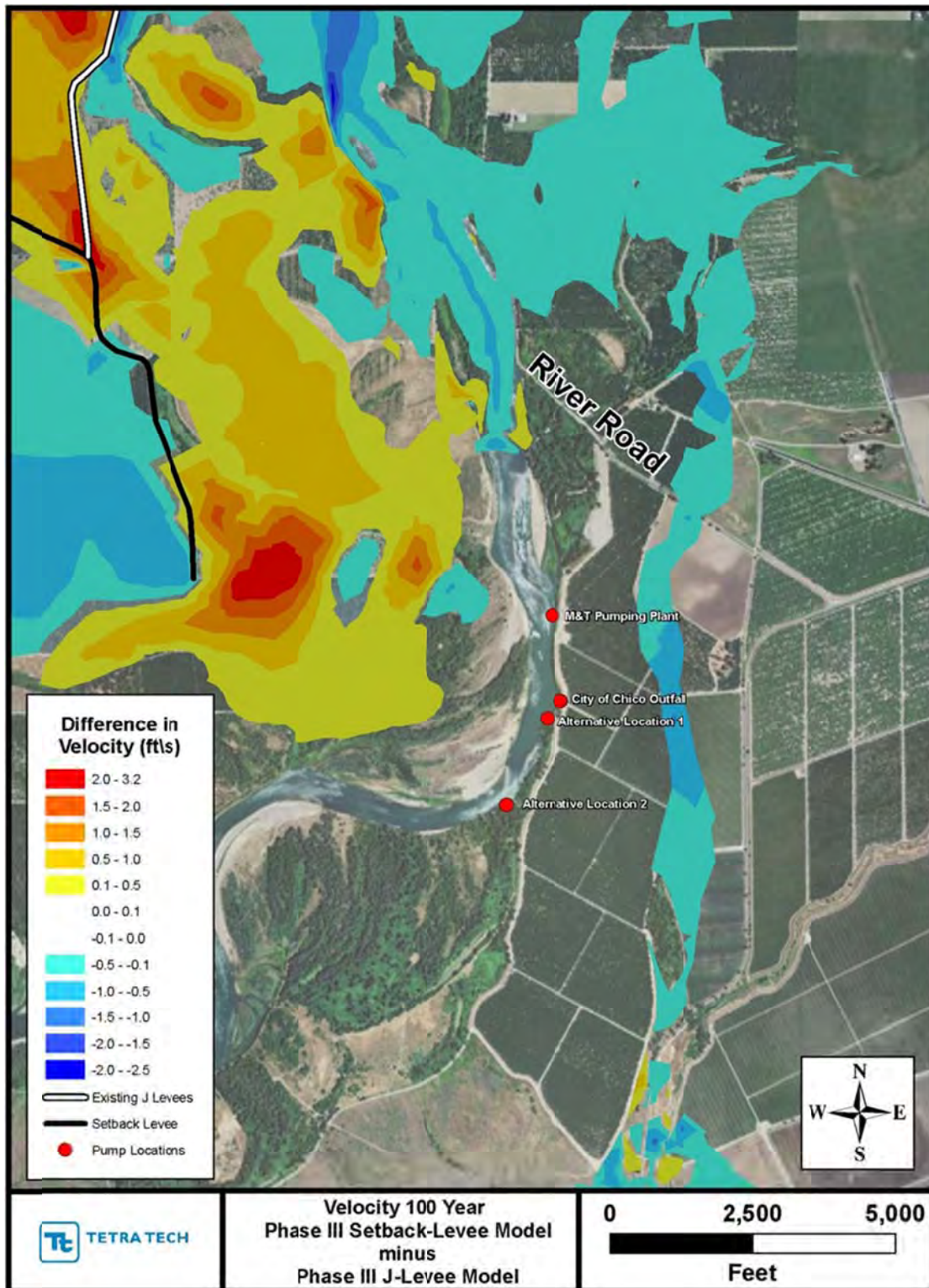


Figure 4.9. Difference in velocity at the 100-year peak flow event between the Phase III Setback Levee model and Phase III J-Levee model conditions.



Figure 4.10. Locations of the Alternative Pump Sites 1 and 2 located 2,200 and 3,500 feet, respectively downstream from the existing site.

river at the downstream end of the riprap (**Figure 4.11**). In addition, the survey data indicate there is a significant scour hole located in the channel opposite the downstream end of the riprap (Figure 4.11). The relocation of the pump to the 3,500-foot site will require realignment of the left bank. Therefore, a separate hydraulic model of the 3,500-foot site was created by adjusting the topography along the left bank to represent the proposed alignment shown in Figure 4.11. The extents and elevations of the scour hole following the bank realignment were estimated using the topography predicted by the bank realignment scenario in the physical model study (CSU, 2011).

4.3.1 Model Results

The model results were used to develop depth-discharge, velocity-discharge and shear stress-discharge rating curves to evaluate the hydraulic conditions at the Alternative 1, Alternative 2, M&T Pump and the City of Chico outfall sites. In addition, the hydraulic model results were used to develop a water-surface elevation-discharge rating curve at the downstream boundary of the physical model that was provided to CSU.

Results from the Phase III Setback Levee model at the 50-year peak flow event indicate that the main channel velocities range from 5 to 11 fps (**Figure 4.12**) and maximum depths range from 25 to 50 feet (**Figure 4.13**). Within the M&T reach (RM 192 to RM 193.5), the highest velocities of 11 fps occur approximately 900 feet downstream from the M&T Pumps and the maximum flow depths of 39 feet occur approximately 600 feet downstream from the pumps. At the M&T Pumps, the maximum depth is 33 feet and the velocity is approximately 6 fps. The flow depths adjacent to the Setback Levee generally range from 3 to 10 feet and the velocities range from 1 to 2.5 fps.

At the 100-year peak flow event, the main channel velocities range from 5 to 11 fps (**Figure 4.14**) and maximum depths range from 26 to 53 feet (**Figure 4.15**). Within the M&T reach, the highest velocities of 11 fps occur approximately 900 feet downstream from the M&T Pumps and the maximum flow depths of 41 feet and occur approximately 600 feet downstream from the pumps. At the M&T Pumps, the maximum depth is 32 feet and the velocity is approximately 6 fps. The flow depths adjacent to the Setback Levee generally range from 4 to 11 feet and the velocities range from 1 to 2.5 fps.

4.3.2 Depth-Discharge Rating Curves

Depth-discharge rating curves were developed at the M&T Pump, City of Chico and Alternative 1 sites using the Phase III Setback Levee model results. The results from the Phase III Realignment model were used to develop rating curves at the Alternative 2 site. In addition, the results from the Phase III Realignment models were used to develop a rating curve at the City of Chico outfall and the results were compared to the Phase III Setback Levee model results to evaluate whether bank realignment would have any adverse effects at the City of Chico outfall.

The intakes for the M&T Pumps are located approximately 150 feet from the edge of water during low-flow conditions and the bed elevation was approximately 102.7 feet at the time of the January 2010 survey. The City of Chico outfall extends from the near the edge of the bank to the center of the channel; a bed elevation of 103.5 feet was selected to represent the approximate bed elevation which extend from near the bank to approximately 150 feet towards the center of the channel. (Note: the thalweg elevation opposite the City of Chico Outfall was approximately 100.0 feet during the January 2010 survey). If the Alternative 1 or 2 sites are

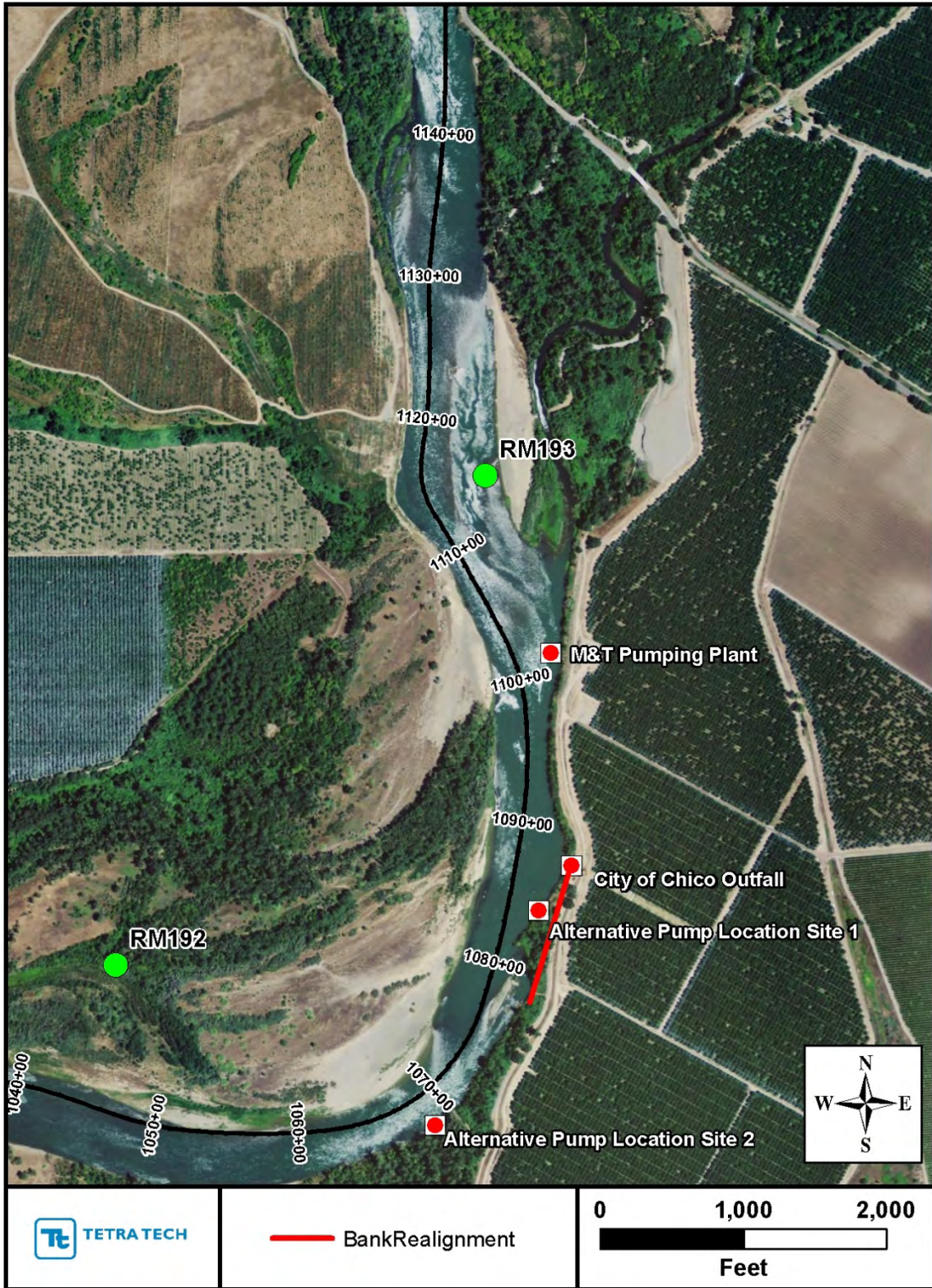


Figure 4.11. The alignment of the left bank used to evaluate the 3,500-foot pump site.

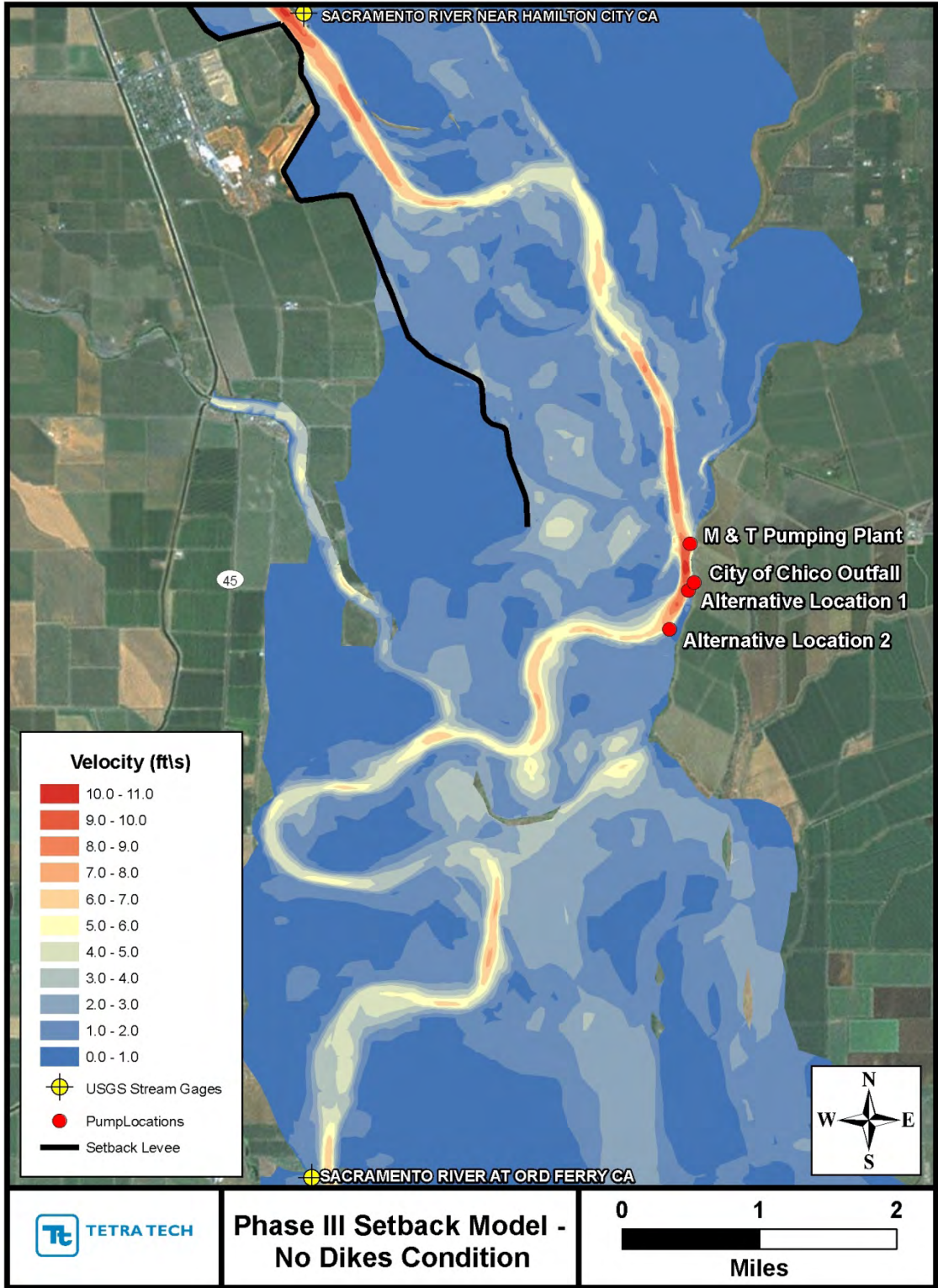


Figure 4.12. Distribution of flow velocities predicted by the Phase III Setback Levee model at the 50-year peak flow event.

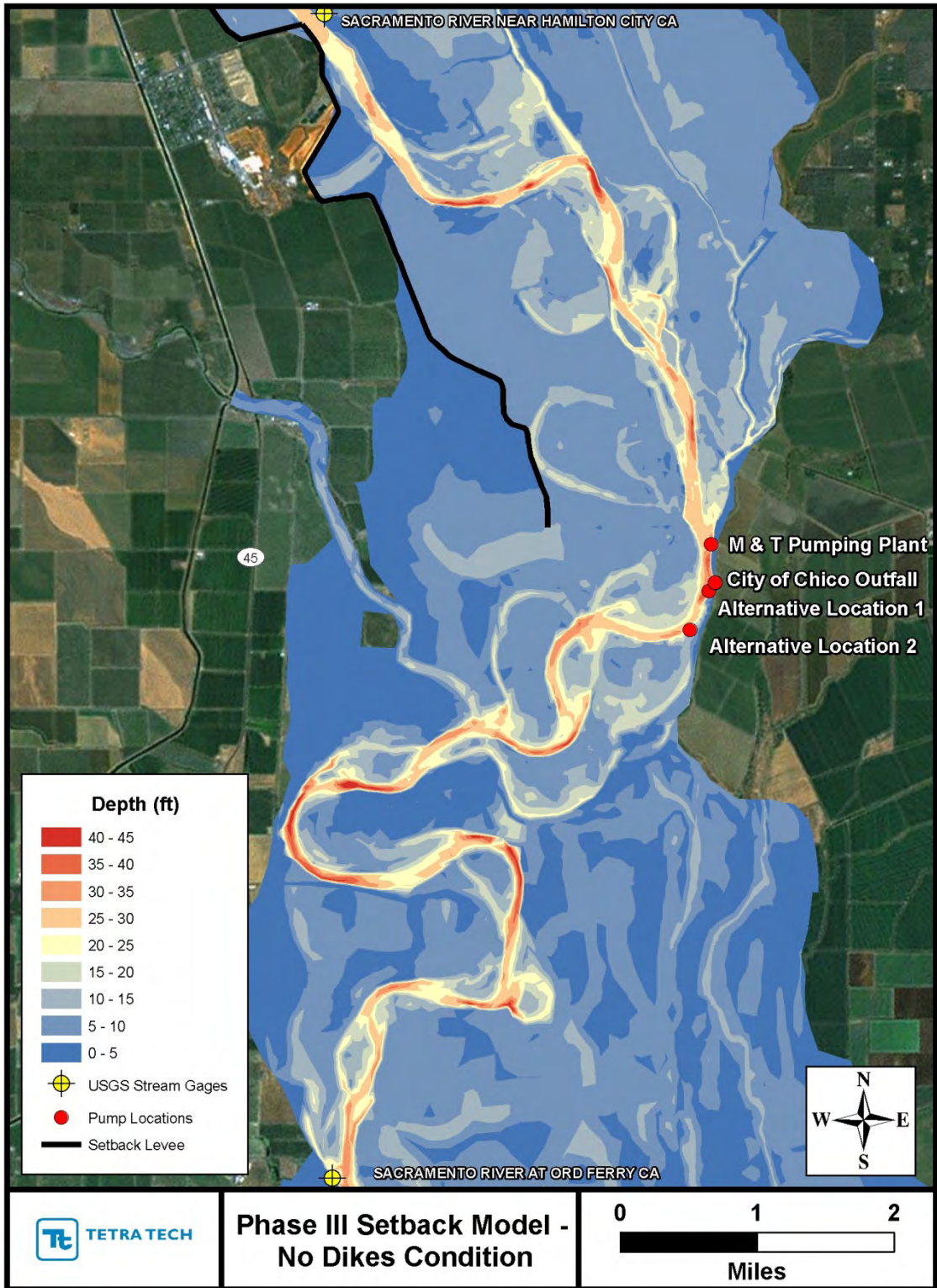


Figure 4.13. Distribution of flow depths predicted by the Phase III Setback Levee model at the 50-year peak flow event.

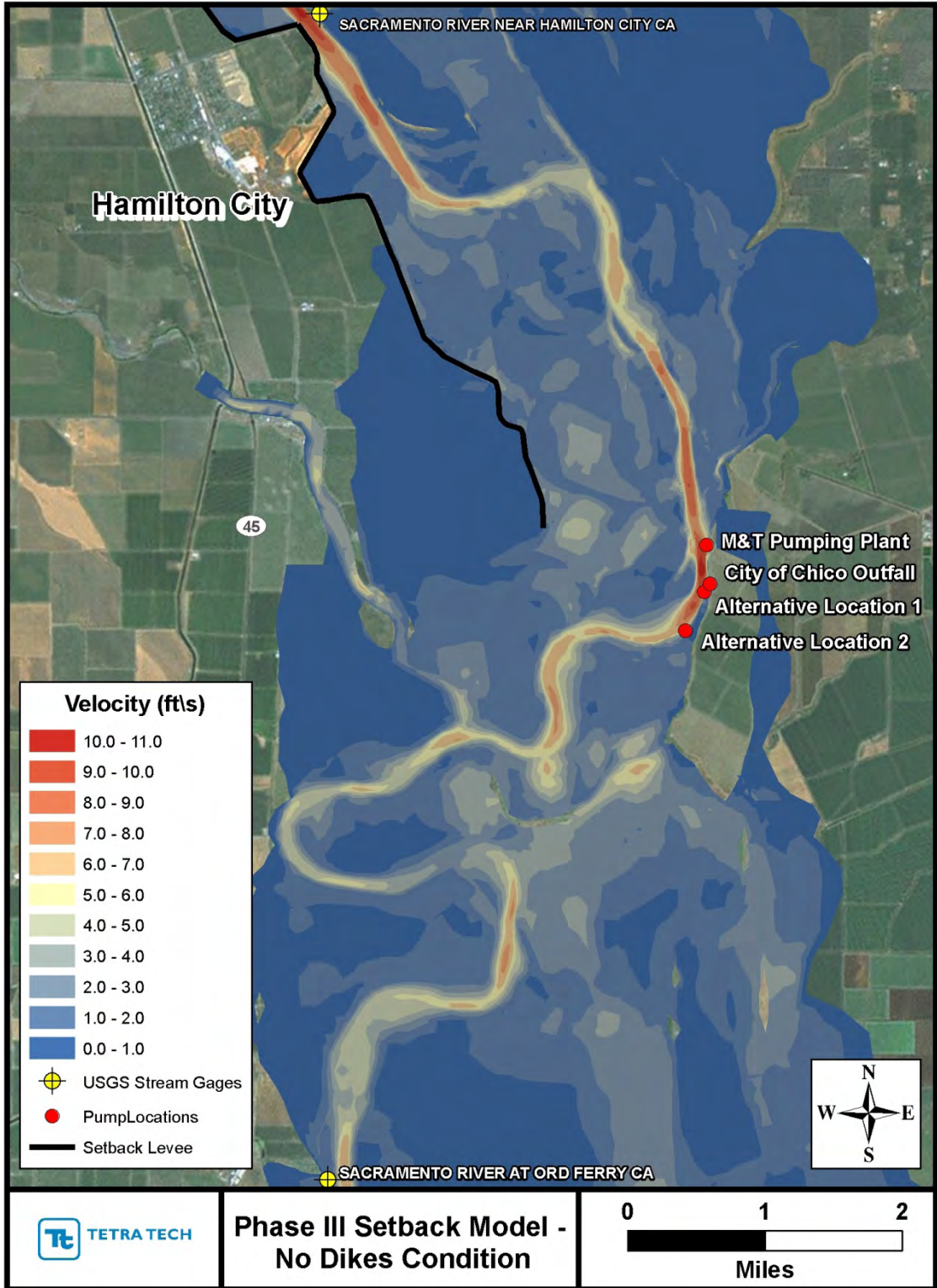


Figure 4.14. Distribution of flow velocities predicted by the Phase III Setback Levee model at the 100-year peak flow event.

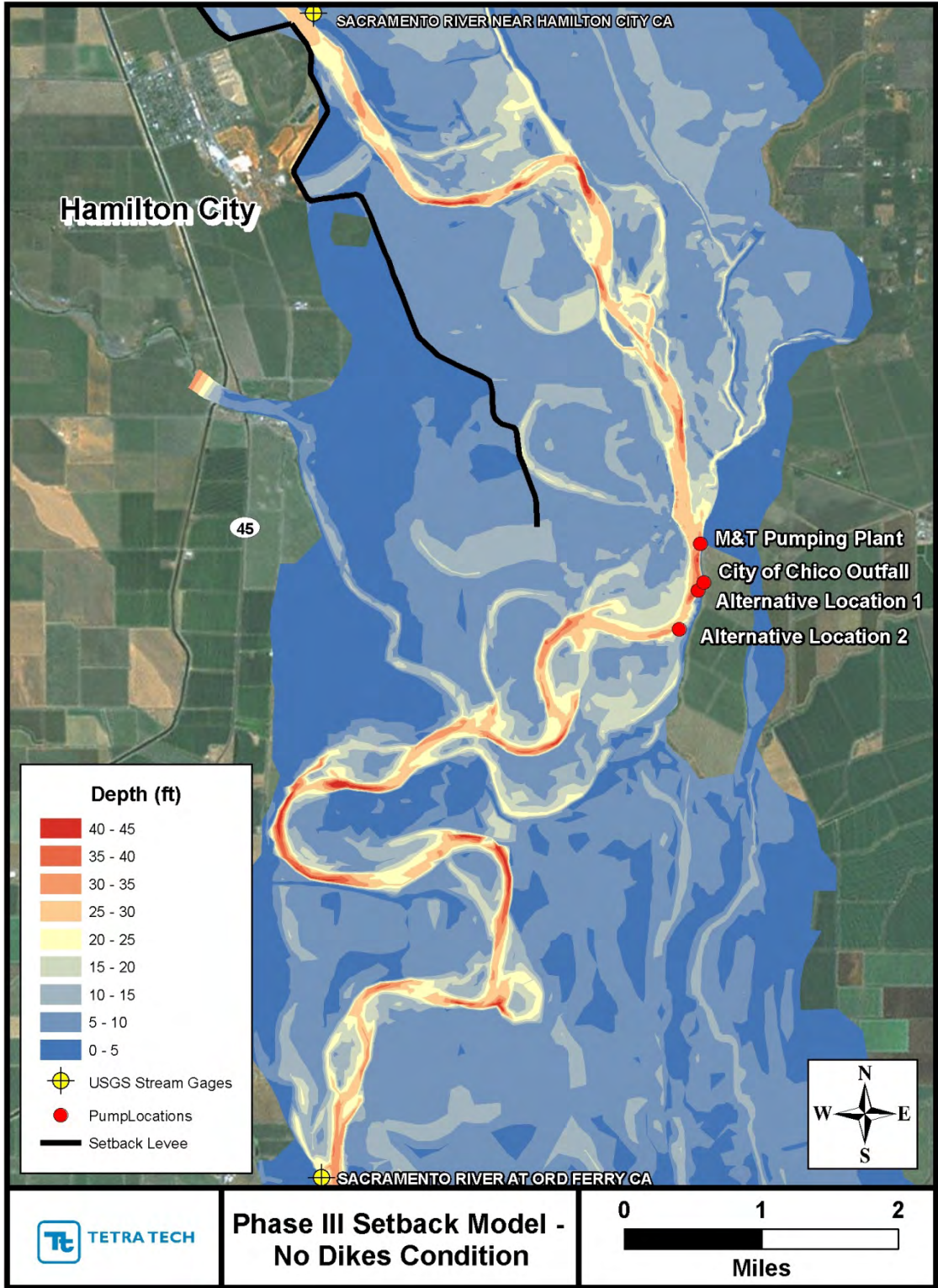


Figure 4.15. Distribution of flow depths predicted by the Phase III Setback Levee model at the 100-year peak flow event.

selected, then the intake screens would be located adjacent to the bank, as opposed to being in the channel at the existing M&T fish screen intake. Therefore the representative bed elevations at the Alternative 1 and 2 sites were selected near the bank and are 102.8 and 105.3 feet, respectively.

The depth-discharge rating curves indicate that the Alternative 1 and M&T Pump sites have similar depths, whereas the flow depths at the City of Chico site are approximately 4 feet lower than the Alternative 1 site over the range of modeled flows (**Figure 4.16**). The Alternative 1 site is located on the outside of the bend and the channel is relatively deep, presumably from bend scour and the turbulence created by the riprap. The City of Chico site is located in a small embayment caused by the alignment of the bank. Sediment deposition has occurred adjacent to the City of Chico outfall, and as a result, the predicted flow depths are lower compared to the M&T site.

At 5,000 cfs, the flow depths at the M&T Pumps, City of Chico outfall and Alternative 1, and Alternative 2 sites are 11.2, 10.3, 11.1 and 7.2 feet, respectively. At the bankfull discharge of 90,000 cfs, the flow depths at the M&T Pumps, City of Chico outfall and Alternative 1 and 2 sites are 26.9, 25.5, 26.2 and 23.8 feet, respectively, and at the 100-year peak flow event, the flow depths are 34.2, 32.6, 33.1 and 30.2 feet, respectively.

Under the Bank Realignment scenario, the predicted depths at the City of Chico outfall are similar to Phase III Setback Levee model results indicating that the bank realignment required for the Alternative 2 pump relocation will have little effect on depths at the City of Chico outfall (**Figure 4.16**). At 5,000 cfs, the predicted flow depth is 0.2 feet lower under the Realignment scenario (10.1 feet) compared to the Phase III Setback Levee model conditions (10.3 feet). In the range of flows from 10,000 to the 100-year peak flow event, the predicted depths for the Realignment scenario are approximately 0.05 feet lower compared to Phase III Setback Levee model results.

The Alternative 2 site is located upstream from a riffle that extends across the channel, which results in relatively shallow depths compared to the M&T site. In general, the Alternative 2 flow depths are 3.8 feet lower compared to the M&T Pump site over the range of flows from 10,000 cfs to the 100-year peak flow event.

4.3.3 Velocity-Discharge Rating Curves

The velocity-discharge rating curve at the M&T Pumps shows the velocity steadily increasing up to 90,000 cfs (bankfull discharge) and then flattening out at higher discharges due to an increase in overbank flooding (**Figure 4.17**). At the M&T Pumps site, the velocities at 5,000 and 90,000 cfs and the 100-year peak flow event are 1.6, 5.4 and 5.7 fps, respectively.

The shape of the velocity-discharge rating curves is similar for the City of Chico outfall and the Alternative 1 sites, with the velocities steadily increasing up to 134,638 cfs (December 2006 peak flow) and then flattening out slightly at higher discharges (**Figure 4.17**). The channel geometry at these two sites is similar in that channel widths are reasonable similar and they are confined by banks on both sides of the channel. At 5,000 cfs, the velocities are 1.0 fps at the City of Chico and at the City of Chico outfall. At 90,000 cfs, the velocities are 4.5 fps at the City of Chico compared to 4.6 fps at the Alternative 1 site. At the 100-year peak flow event, the velocity at the City of Chico is 6.4 fps compared to 7.0 fps at the Alternative 1 site.

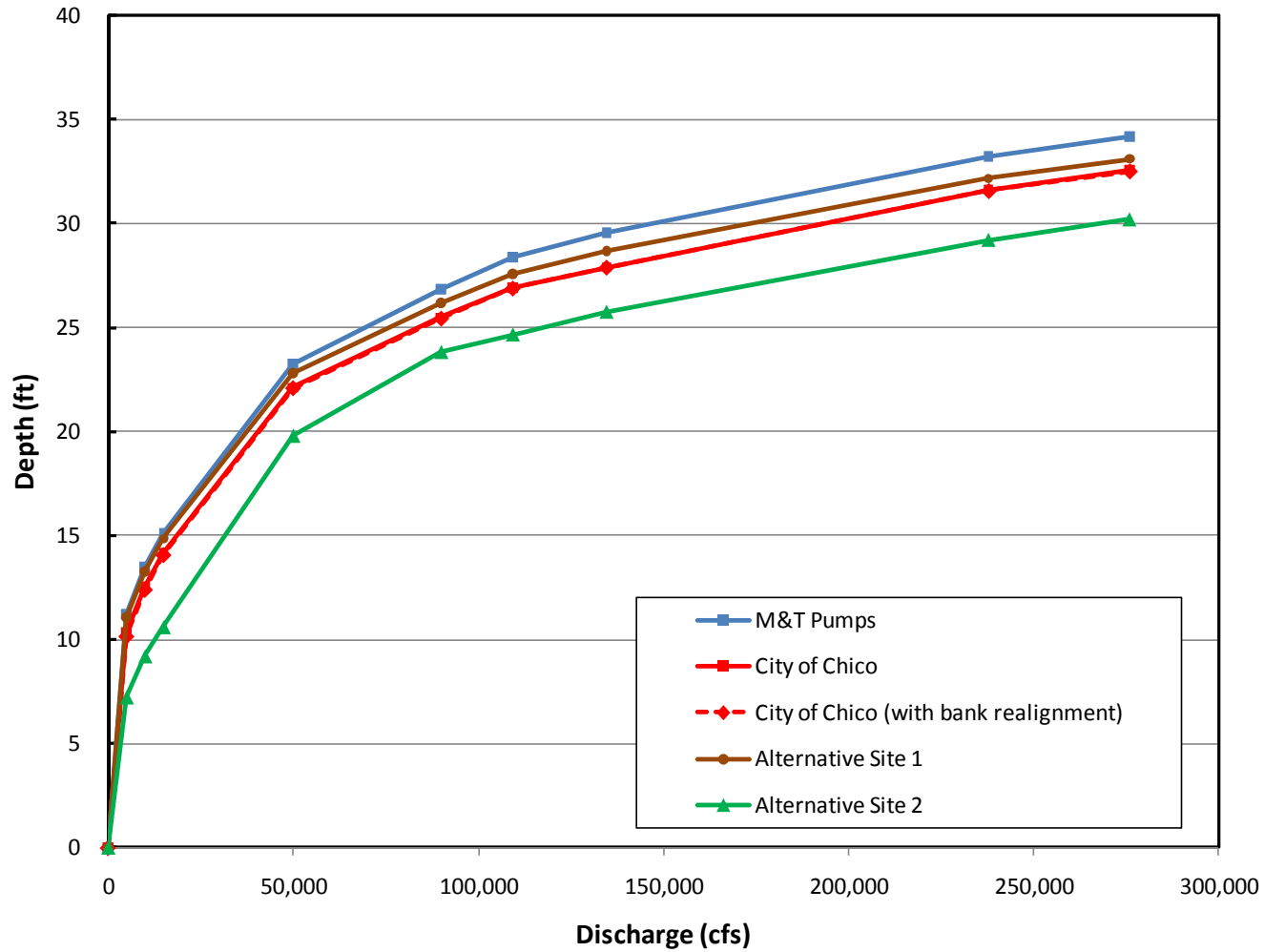


Figure 4.16. Comparison of the predicted depths at the M&T Pumps, City of Chico outfall, Alternative Sites 1 and 2.

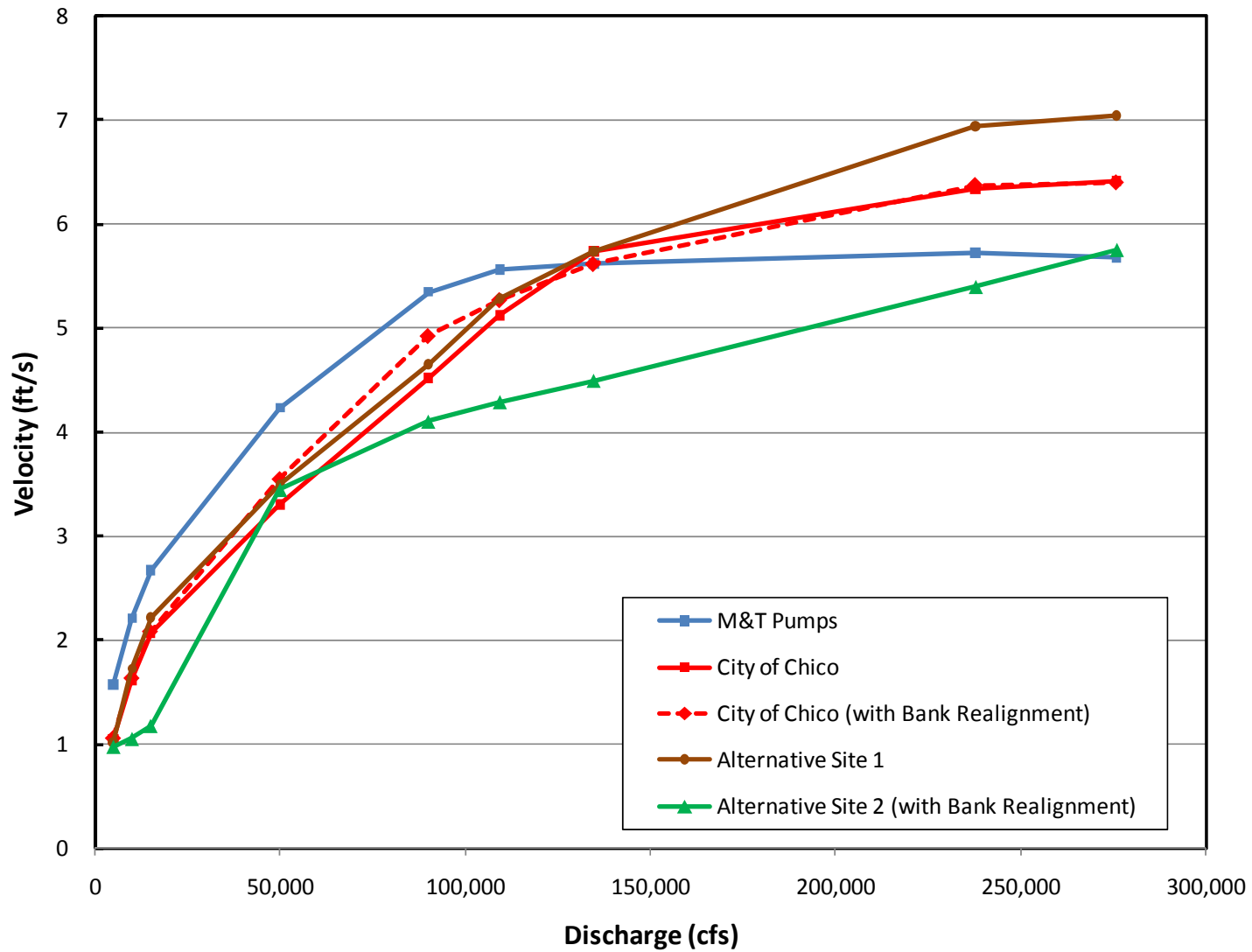


Figure 4.17. Comparison of the predicted velocities at the M&T Pumps, City of Chico outfall, Alternative Sites 1 and 2.

The Alternative 2 site is located downstream from the other sites and the right side of the channel is located adjacent to a large active point bar which is relatively unconfined at higher discharges. The velocity-discharge rating curve at the Alternative 2 sites indicates that the velocity steadily increases up to approximately 50,000 cfs and then flattens out at higher discharges, due to the increase in flows over the point bar. At the Alternative 2 site, the velocities at 5,000 and 90,000 cfs and the 100-year peak flow event are 1.0, 4.1 and 5.7 fps, respectively.

Under the Bank Realignment scenario, the predicted velocities at the City of Chico outfall are similar compared to the Phase III Setback Levee model results (Figure 4.17). At 5,000 cfs, the predicted velocity is 1.1 fps under the Realignment scenario compared to 1.0 fps under the Phase III Setback Levee model conditions at a distance of approximately 150 feet from the bank. At 90,000 cfs, the predicted velocity is 4.9 fps under the Realignment scenario compared to 4.5 fps under the Phase III Setback Levee model conditions. At flows greater than 90,000 cfs, the predicted velocities for the Realignment scenario are on average 0.1 fps higher compared to Phase III Setback Levee model results.

4.3.4 Shear Stress-Discharge Rating Curves

The shape of the shear stress-discharge rating curves is similar to the velocity-discharge rating curves; the shear stress values at the M&T Pumps steadily increase up to 90,000 cfs and then flatten out at higher discharges (**Figure 4.18**). The shear stresses at the M&T Pumps at 5,000 and 90,000 cfs and the 100-year peak flow event are 0.04, 0.34 and 0.35 lb/ft², respectively.

The shape of the shear stress-discharge rating curves is similar for the City of Chico outfall and the Alternative 1 sites, with the shear stresses steadily increasing up to 134,638 cfs (December 2006 peak flow) and then flattening out slightly at higher discharges (Figure 4.17). The bed shear stresses at the City of Chico outfall and at the Alternative 1 site are also reasonably low at discharges up to 90,000 cfs compared to the M&T Pumps site. At the City of Chico outfall, the shear stress at 5,000 cfs, 90,000 cfs and the 100-year peak flow event are 0.02, 0.25 and 0.46 lb/ft², respectively. At the Alternative 1 site, the shear stress at 5,000 cfs, 90,000 cfs and the 100-year peak flow event are 0.03, 0.22 and 0.37 lb/ft², respectively.

The shear stress-discharge rating curve at the Alternative 2 site steadily increases up to 50,000 cfs and then flattens slightly at higher discharges. In general, at discharges greater than 90,000 cfs, the shear stress values at the Alternative 2 site are lower than at the other sites. At the Alternative 2 Site, the shear stresses at 5,000 and 90,000 cfs and the 100-year peak flow event are 0.07, 0.40 and 0.62 lb/ft², respectively.

Under the Bank Realignment scenario, the predicted shear stresses at the City of Chico outfall are, on average, 0.02 lb/ft² higher compared to Phase III Setback Levee model conditions over the entire range of flows. Under the Bank Realignment scenario, the predicted shear stresses at the City of Chico outfall at 5,000 and 90,000 cfs and the 100-year peak flow event are 0.02, 0.29 and 0.46 lb/ft², respectively.

The design criteria of the City of Chico outfall were not available to evaluate the hydrodynamic effects of the bank realignment on the outfall, however, the very small changes in hydrodynamic conditions indicates that the bank realignment will not likely affect the City of Chico outfall.

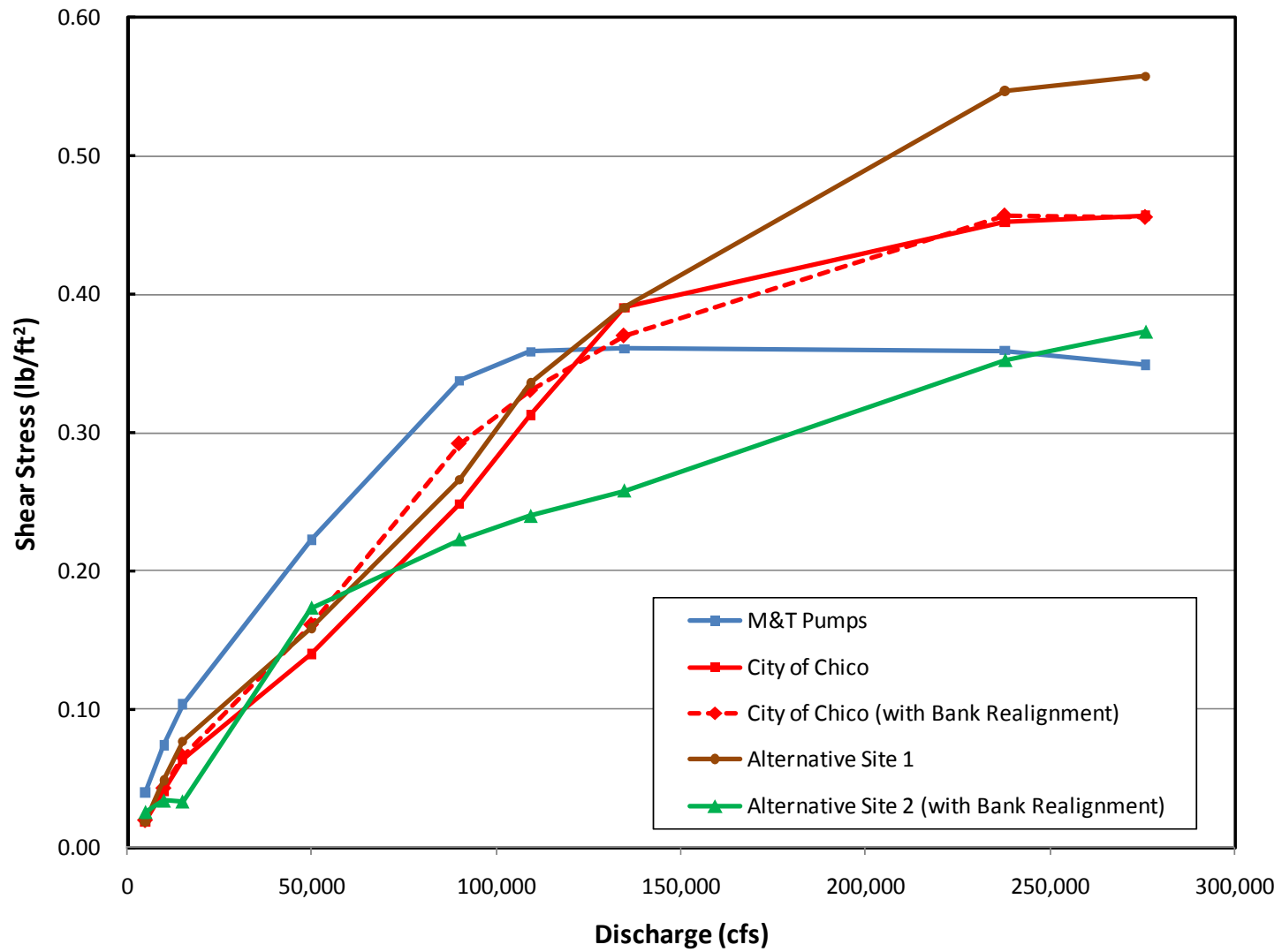


Figure 4.18. Comparison of the predicted shear stress at the M&T Pumps, City of Chico outfall, Alternative Sites 1 and 2.

4.4 Evaluation of Proposed Dike Field impacts on the Setback Levee

Based on review of the Phase II report (MEI, 2008), the Steering Committee recommended an evaluation of the hydrodynamic impacts of the proposed spur dikes on Hamilton City Setback Levee project, and conversely, the hydrodynamic effects of the Setback Levee project on the M&T spur dikes. To evaluate the impacts, the Phase III Setback Levee model was modified to include the nine dikes developed for the Phase II analysis.

4.4.1 Model Development

The Phase III Dike-Setback Levee model was developed by incorporating the 9-dike design developed during the Phase II analysis into the Phase III Setback Levee model. During the Phase II analysis, 8- and 9-dike configurations were evaluated and it was concluded that the 9-dike configuration was required to meet the project objectives.

For the Phase III Dike-Setback Levee model, an n -value of 0.20 was applied to the dikes to reflect the roughness and additional turbulence associated with the dikes. The other roughness values and the other model parameters of the Phase III Dike-Setback Levee model remained the same as the Phase III Setback Levee model for consistency.

4.4.2 Model Results

The Phase III Dike-Setback Levee model was run at the 50- and 100-year peak flow events and the hydraulic results were compared to the Phase III Setback Levee model by comparing the difference in predicted water-surface elevations and velocities between the two conditions.

The results of the analysis indicate that at the 50-year peak flow event, the predicted water-surface elevations are a maximum of 2.8 feet higher under the Phase III Dike-Setback Levee condition compared to Phase III Setback Levee model conditions (**Figure 4.19**). In general, the increase in water-surface elevations occur on the upstream side of the dikes and the decreases in water-surface elevations occur on the downstream side of the dikes.

The effect of the dikes, as shown by the area with the increase in water-surface elevations, is limited to the approximate extent of the dikes. Increases in water-surface elevation of up to 0.12 feet occur approximately 200 feet upstream of the most upstream dike, and the increased water-surface elevations extend a maximum of 250 feet to the west of the dikes. It is important to note that the increases in water-surface elevations do not extend up to River Road or across to the Setback Levee. The maximum velocity differences at the 50-year event are approximately 4.0 fps and these differences occur at the dikes (**Figure 4.20**). As expected, the largest decreases in velocities of up to 5.5 fps occur between the dikes and the largest increases in velocity of up to 5 fps occur over the top of the dikes. The increases in velocity of approximately 0.15 fps extend approximately 800 feet to the west of the channel. In addition, there are velocity increases of up to approximately 0.8 fps over the bank-attached bar and increases of 0.2 fps in the vicinity of the M&T Pumps. Similar to the depth results, the predicted area of velocity change is located in the immediate vicinity of the dikes and does not extend across to the Setback Levee.

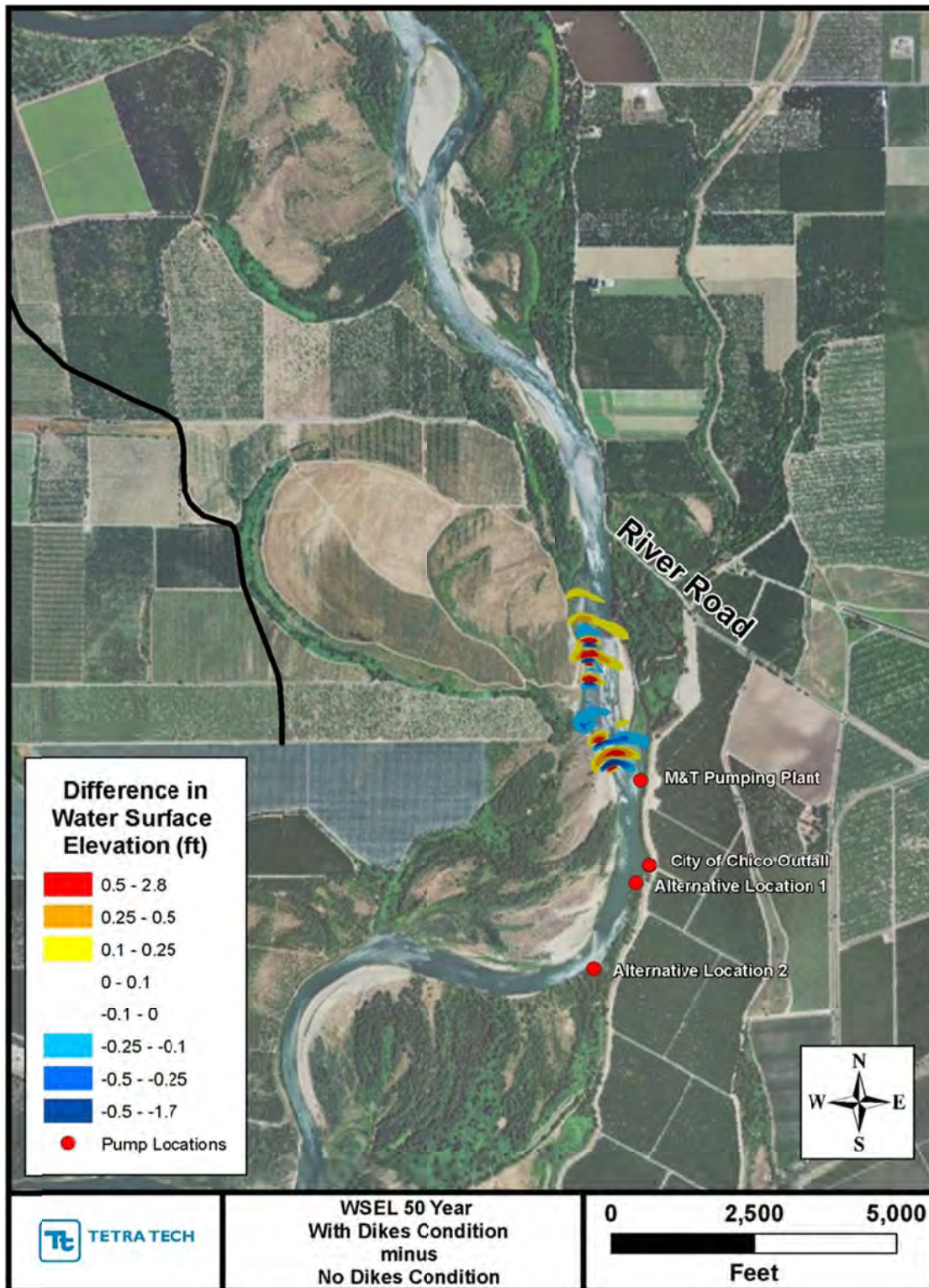


Figure 4.19. Difference in water-surface elevation at the 50-year peak flow event between the Phase III Dike-Setback Levee model and Phase III Setback Levee model conditions.

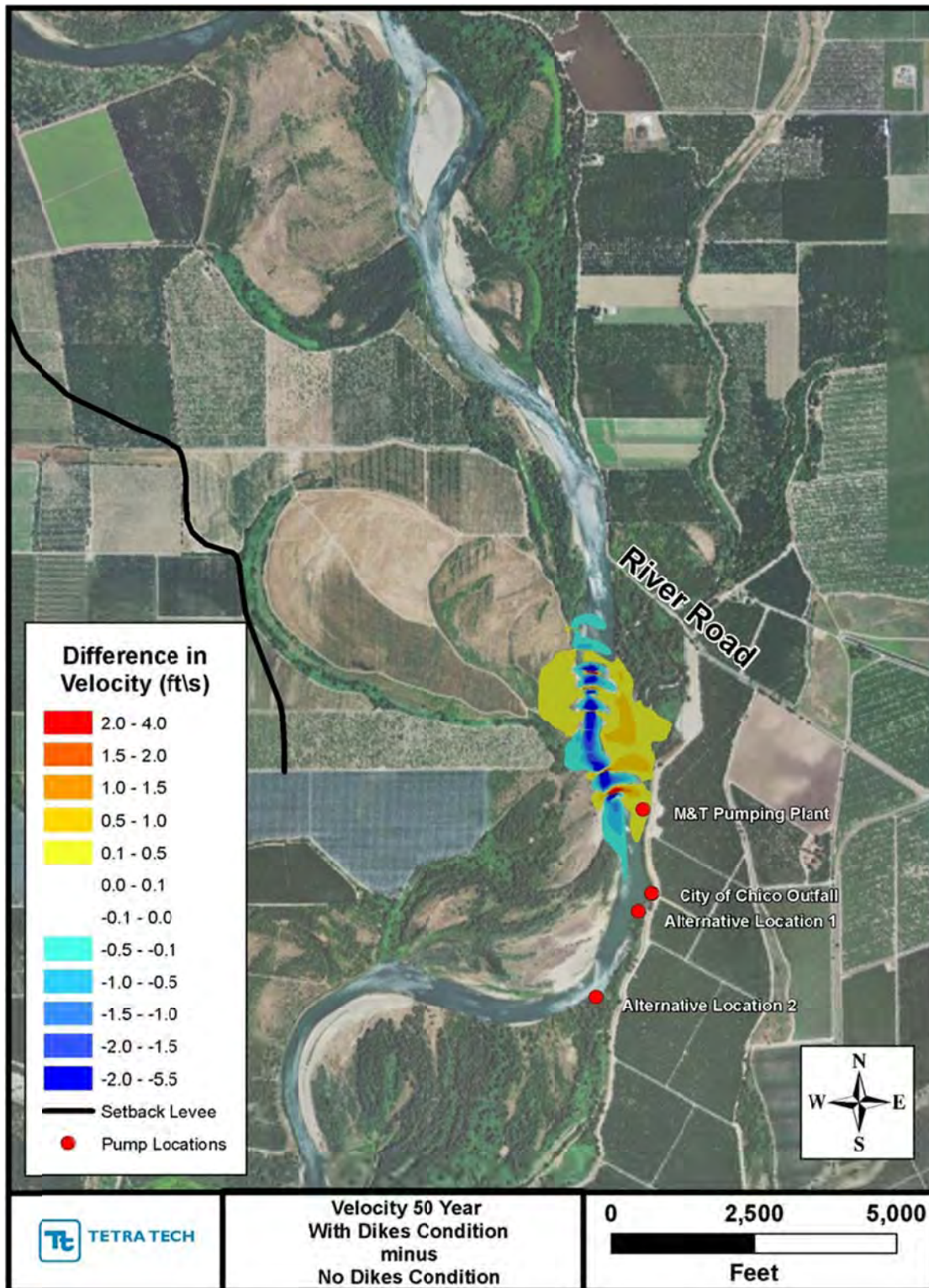


Figure 4.20. Difference in velocity at the 50-year peak flow event between the Phase III Dike-Setback Levee model and Phase III Setback Levee model conditions.

At the 100-year peak flow event, the predicted maximum increase in water-surface elevation is 3.0 feet under the with-dike conditions compared to the without-dike conditions (**Figure 4.21**). Similar to the 50-year event, the increases occur on the upstream side of the dikes and maximum decreases of 2.2 feet occur on the downstream side of the dikes. In addition, the area of predicted change in water-surface elevation is similar to the 50-year event, in that the changes are effectively located within the channel and along the length of the dike field.

The maximum velocity increases at the 100-year event are approximately 4.0 fps (**Figure 4.22**) and they occur over the top of the dikes, and the largest maximum decreases of up to 5.7 fps occur between the dikes. Increases in velocity of approximately 0.15 fps extend approximately 950 feet to the west of the channel. The increase in velocity over the bank attached bar is approximately 1 fps and the increase at the M&T Pumps is approximately 0.2 fps. Similar to the depth results, the predicted area of velocity change is located in the immediate vicinity of the dikes and does not extend across to the Setback Levee.

In summary, the predicted differences in water-surface elevations and velocities between the with-dikes condition and the Phase III Setback Levee model condition for both the 50- and 100-year peak flow events, indicate that the influences of the dikes are limited to the area in the vicinity of the dikes and the effects of the dikes do not extend across to the Setback Levee or up to River Road.

4.5 Evaluation of the Right Bank Gravel Stockpile

In 2000 and 2008, approximately 189,000 yd³ (255,200 tons) and 84,500 yd³ (114,100 tons) of material, respectively, were dredged from the gravel bar located on the east side of the river upstream from the M&T Pumps as a short-term solution to limit sedimentation at the pump inlet. The dredged gravel was stockpiled on the left (east) overbank, just downstream from Big Chico Creek. It is unlikely that there is sufficient room to stockpile future dredge material in this location. An alternative stockpile site was proposed on the right (west) bank just downstream of the M&T Pump (**Figure 4.23**) on land managed by The Nature Conservancy. An analysis was conducted to determine the hydrodynamic impacts of creating a gravel stockpile on the right bank opposite the M&T Pump. In addition, an incipient motion analysis was conducted to evaluate the stability of the gravel in the stockpile.

4.5.1 Model Development

For the purposes of the analysis, it was assumed the stockpile would store 100,000 tons of gravel. The proposed stockpile area is 1,000 feet long by 300 feet wide by 9 feet high and would be rounded on the upstream side to deflect flow (**Figure 4.23**). The sideslopes of the stockpile were assumed to be 1.5H:1V. The Phase III Stockpile model was developed by raising the overbank elevations of the Phase III Setback Levee model to represent the stockpile. An *n*-value of 0.06 was used in the model to reflect the roughness of the stockpile. The other roughness values and the other model parameters of the Phase III Stockpile model remained the same as the Phase III Setback Levee model for consistency.

4.5.2 Model Results

The Phase III Stockpile model was run at the 50- and 100-year peak flow events and the hydraulic results were compared to the Phase III Setback Levee model results.

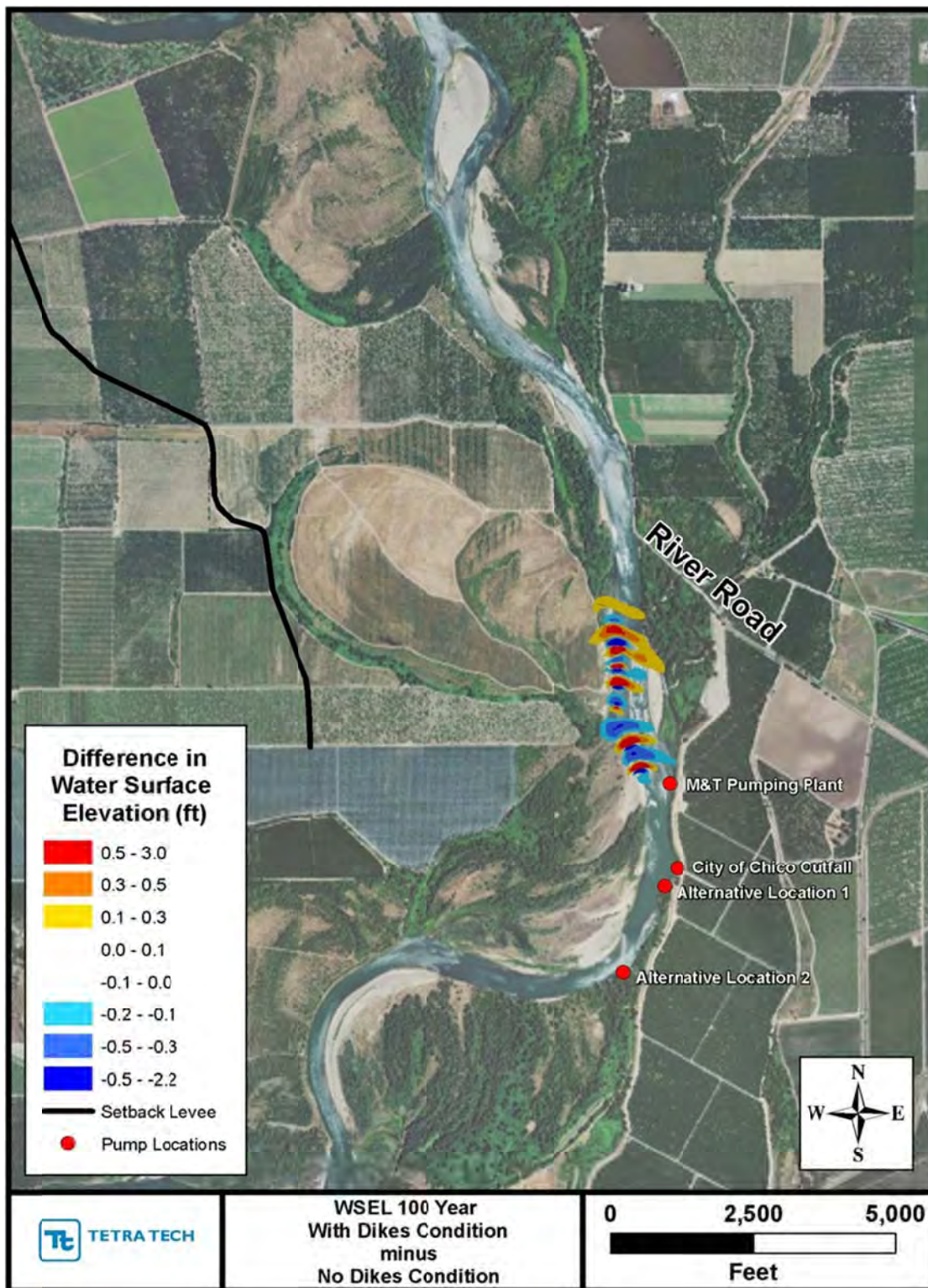


Figure 4.21. Difference in water-surface elevation at the 100-year peak flow event between the Phase III dike-Setback Levee model and Phase III Setback Levee model conditions.

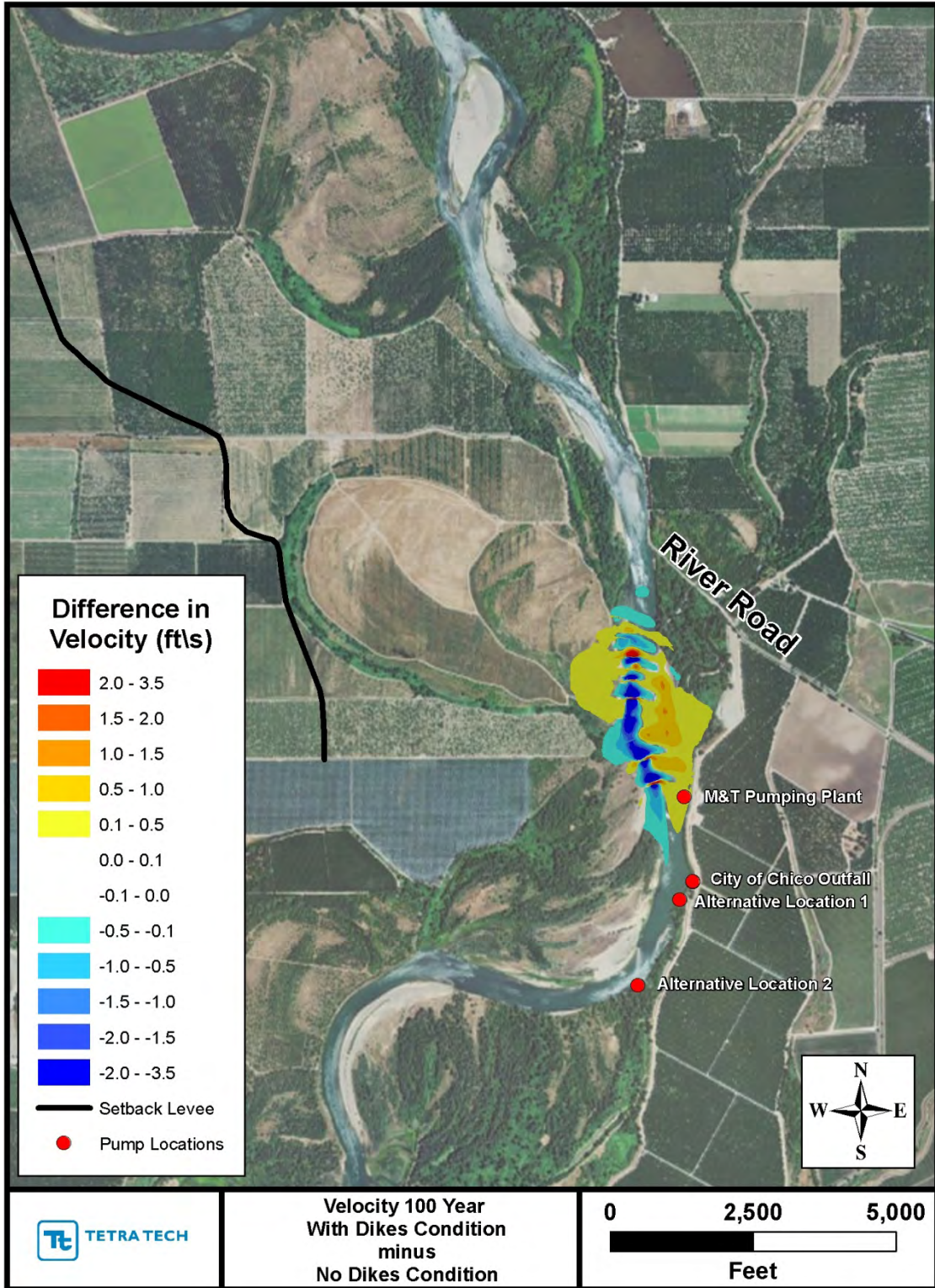


Figure 4.22. Difference in velocity at the 100-year peak flow event between the Phase III Dike-Setback Levee model and Phase III Setback Levee model conditions.



Figure 4.23. Location of the gravel stockpile.

The Phase III Setback Levee model indicates that at the 50-year peak flow event, the water depth around the stockpile ranges from 5 feet along the western side to 8 feet along the east side. At the 100-year flow event, the water depths range from 6 feet along the western side to 9 feet along the east side. The model predicts that the velocities are approximately 1 fps along the west side of the stockpile and range from 2 to 5 fps along the east side of the stockpile at both the 50- and 100-year peak flow events.

The results from the Phase III Stockpile model at the 50-year peak flow event, indicate that the water-surface elevations will increase by approximately 0.25 feet along the upstream side of the stockpile, will increase by approximately 0.2 feet along the western side and will decrease by approximately 0.2 feet along the downstream end of the stockpile compared to the Phase III Setback Levee model results (**Figure 4.24**). The model output from the 50-year peak flow event also predicts that the velocity will decrease by approximately 0.2 fps along the west side of the stockpile and decrease by a maximum of 2.1 fps at the upstream end of the stockpile (**Figure 4.25**). The velocity is predicted to increase by approximately 0.3 fps along the upper half of the east side of the stockpile. The velocity is also predicted to increase by approximately 0.2 fps in the area between the stockpile and the M&T Pump and in the main channel from the M&T Pumps to approximately 1,500 feet downstream from the pumps.

At the 100-year peak flow event, the model predicts that the water-surface elevation will increase by approximately 0.3 feet along the upstream side of the stockpile, will increase by approximately 0.2 feet along the western side and will decrease by approximately 0.2 feet along the downstream end of the stockpile compared to Phase III Setback Levee model conditions (**Figure 4.26**). The model output from the 100-year peak flow event also indicates that the velocity will decrease by approximately 0.2 fps along the west side of the stockpile and decrease by approximately 1 fps along the lower half of the east side of the stockpile. The velocity is predicted to increase by approximately 0.3 fps along the upper half of the east side of the stockpile (**Figure 4.27**). Similar to the 50-year model results, the velocity is also predicted to increase by approximately 0.2 fps in the area between the stockpile and the M&T Pump and in the main channel from the M&T Pump to approximately 1,500 feet downstream from the pump.

In summary, the predicted differences in water-surface elevations and velocities between the Phase III Stockpile model and the Phase III Setback Levee model conditions for both the 50- and 100-year peak flow events, indicate that the effects of the stockpile are limited to the approximate vicinity of the stockpile and the effects have a negligible effect at the M&T Pumps. In addition, the results at the 50- and 100-year peak flow events indicate that the Stockpile will not have any effect on the water-surface elevations or the velocities at the City of Chico outfall or at the alternative sites.

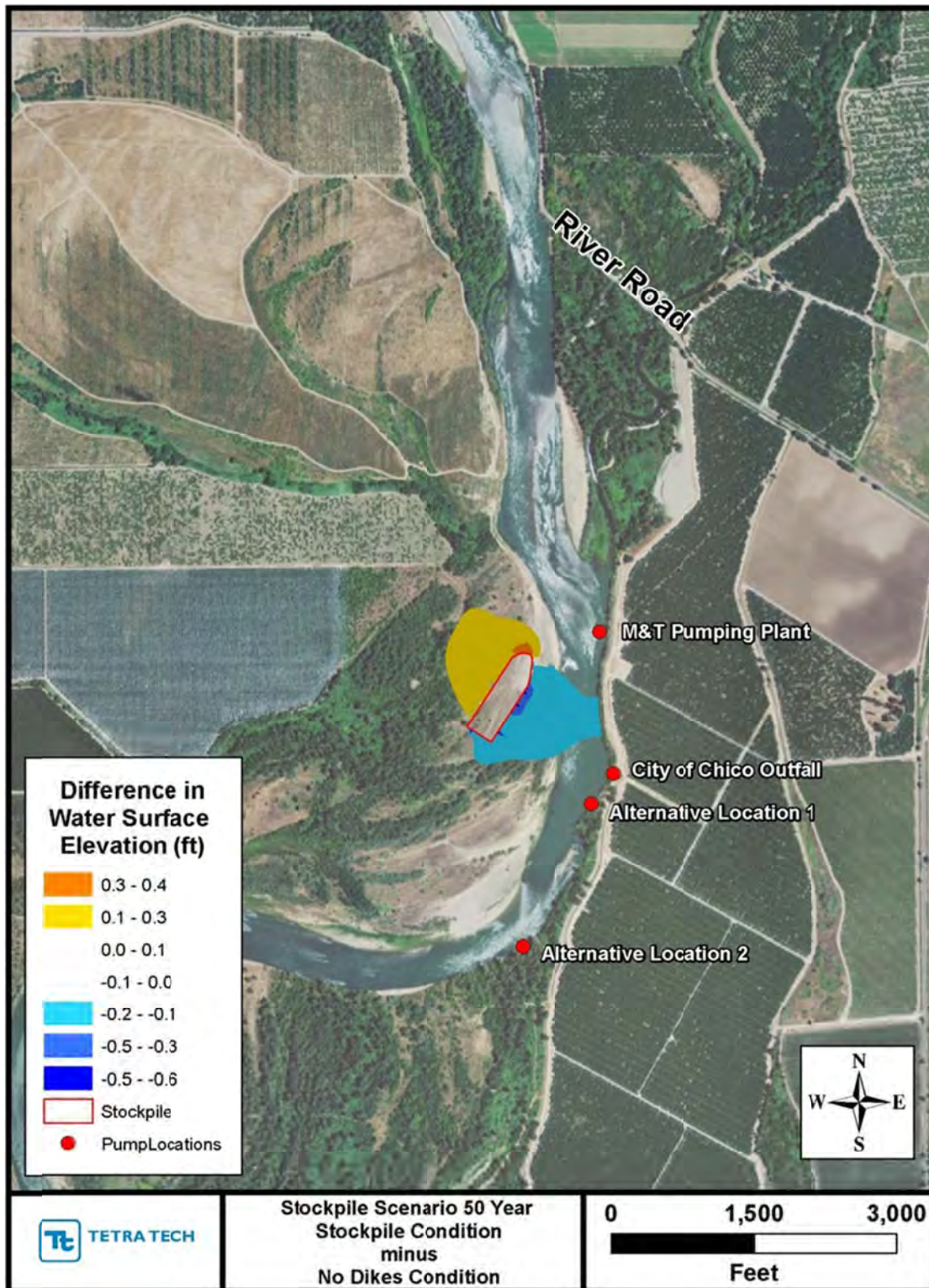


Figure 4.24. Difference in water-surface elevation at the 50-year peak flow event between the Phase III Stockpile and Phase III Setback Levee model conditions.

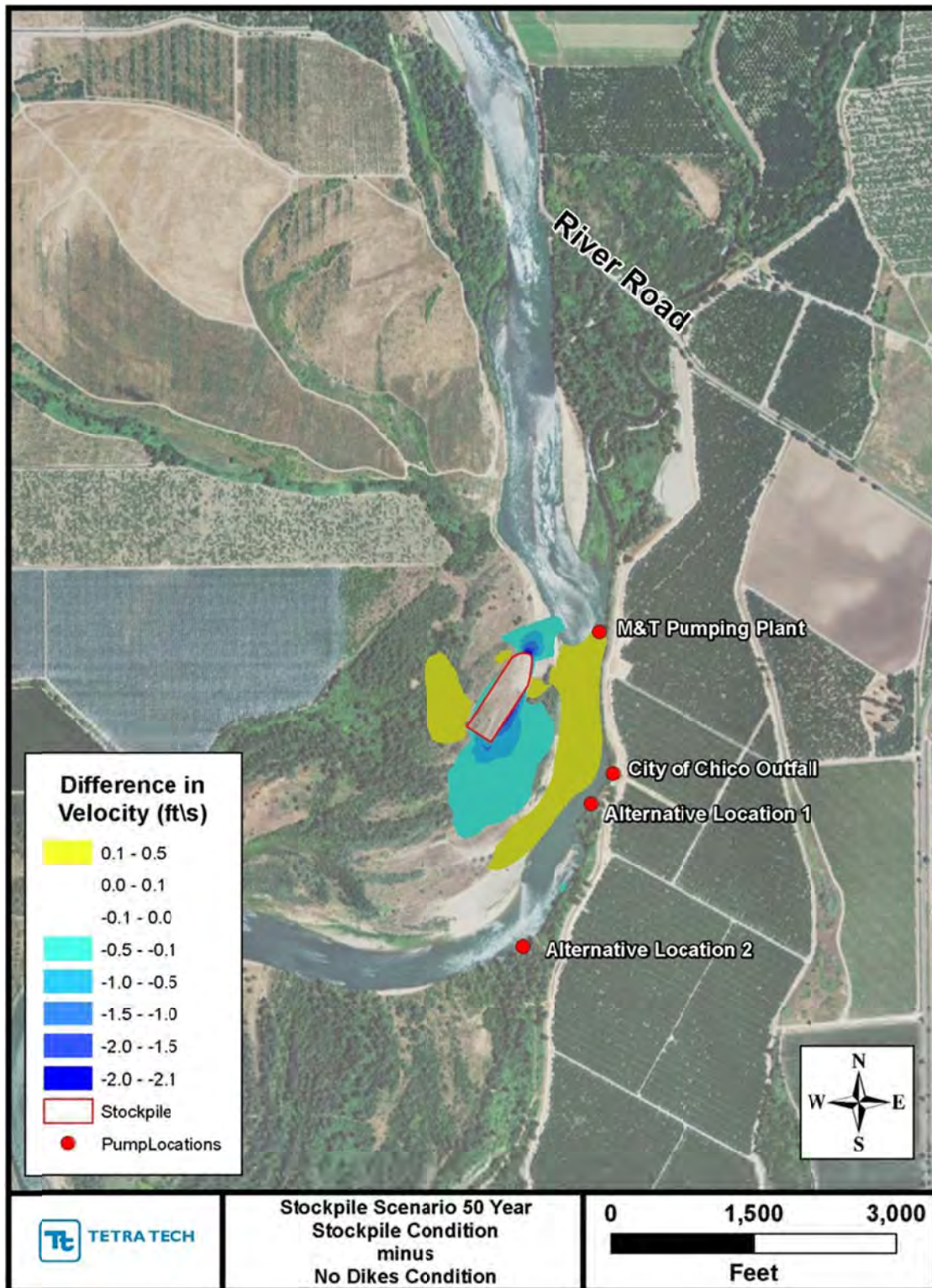


Figure 4.25. Difference in velocity at the 50-year peak flow event between the Phase III Stockpile and Phase III Setback Levee model conditions.

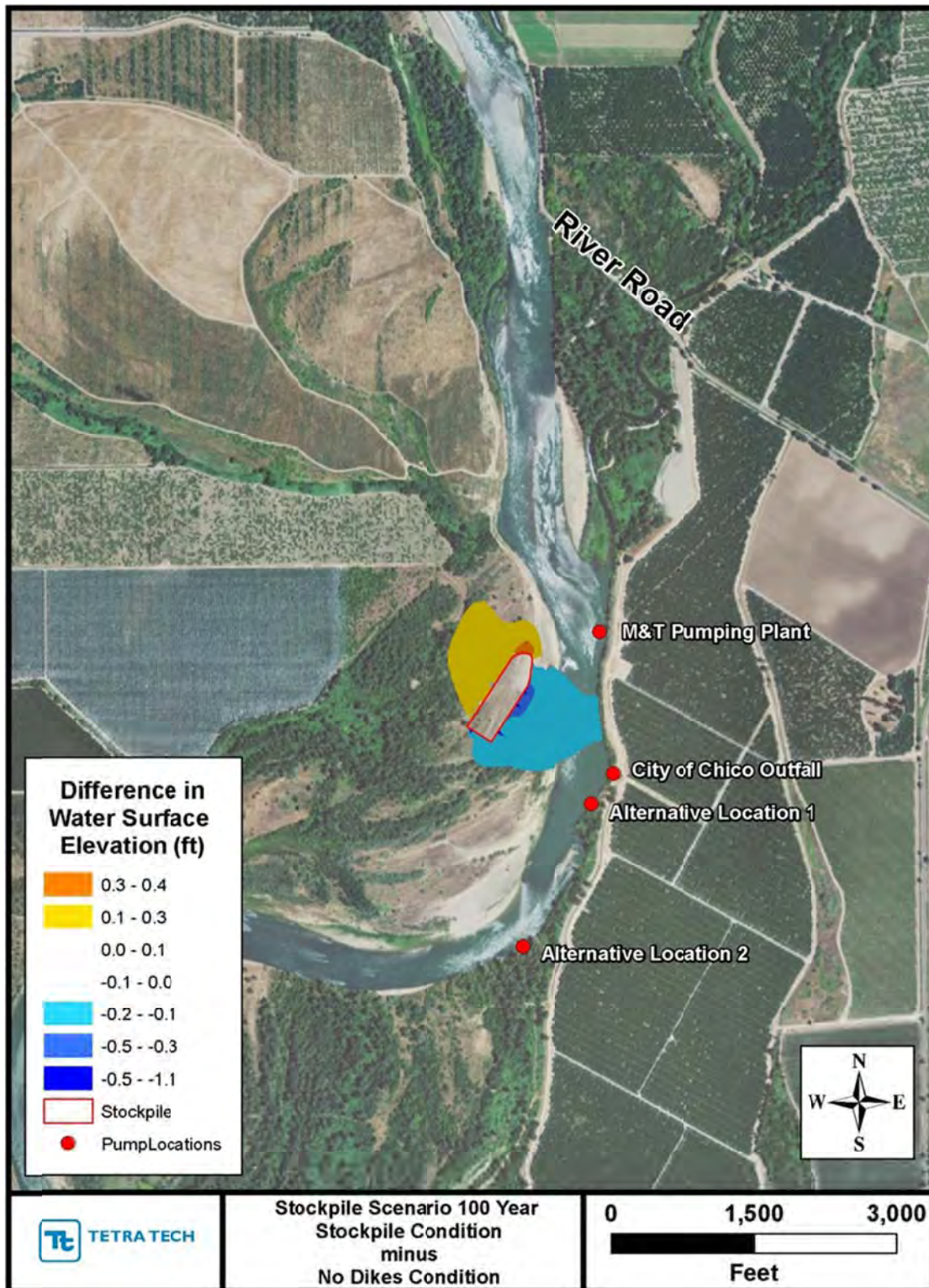


Figure 4.26. Difference in water-surface elevation at the 100-year peak flow event between the Phase III Stockpile and Phase III Setback Levee model conditions.

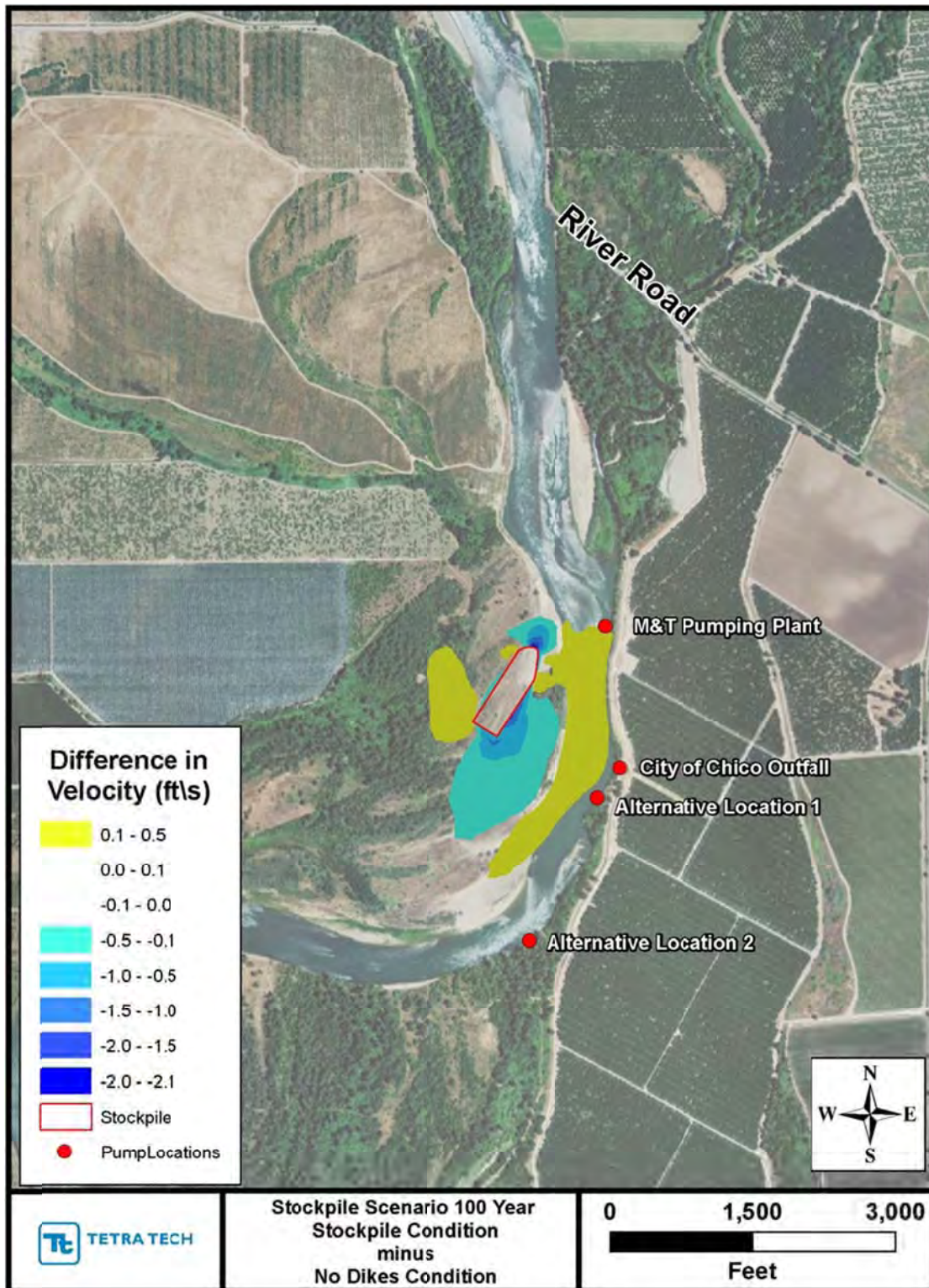


Figure 4.27. Difference in velocity at the 100-year peak flow event between the between the Phase III Stockpile and Phase III Setback Levee model conditions.

4.5.3 Incipient Motion Analysis

To determine the mobility of the gravel in the stockpile located on the floodplain, and in particular the mobility of the gravel located around the base of the stockpile, an incipient motion analysis was conducted using the hydraulic output from the 2-D model at the 50-year peak flow event ($Q=237,829$ cfs).

The incipient-motion analysis was performed by evaluating the effective shear stress on the stockpile material in relation to the amount of shear stress that is required to move the material. A representative bed material gradation ($D_{50}=39$ mm) was used in the analysis which was based on three pebble counts that were conducted on the bank-attached bar in December 2005 (MEI, 2006). The bank attached bar was excavated in 2000 and 2008 and it is assumed that the stockpile will contain the same sized material.

The normalized grain shear (NGS) stress (τ^*) for a specific discharge, which is defined as ratio of the grain shear stress (τ') to the critical shear stress (τ_c), provides a measure of the relative ability of that discharge to mobilize the bed material. Values of the normalized grain shear stress less than one (corresponding to a Shields value of 0.03) indicate that the bed material is not mobile and values greater than one indicate bed-material mobility. Additionally, when the normalized grain shear stress is between 1 and about 1.5 (Shields value of 0.045), the bed-material transport rate is very low. Under these marginal transport conditions, when the upstream supply is also low, the bed will armor and significant channel bed adjustments will not typically occur.

The normalized grain shear stress (τ^*) was computed using the sediment transport routines in SRH-2D at the 50-year peak flow event and using a Shields value of 0.03 and the representative grain size of 39 mm was applied to the entire reach of the main channel and to the gravel stockpile.

4.5.3.1 Results of the Incipient Motion Analysis of the Stockpile Scenario

The results of the incipient motion analysis indicate that the stockpile material is not mobilized at the 50-year peak flow event. The normalized grain shear (NGS) values around the base of the stockpile range from 0.1 near the downstream end of the stockpile to 0.4 at the upstream end of the stockpile (**Figure 4.28**); these NGS values are significantly lower than the NGS value of 1.5 required to mobilize the bed material.

The highest NGS values in the vicinity of the stockpile, occur approximately 350 feet due east of the upstream end of the stockpile along the west bank of the channel (Figure 5.31). In this location, the NGS values along the right bank and towards the center of the channel, range from approximately 1.5 to 2.2, indicating that significant sediment transport is occurring in this area.

Because the hydrodynamic conditions in the vicinity of the stockpile located on the floodplain are not sufficient to mobilize the stockpile material, a bank-edge stockpile scenario was developed to determine if the hydrodynamic conditions would be sufficient to mobilize a stockpile located on the right bank (west bank) in the area predicted to have high sediment transport.

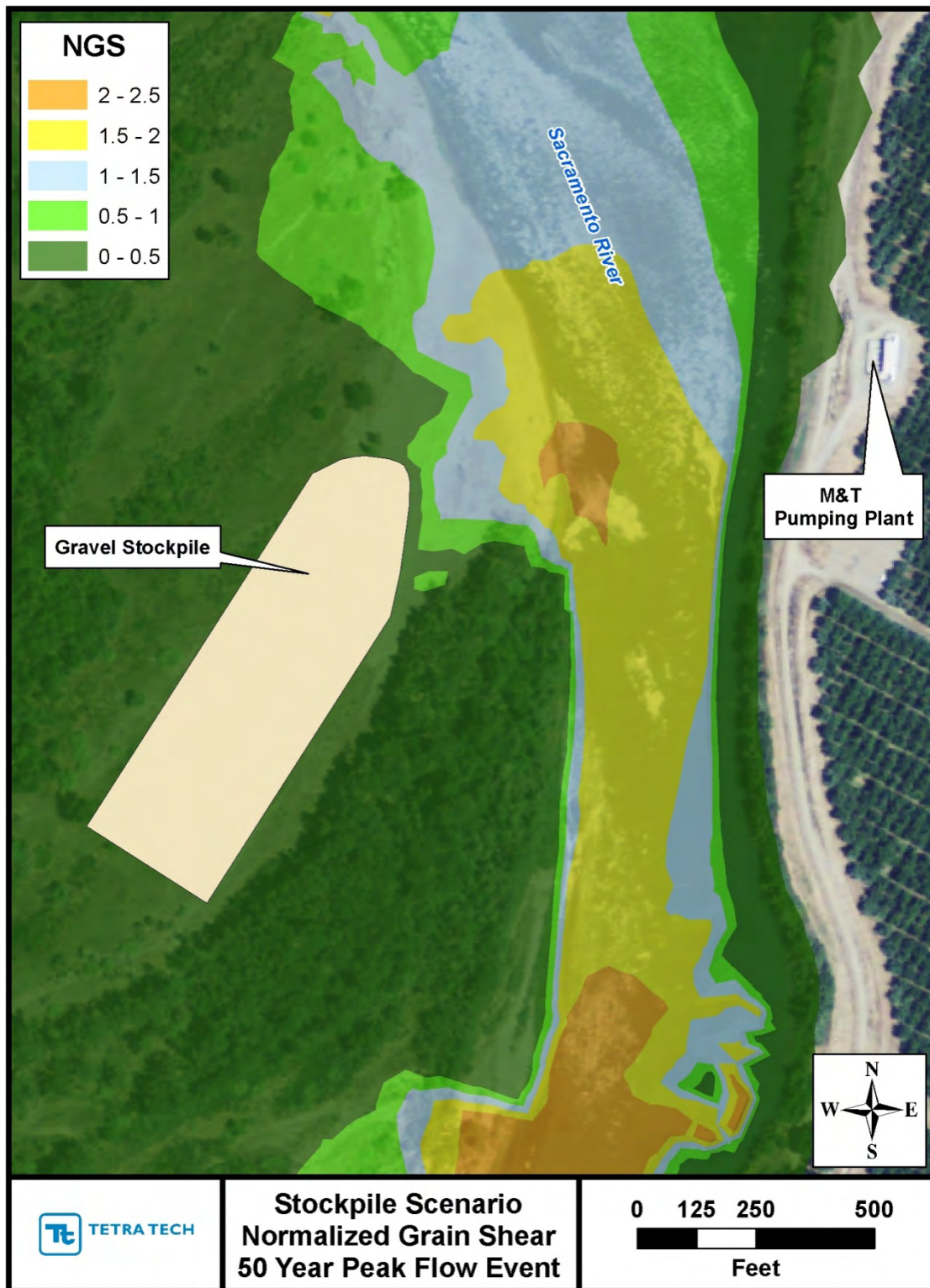


Figure 4.28. Normalized grain shear (NGS) distribution in the vicinity of the gravel stockpile at the 50-year peak flow event.

4.5.3.2 Bank-Edge Stockpile Scenario

A 1,200-foot long bank-edge stockpile was evaluated which extends from the downstream end of the longitudinal stone toe to just upstream of the thick stand of established trees (**Figure 4.29**). To maximize the potential sediment-transport rates, the toe of the bank-edge stockpile was located at the edge of water at a discharge of 5,000 cfs (this discharge corresponds to the lowest discharge modeled and it is exceeded 94.4 percent of the time based on the flow duration curve discussed in Chapter 2). The size of the bank-edge stockpile is 101,500 tons (compared to original stockpile of 100,000 tons), the sideslopes of the bank stockpile were set at the angle of repose (37 degrees) and the elevation of the top of the bank stockpile was set at the same elevation as the original stockpile.

It was hypothesized that if the shear forces are sufficient to mobilize the gravel at the toe of the bank stockpile, then the stockpile will erode backwards (at the angle of repose which is approximately 35 to 40 degrees for gravel) and will supply gravel to the river that will be then transported downstream by the river. However, as the bank-stockpile erodes away from the channel, the hydraulic energy and associated shear stresses acting on the higher elevation toe of the stockpile will decrease, and at some distance back from the original toe location, the shear stresses acting on the toe of the stockpile will decrease to the point that the toe will become stable.

The bank-edge stockpile scenario was run at the representative bankfull discharge of 90,000 cfs (2-year return interval) to evaluate the sediment transport conditions during relatively frequent flow events, and at 145,000 cfs which corresponds to approximately the 10-year peak flow event ($Q_{10} = 145,800$ cfs). Discharges greater than the 10-year peak flow event were not modeled, because these events were considered to occur too infrequently to transport significant amounts of material from the bank stockpile.

4.5.3.3 Results of the Incipient Motion Analysis of the Bank-edge Stockpile Scenario

Because, the SRH-2D software used for this analysis does not currently contain a bank erosion module and because the simulations were conducted under steady-state conditions, it was not possible to estimate the amount of erosion that would occur along the toe of the stockpile during a representative flood hydrograph. Therefore, to evaluate the incipient motion conditions following erosion of the stockpile, the toe of the stockpile was re-located 20 feet towards the bank and the eroded bank-edge stockpile conditions model was run at 90,000 and 145,000 cfs.

The results of the incipient motion analysis at 90,000 cfs indicate that the NGS values along the toe of the bank-edge stockpile range from 0.4 near the upstream end of the bank stockpile to 2.0 near the downstream end of the stockpile. The NGS value of 1.0 required for very low rates of sediment transport is exceeded along the toe of the bank stockpile for a distance of 290 feet near the downstream end of the stockpile (**Table 4.2**).

At 145,000 cfs, the NGS values along the toe of the bank-edge stockpile range from 0.4 near the upstream end of the stockpile to 2.5 near the downstream end. The NGS values are greater than 1.0 along 65 feet of the toe of the stockpile and are greater than 1.5 along approximately 410 feet of the bank stockpile near the downstream end of the stockpile (Table 1) (**Figure 4.30**).

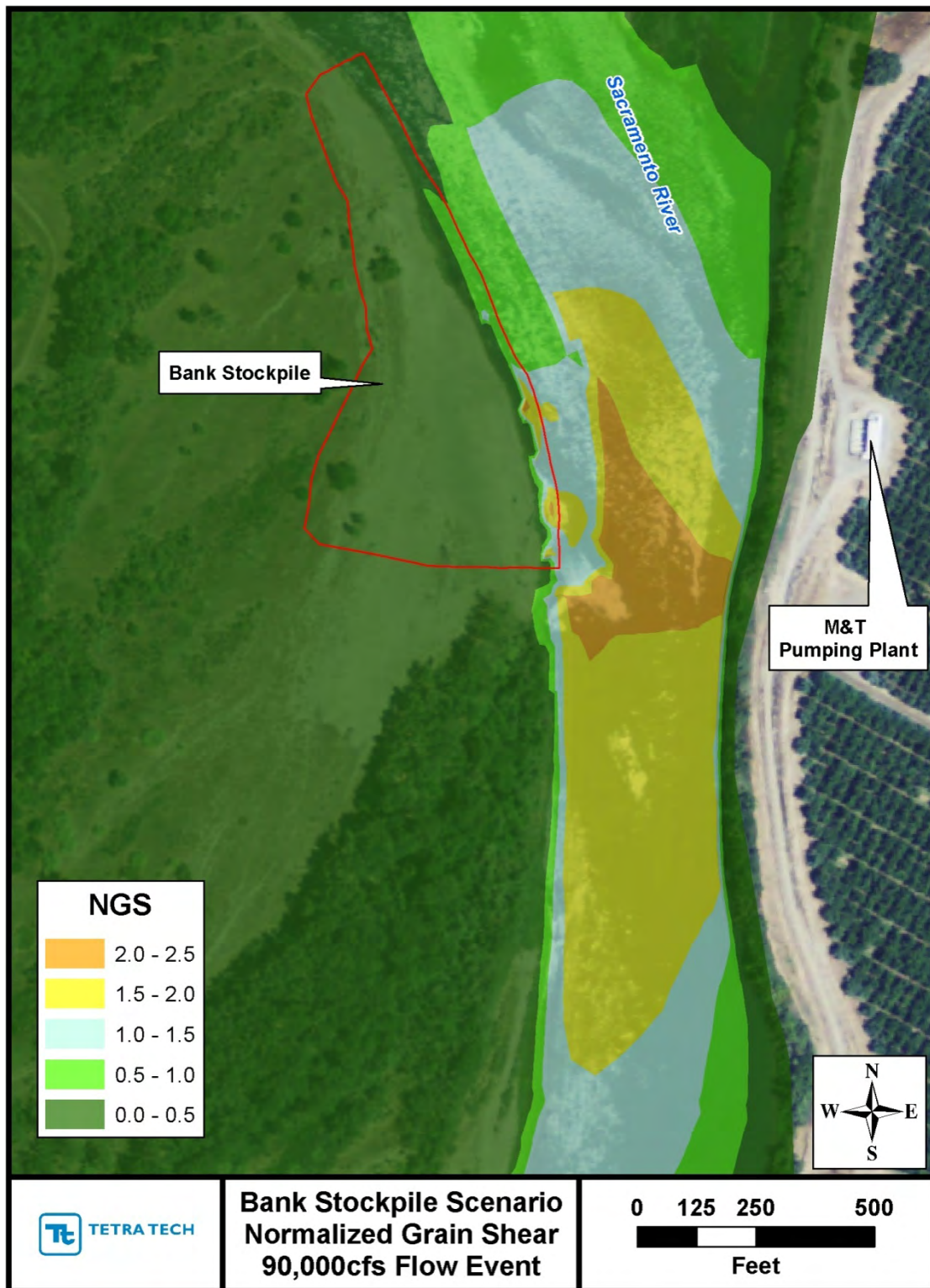


Figure 4.29. Normalized grain shear (NGS) distribution in the vicinity of the bank-edge stockpile at 90,000 cfs.

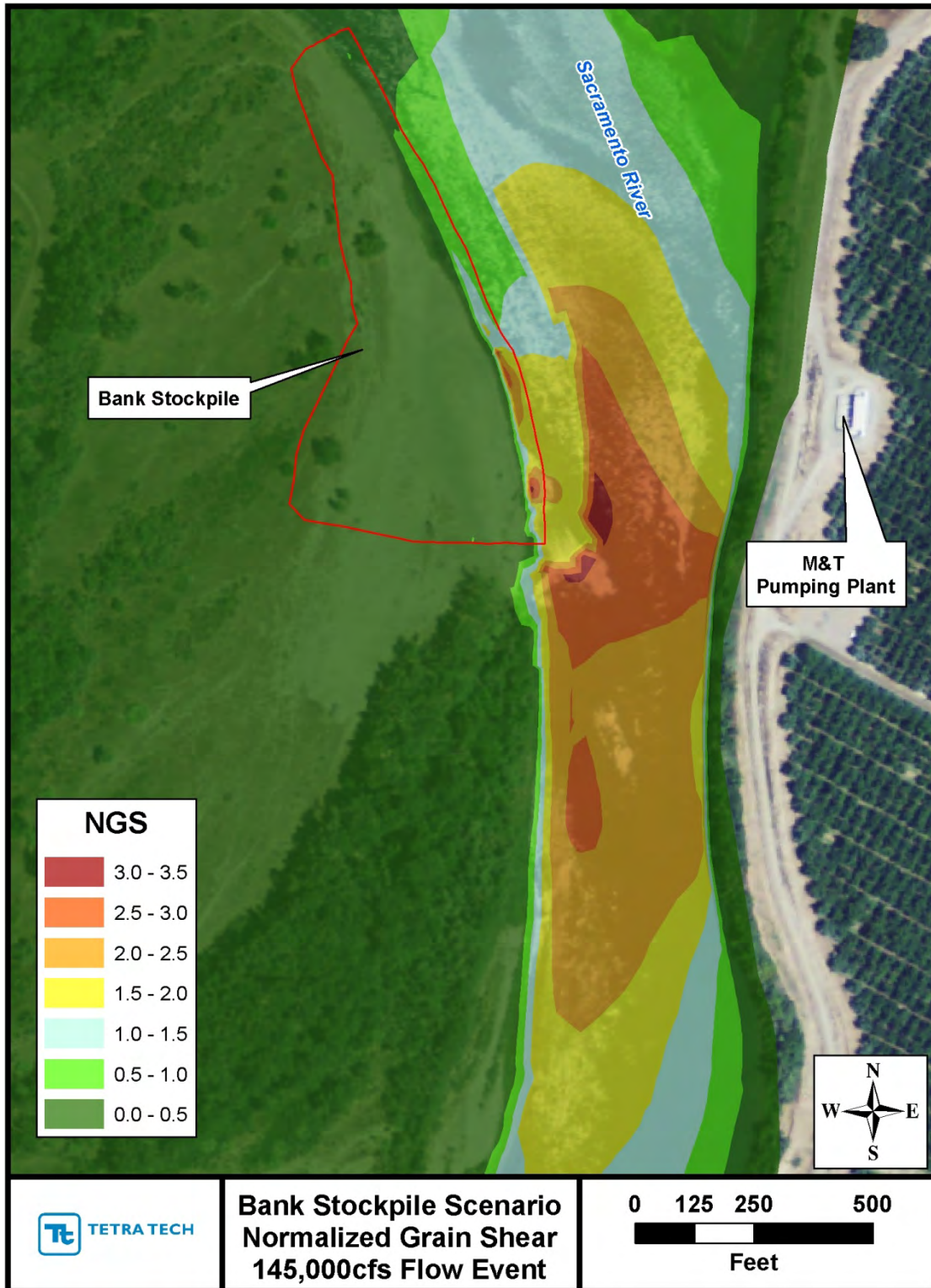


Figure 4.30. Normalized grain shear (NGS) distribution in the vicinity of the bank-edge stockpile at 145,000 cfs.

Table 4.2. Summary of the length along the toe of the stockpile that the NGS values exceed 1.0 and 1.5.			
	Discharge	$1 \leq \text{NGS} < 1.5$	$1.5 \leq \text{NGS}$
Bank-edge Stockpile Scenario	90,000 cfs	290	140
	145,000 cfs	65	410
Eroded Bank-edge Stockpile Scenario	90,000 cfs	330	40
	145,000 cfs	250	180

Under the eroded bank stockpile condition at 90,000 cfs, the NGS values of 1.0 required to mobilize the bed material are exceeded along the toe of the bank stockpile for a distance of 330 feet and the NGS values of 1.5 required for significant bed-material transport rates are exceeded along a length of 40 feet (**Figure 4.31**). The NGS values at 145,000 cfs exceed 1.0 along a length of 250 feet and the NGS values exceed 1.5 along a length of 180 feet (**Figure 4.32**).

In summary, the incipient motion analysis indicates that the stockpile located on the floodplain is stable at the 50-year peak flow event. Significant sediment-transport rates ($\text{NGS} \geq 1.5$) exist along 140 feet of the toe at 90,000 cfs and along 410 feet at 145,000 cfs under the bank-edge stockpile scenario. Under the eroded bank-edge stockpile condition, significant sediment-transport rates occur along 40 feet of the toe at 90,000 cfs and along 180 feet at 145,000 cfs. The reduction in length of sediment-transport rate from 410 feet under the bank-edge stockpile scenario to 180 feet under the eroded bank-edge stockpile scenario indicates that relatively little sediment will be eroded from the stockpile compared to the total volume of the stockpile during a significant flood event, and that significant erosion will only take place during relatively infrequent flood events.

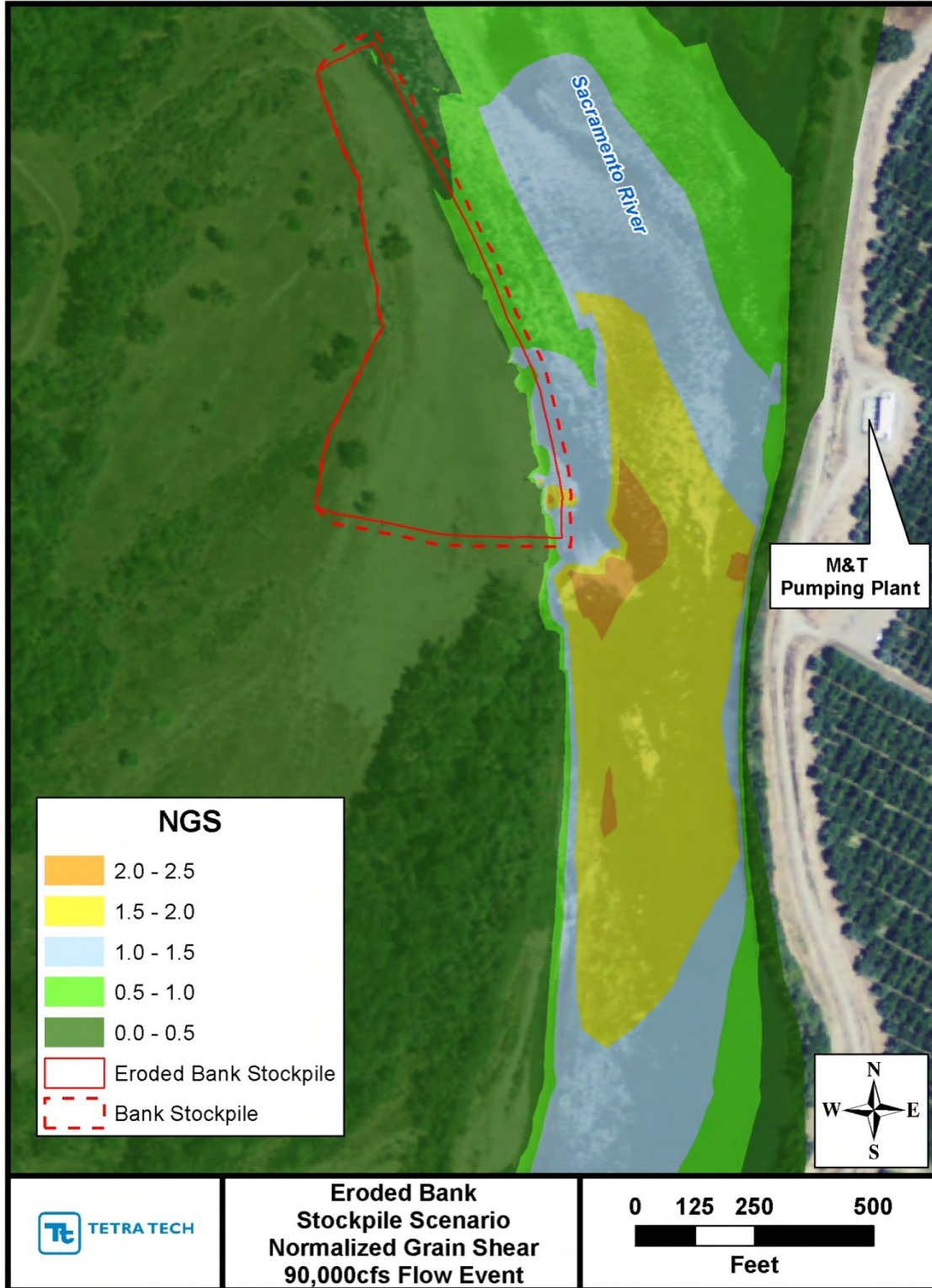


Figure 4.31. Normalized grain shear (NGS) distribution in the vicinity of the eroded bank-edge stockpile at 90,000 cfs.

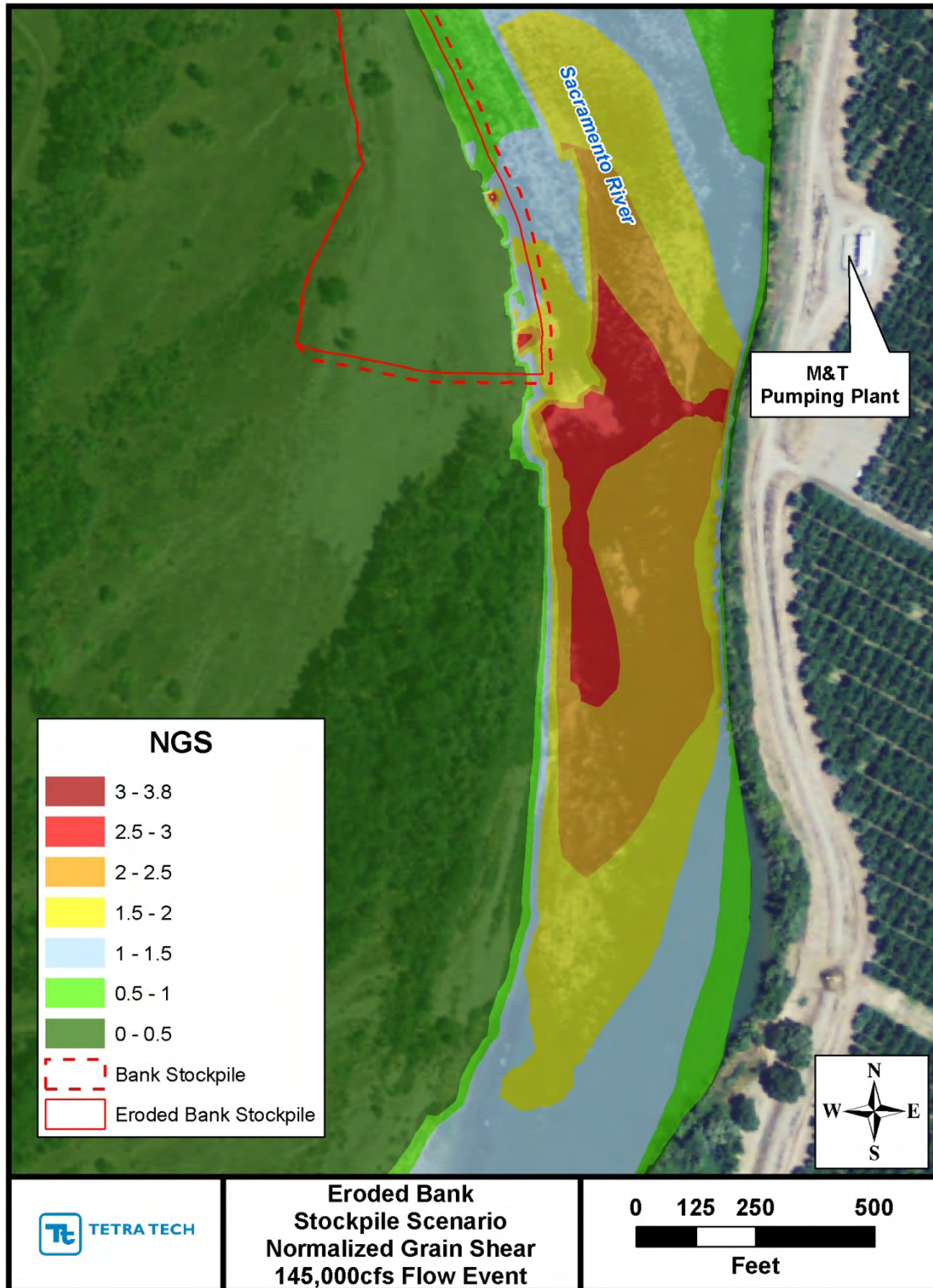


Figure 4.32. Normalized grain shear (NGS) distribution in the vicinity of the eroded bank-edge stockpile at 145,000 cfs.

5 SUMMARY AND CONCLUSIONS

5.1 Summary

Two-dimensional hydraulic modeling was conducted for Phase III of the M&T/Llano Seco Pumping Plant project. Based on review of the Phase II report (MEI, 2008), the Steering Committee recommended that additional 2-D modeling be performed to evaluate two alternative pump station locations. The Steering Committee also recommended that the 9-dike configuration be incorporated into the coarser scale 2-D model of the Setback Levee that extends from RM 192 to RM 202 (USACE, 2008) to evaluate the hydrodynamic impacts of the proposed spur dikes on Hamilton City Setback Levee project, and conversely the impacts of the Hamilton City Setback Levee project on the M&T reach and the spur dikes. In addition, an analysis was conducted to evaluate the hydrodynamic effects of locating a gravel stockpile on the west bank of the Sacramento River opposite the M&T Pumps.

The specific objectives of the Phase III study were to:

1. Investigate the impacts, if any, of the Hamilton City Setback Levee project at the M&T Pumps and the City of Chico outfall,
2. Evaluate two alternative pumping sites located 2,200 and 3,500 feet downstream from the existing site,
3. Investigate the impacts, if any, of the Hamilton City Setback Levee project on the potential long-term solution alternatives at the M&T Pumps and the City of Chico outfall, and conversely, investigate the long-term solution alternatives impacts on the Hamilton City Setback Levee project.
4. Investigate the hydrodynamic impacts of locating a gravel stockpile on the west overbank opposite the M&T Pump.

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

1. The existing conditions Phase II model (MEI 2008) was updated with the January 2010 survey data and was calibrated to water-surface elevations measured during the survey; this model is referred to as the Phase III model and was used to simulate flows up to 15,000 cfs. The Phase III model was incorporated into the coarser scale USACE (2008) Setback Levee model that extends from RM 192 to RM 202; this model is referred to as the Phase III Setback Levee model, and was used to simulate flows from 50,000 cfs to the 100-year peak flow event. The Phase III Setback Levee model and the Phase III J-Levee model were run at the 50- and 100-year peak flow events and the results were compared to evaluate the hydrodynamic changes in the M&T reach.
2. The Phase III and Phase III Setback Levee models were used to develop rating curves to evaluate the hydraulic conditions at the M&T Pumps, City of Chico outfall and at the Alternative 1 site (2,200-foot site). The Alternative Site 2 (3,500-foot site) is located approximately 1,270 feet downstream from the end of the riprap along a section of the left bank that has eroded approximately 200 feet back from the end of the riprap. As part of Alternative 2 site relocation (3,500-foot site), the left bank will be re-aligned to remove the

spur created by the riprap and eroding bank. A Phase III Bank Realignment model was developed to represent the realignment of the left bank and the model results were used to develop rating curves at the Alternative 2 site and to evaluate potential changes at the City of Chico outfall.

3. The 9-dike configuration was incorporated into the Phase III Setback Levee model to evaluate the hydrodynamic impacts of Hamilton City Setback Levee project on the M&T Pumps alternatives and the City of Chico outfall, and conversely, investigate the potential effects of the 9-dike configuration on the Hamilton City Setback Levee project. The Phase III Setback Levee model and the Phase III Dike-Setback Levee model were run at the 50- and 100-year peak flow events and the results were compared to evaluate the hydrodynamic changes in the M&T reach.
4. To evaluate the hydrodynamic impacts of creating a gravel stockpile on the right (west) overbank opposite the M&T Pumps, the Phase III Setback Levee model was modified to represent a 100,000-ton gravel stockpile that is 1,000 feet long by 300 feet high. This Phase III Stockpile model was run at the 50- and 100-year peak flow events and the results were compared to Phase III Setback Levee model results. An incipient motion analysis was performed to evaluate the stability of the gravels in the stockpile. In addition, the Phase III Setback Levee model was also modified to represent the inclusion of a gravel stockpile located on the edge of the west bank of the river opposite the M&T Pump. The Phase III bank-edge stockpile model was run at 90,000 and 145,000 cfs and an incipient motion analysis was conducted at these discharges to determine if the hydrodynamic conditions are sufficient to mobilize the gravel in the bank-edge stockpile.

5.2 Conclusions

This investigation led to the following conclusions:

1. Comparison of the hydraulic output for the Phase III Setback Levee model and Phase III J-Levee model at the 50- and 100-year peak flow events, indicates that the predicted water-surface elevations are a maximum of 2 and 1.8 feet higher, respectively, under the Phase III Setback Levee condition compared to Phase III J-Levee condition; these increases primarily occur to the east of the proposed Setback Levee. The largest decreases in water-surface elevations occur to the west of the Setback Levee. Similarly, the largest velocity increases of 2.3 fps at the 50-year peak flow and 2.5 fps at the 100-year peak flow event occur to the east of the Setback Levee and the largest decreases occur to the west of the Setback Levee. There are no predicted changes in water-surface elevation or velocity at the M&T Pumps, City of Chico outfall or at the two alternative pump sites, indicating that the Hamilton City Setback Levee Project will not affect the M&T/Llano Seco Pumping Plant project.
2. The predicted depth-discharge rating curves indicates that the M&T pumps site has the largest flow depths over the range of modeled flows from 5,000 to 275,900 cfs. The depth-discharge rating curves indicates the Alternative 1 and City of Chico outfall sites have similar depths over the range of modeled flows from 5,000 to 275,900 cfs and the flow depths are approximately 1.0 feet less than at M&T Pumps site over the range of modeled flows.
3. Under the bank realignment condition, the rating curve at the Alternative 2 site predicts the flow depths are approximately 4 feet less than at the M&T Pumps site over the range of modeled flows from 5,000 to 275,900 cfs. The Phase III Bank Realignment model predicts that the water-surface elevation at the City of Chico outfall will be 0.2 feet lower at 5,000 cfs and an average of 0.05 feet lower at flows from 10,000 cfs to the 100-year peak flow event.

4. The velocity-discharge rating curves indicate that the M&T Pumps have the highest velocities over the range of flows from 5,000 to 109,322 cfs (December 2005 peak flow) compared to the other sites. At discharges greater than 109,322 cfs, the velocity-discharge rating curve becomes flat. The velocity-discharge rating curves at the City of Chico outfall and Alternative 1 sites have similar velocities over the range of flows from 5,000 to 134,638 cfs (January 2006 peak flow). In general, the velocities at the Alternative 2 site are lower compared to the other sites over the entire range of modeled flows. The velocities at the 100-year peak flow event at M&T Pumps, City of Chico outfall, Alternative 1 site and Alternative 2 site are 5.7, 6.4, 7.0 and 5.1 fps, respectively. Under the bank realignment condition, the velocities at the City of Chico outfall increase by approximately 0.15 fps in the range of flows from 5,000 to the 109,322 cfs, and are approximately the same at discharges higher than 109,322 cfs.
5. The shear stress-discharge rating curves show similar trends to the velocity-discharge rating curves. The shear stress values at the M&T Pumps steadily increase up to 90,000 cfs and then flatten out at higher discharges. The shear stress values at the City of Chico outfall and the Alternative 1 site steadily increase up to 134,638 cfs and then flatten out slightly at higher discharges. The shear stress at the Alternative 2 site steadily increases up to 50,000 cfs and then flattens out slightly at higher discharges. The shear stresses at the 100-year peak flow event at M&T Pumps, City of Chico outfall, Alternative 1 site and Alternative 2 site are 0.35, 0.46, 0.56 and 0.31 lb/ft², respectively. Under the bank realignment condition, the shear stresses at the City of Chico outfall increase by approximately 0.02 lb/ft² in the range of flows from 5,000 to the 100-year peak flow event. Comparison of the depth-discharge, velocity-discharge and shear stress discharge rating curves at the City of Chico outfall for the Phase III Setback Levee and Phase III Realignment scenarios indicates that there will be very little change in hydrodynamic conditions at the City of Chico outfall. The design criteria of the City of Chico outfall were not available to evaluate the hydrodynamic effects of the bank realignment on the outfall, however, the very small changes in hydrodynamic conditions indicates that the bank realignment will not likely affect the City of Chico outfall.
6. Comparison of the hydraulic output for the Phase III Setback Levee model and Phase III Dike-Setback Levee model at the 50- and 100-year peak flow events, indicates that the predicted water-surface elevations are a maximum of 1.1 and 3 feet higher, respectively, under the Phase III Dike-Setback Levee condition compared to Phase III Setback Levee condition. The maximum velocity increases of 3.5 fps occur over the top of the dikes and the maximum flow decreases of 5 fps occur between the dikes for both the 50- and 100-year peak flow events. The dikes increase the flow velocity over the bank-attached bar by approximately 0.8 and 1 fps at the 50- and 100-year peak flow events, respectively. The velocities at the M&T Pumps increase by 0.2 fps at both the 50- and 100-year peak flow events. The predicted differences in water-surface elevation and velocity between the with-dikes condition and the without-dikes condition for both the 50- and 100-year peak flow events, indicate that the influence of the dikes is limited to the area in the vicinity of the dikes and the effects of the dikes do not extend across to the Setback Levee or up to River Road.
7. Comparison of the hydraulic output from the Phase III Stockpile model with the Phase III Setback Levee model indicates that at the 50-year event, the water-surface elevations will increase by a maximum of 0.25 feet along the upstream side of the stockpile and decrease by approximately 0.1 feet along the downstream end of the stockpile. At the 50-year peak flow event, the model predicts that the velocity will decrease by a maximum of 1.5 fps at the upstream end of the stockpile and along the lower half of the east side of the stockpile, and the velocities will increase by approximately 0.3 fps along the upper half of the east side of the stockpile. At the 100-year peak flow event, the model predicts that the water-surface

elevation will increase by approximately 0.3 feet along the upstream side of the stockpile and will decrease by approximately 0.2 feet along the downstream end of the stockpile. The model output from the 100-year peak flow event also predicts that the velocity will decrease by approximately 0.2 fps along the west side of the stockpile, decrease by approximately 1 fps along the lower half of the east side of the stockpile and will increase by approximately 0.3 fps along the upper half of the east side of the stockpile. The velocity is predicted to increase by approximately 0.2 fps in the area between the stockpile and the M&T Pumps and in the main channel from the M&T Pumps to approximately 1,500 feet downstream at the 50- and 100-year peak flow events. The effects of the stockpile do not extend to the City of Chico outfall or to the two Alternative pump sites. The effects of the stockpile do not extend upstream to River Road or across to the Setback Levee. The incipient motion analysis indicated that the stockpile is stable at the 50-year peak flow event.

8. The incipient motion analysis of the bank-edge stockpile at 90,000 and 145,000 cfs indicated that relatively little sediment would be eroded from the stockpile compared to the total volume of the stockpile during a significant flood event, and that significant erosion will only take place during relatively infrequent flood events.

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