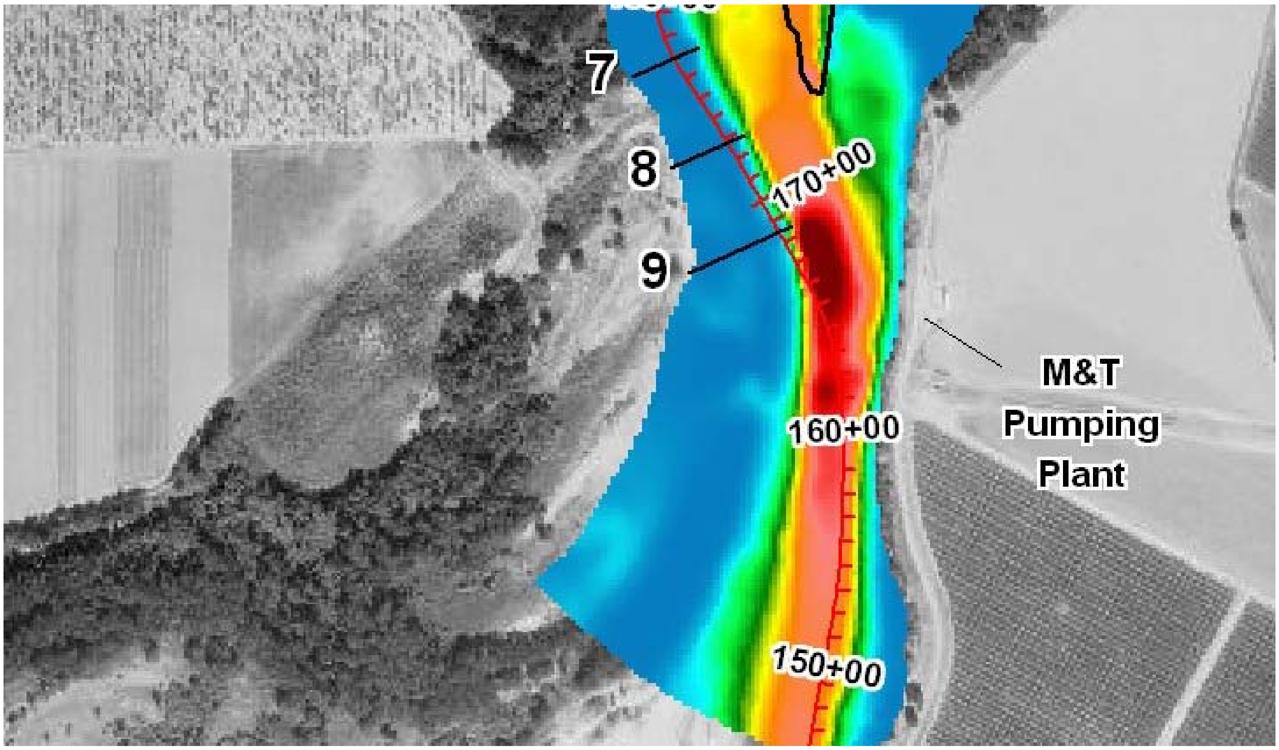
Two-dimensional Modeling to Evaluate Potential River Training Works at M&T Pumping Plant Sacramento River, RM 192.5 (2005 Topography and Bed Material)



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April 18, 2006

EXECUTIVE SUMMARY

The objective of this hydraulic and sediment-transport investigation of the Sacramento River between River Mile (RM) 192.5 and RM 194.4 (Figure 1.1) was 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 recreate hydrodynamic conditions that will permit sustainable operation of the pumps for the next 40 years.

Three specific questions were addressed by the study:

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?

An existing two-dimensional (2-D) hydrodynamic model (RMA2) (MEI, 2005) was modified to represent the current (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 (Figures 3.1 and 3.2):

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)
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 that were 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. Cost estimates for permitting, construction, mitigation and operation and maintenance were developed for the three with-dike scenarios.

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,
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,

4. If the dikes are constructed from new rock, and full mitigation is required for the 3,200 feet of affected bankline, the costs for Scenarios 2, 3 and 4 are $7.9M, $8.7M and $13.4M, respectively.
5. If mitigation can be offset by removal of an equivalent length of existing bank protection on Golden State Island that is owned by the M&T Ranch, and the recovered rock is incorporated into the spur dikes, costs for Scenarios 2, 3 and 4 decrease to $5.1M, $5.5M and $10.2M, respectively.

Because spur dikes are not commonly used on the Sacramento River, there is little information available to assess their performance for river stabilization or their environmental impacts or benefits. A 5-year Adaptive Management Experiment is proposed to inform future use of these structures where infrastructure protection is required as envisaged in the House Bill 1086 process. The likely cost of the experiment is $345,000. Physical modeling of any selected dike scenario is highly recommended to validate the numerical model results at the fish screens and pump inlets. Physical modeling of a selected dike alternative could be conducted for approximately $190,000.

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1. INTRODUCTION

In 1997, the M&T/Llano Seco Pumping and Fish Screen Facility was moved 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 at the mouth of Big Creek, just upstream from the intake.

Based on the available aerial photography and survey measurements, the west bank eroded by up to 330 feet just upstream from the intake between 1996 and 2006. In 2000, 189,000 yd^3 of material was dredged from the gravel bar as a short-term solution to limit sedimentation at the pump inlet. Previous work by the Steering Committee 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). A significant conclusion from the Steering Committee report was, as follows:

> *The sediment-transport analyses confirms that the locus of sediment deposition on the bar immediately upstream of the M&T pump inlets is due to local hydraulic conditions that favor deposition. These conditions can be expected to persist under the existing channel morphology, and will most likely become worse if the right bank is allowed to continue to erode. If the difference in sediment-transport capacities at the head and toe of the bar is a reasonable estimate of the volume of material deposited on the gravel bar on an average annual basis, then the bar could rebuild to its 2000 pre-dredged configuration within about four years. On the other hand, if an infrequent flood event like the 1974 flood were to occur (a 2-percent chance exists of a flood of this magnitude occurring), the bar could be rebuilt within a single event. Given the difficulties associated with securing permits for dredging, and the need to find disposal areas for the dredged sediments, the status quo almost certainly cannot be maintained.*

The Steering Committee report also suggested that a possible solution to the problem could include a series of eight 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.

At the request of the Steering Committee, Mussetter Engineering, Inc. (MEI) conducted a two-dimensional (2-D) hydrodynamic modeling and sediment-transport study of the Sacramento River from about RM 192 to RM 194 (MEI, 2005). This analysis demonstrated that the proposed spur dike configuration that included eight rock dikes should create hydraulic conditions within the reach that would prevent downstream migration of the gravel bar during high flows and prevent build-up of sands at the intake during lower flows when the pumps are generally operated (4,000 to 14,000 cfs). The modeling was, however, conducted with 1996 in-river topography that was modified to approximate the 2000 bar dredging and the bankline shown on 2003 aerial photography. As a result, modeled conditions did not accurately represent existing conditions at the site, especially since there has been localized retreat of the right bank of up to 90 feet since the 2003 aerial photography was obtained. The Steering Committee,

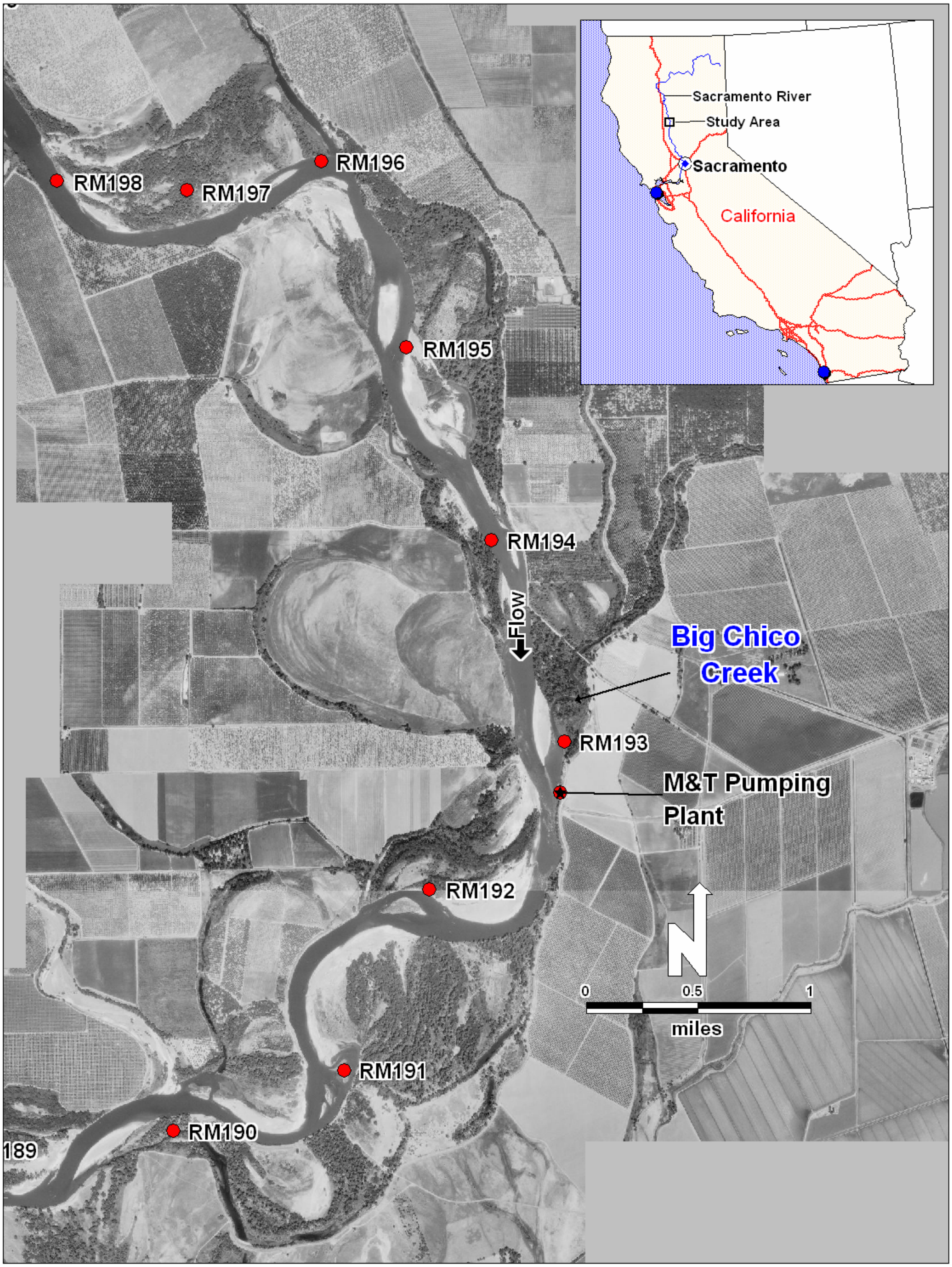

Figure 1.1. Location of the Sacramento River and the study site.

therefore, concluded that in-channel surveys were required to provide existing conditions topography, and the hydrodynamic modeling needed to be repeated using the new topography to further evaluate the utility of the proposed spur dike alternative. They also recommended that the model be used to refine the configuration of the dike field to improve the likelihood that it will meet project objectives. This report describes the methods and results obtained from the recommended analysis. In addition, a series of photographs showing examples of rock spurs that have been used at various locations throughout the United States was compiled, and these photographs are provided as part of this report to provide the reader with additional information on the general appearance and use of these installations in other settings (**Appendix A**).

The work performed for this study included the following tasks:

1. Information regarding the hydrology of the Sacramento River and Big Chico Creek was obtained from the U.S. Geological Survey (USGS) for the Sacramento River at Hamilton City gage and from the California Data Exchange Committee (CDEC) for the Big Chico Creek gage. The work performed for this study consisted primarily of updating and refining the analyses that were presented in the MEI (2005) report.

2. A bathymetric survey of the river between approximately RM 190 and RM 195 was conducted in December 2005.

3. The 2-D hydrodynamic model that was developed using RMA2, Version 4.5 (U.S. Army, Engineer Research and Development Center, Waterways Experiment Station, 2000) with Version 8.1 of the BOSS Surface Water Modeling System (SMS) graphical user interface (BOSS International, Inc. and Brigham Young University, 2004) was updated with 2005 bathymetric survey data. The 2-D model was used in conjunction with a 1-D model (HEC-RAS, USACE, 2002a) that was developed using the 2005 bathymetry data to validate Manning's *n*-values and to provide downstream boundary conditions for the 2-D model. The validated 2-D model was then used to evaluate hydraulic and sediment-transport characteristics in the vicinity of the M&T intake for topographic conditions that existed at the time of the mapping.

4. Three with-dike project condition models were developed by modifying topography in the 2005 model to reflect the dike configurations that were proposed in the original Steering Committee report (8 dikes) and a configuration that included 9 dikes and a 9-dike configuration with the ends of the dikes extended to the location of the 1996 bankline. These models were used to evaluate the potential effects of the dikes field.

1.1. Authorization

This was carried out by Mussetter Engineering, Inc. (MEI) under a contract agreement with Ducks Unlimited. MEI technical 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. (Colorado), Staff Engineer

2. HYDROLOGY

The analysis of the discharge regime in the study reach that was performed by MEI (2005) was updated for this study by adding data collected since 2003 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. Additional data for the Big Chico Creek gage (California Data Exchange Gage ID. BIC), which is located approximately 11 river miles northeast (upstream) of the confluence with the Sacramento River were also collected.

Data used to update flood-frequency curve for the post-Shasta Dam (1946-2006) period at the Hamilton City gage (**Figure 2.1, Table 2.1**) included the provisional peak discharge of 134,600 cfs that was recorded on January 1, 2006. 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 peak discharges are about 71,000 and 90,000 cfs, respectively. (The previous analysis indicated peak discharges for these recurrence intervals of 72,000 and 90,000 cfs, respectively.)

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)
59,500	1.2
71,000	1.5
90,000	2
132,200	5
148,200	10
169,900	50
374,100	100

The mean daily flow-duration curve for the Hamilton City gage was also updated using the available record of complete water years, WY1964-WY1980, from the USGS data, WY1997-WY2000 and WY2005 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 9,760 cfs, and the 10- and 90-percent exceedence flows are 25,500 and 5,920 cfs, respectively. (These values vary somewhat from the previously reported values of 9,000, 23,160, and 5,460 cfs, respectively, because the previous values were inadvertently developed using a data set that included the pre-Shasta Dam period back to 1946.) Annual runoff past the Hamilton City gage during the 24-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 10.6M ac-ft per year (**Figure 2.3**).

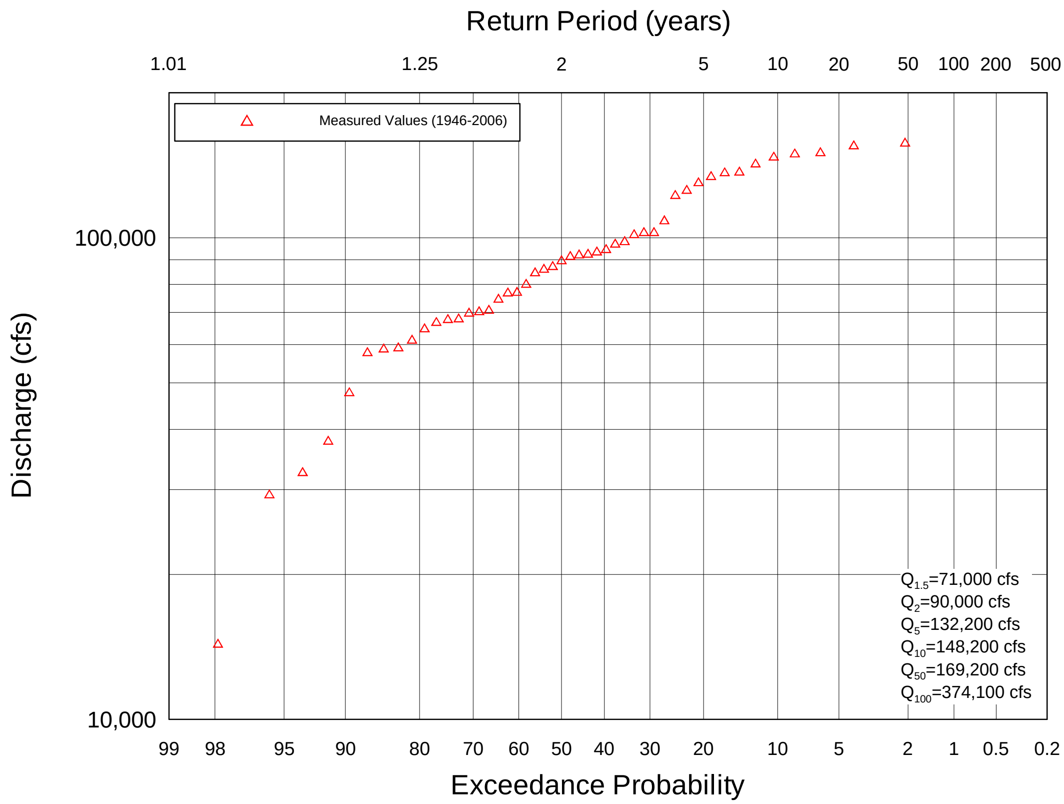

Figure 2.1. Post-Shasta Dam flood-frequency curve for Hamilton City gaging station.

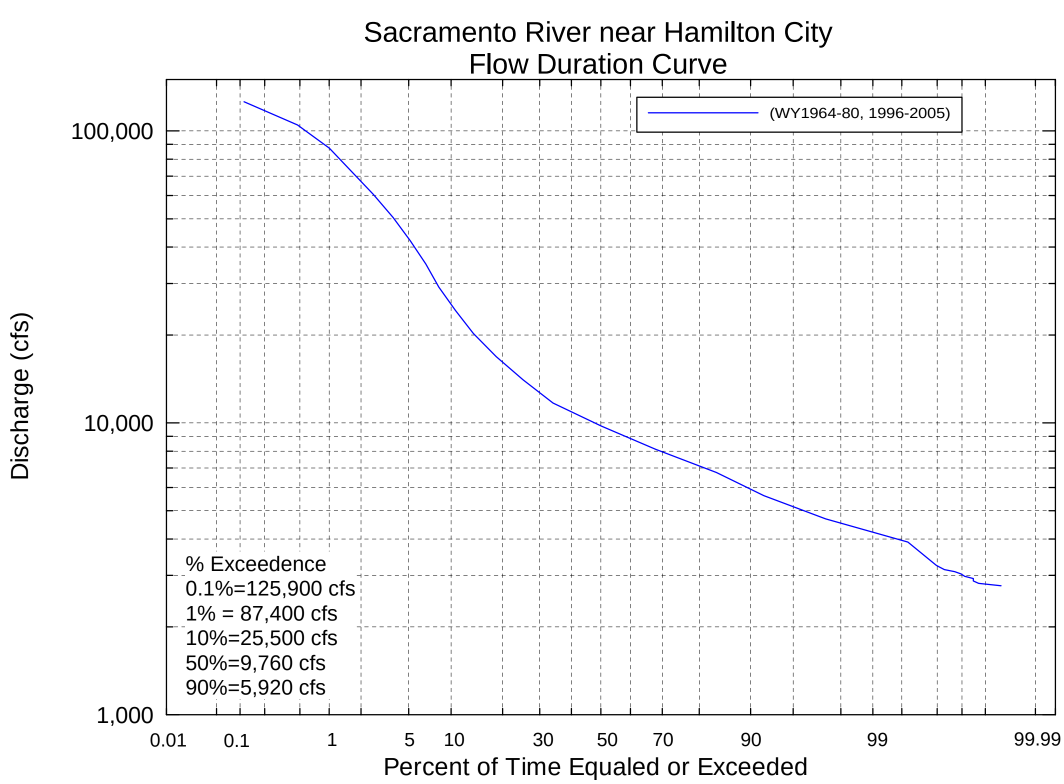

Figure 2.2. Post-Shasta flow-duration curve for Hamilton City gaging station.

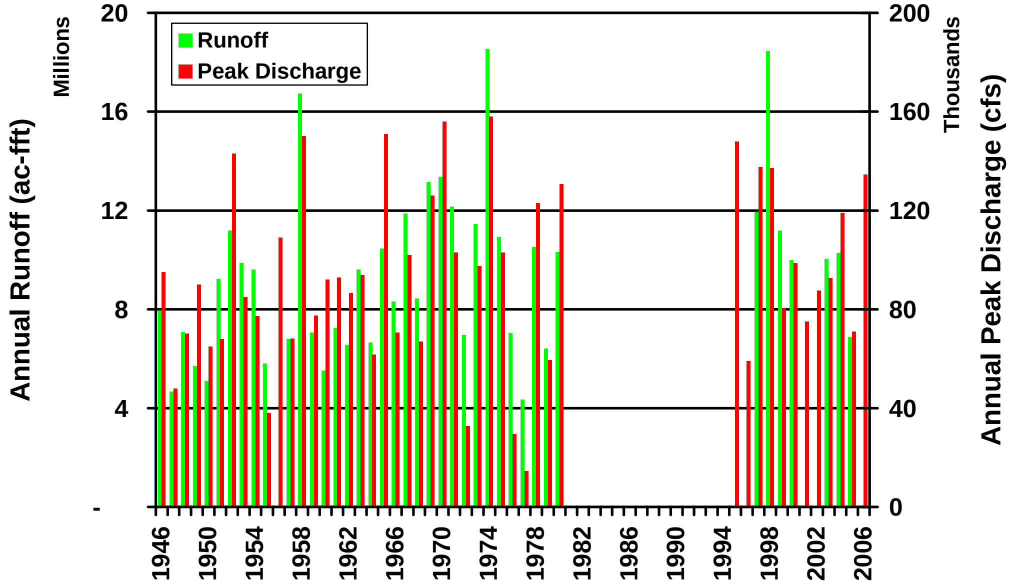

Figure 2.3. Annual runoff volumes and peak discharges at the Hamilton City gaging station from 1946 to 2005. No data are available for the 1981 to 1984 period and complete water years were not available for 1995, 1996, 2001 and 2002.

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, and 1998. Based on the flow records at other gages on the Sacramento River, large floods also occurred in 1983 and 1986. The formation and migration of the bar is very likely related to the occurrence of these high-magnitude flows.

3. TWO-DIMENSIONAL HYDRODYNAMIC MODELING AND SEDIMENT-TRANSPORT ANALYSIS

Two-dimensional (2-D) hydrodynamic models of the Sacramento River in the vicinity of the M&T pumping plant were developed to evaluate the effects of a series of spur dikes along the right (west) bank of the Sacramento River that were proposed in the original Steering Committee report to prevent further westward migration of the right bank and downstream migration of the gravel bar at the mouth of Big Chico Creek. The modeling was carried out using RMA2, Version 4.5 (U.S. Army, Engineer Research and Development Center, Waterways Experiment Station, 2000) with Version 8.1 of the BOSS Surface Water Modeling System (SMS) graphical user interface (BOSS International, Inc. and Brigham Young University, 2004). RMA2 is a depth-averaged, finite element, hydrodynamic model that computes water-surface elevations and horizontal velocity components for subcritical, free-surface flow in two-dimensional flow fields. RMA2 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. RMA2 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 than can be obtained from the more simplified 1-D models, and because it has been successfully applied on other projects in the Sacramento River system.

To improve the study team's understanding of the dynamics of the river in the study reach and to evaluate the effectiveness of the proposed dike configuration, models were developed and applied for the following scenarios:

1. 2005 channel alignment and geometry (baseline conditions),
2. Proposed 8-dike configuration (**Figure 3.1**)
3. Proposed 9-dike configuration (Figure 3.1)
4. Extended 9-dike configuration (**Figure 3.2**)

Results from these models were used to evaluate incipient motion and bed-material transport conditions in the study reach. As will be explained in a later section of this report, the results from the 2005 conditions analysis with the 8- and 9-dike configurations (Scenarios 2 and 3), indicate that the area in the vicinity of the pump intake would remain depositional, even in the presence of the dikes.

An additional 9-dike model was, therefore, developed by extending the three downstream dikes to the location of the 1996 bank line and increasing the height of the three dikes to the top of the bank (Scenario 4) to determine if these extended and raised dikes would prevent sediment deposition in the vicinity of the pumps.

3.1. Baseline (2005) Conditions Models (Scenario 1)

The 2005 baseline conditions 2-D model was developed to provide a means of calibrating the model to the observed water-surface elevations and to provide a basis of comparison for evaluating the effectiveness of the proposed dike fields.

3.1.1. Topographic Data

The 2-D finite element model is based on a mesh composed of triangular and quadrilateral elements with corner and mid-point nodes that represent the planform geometry and channel

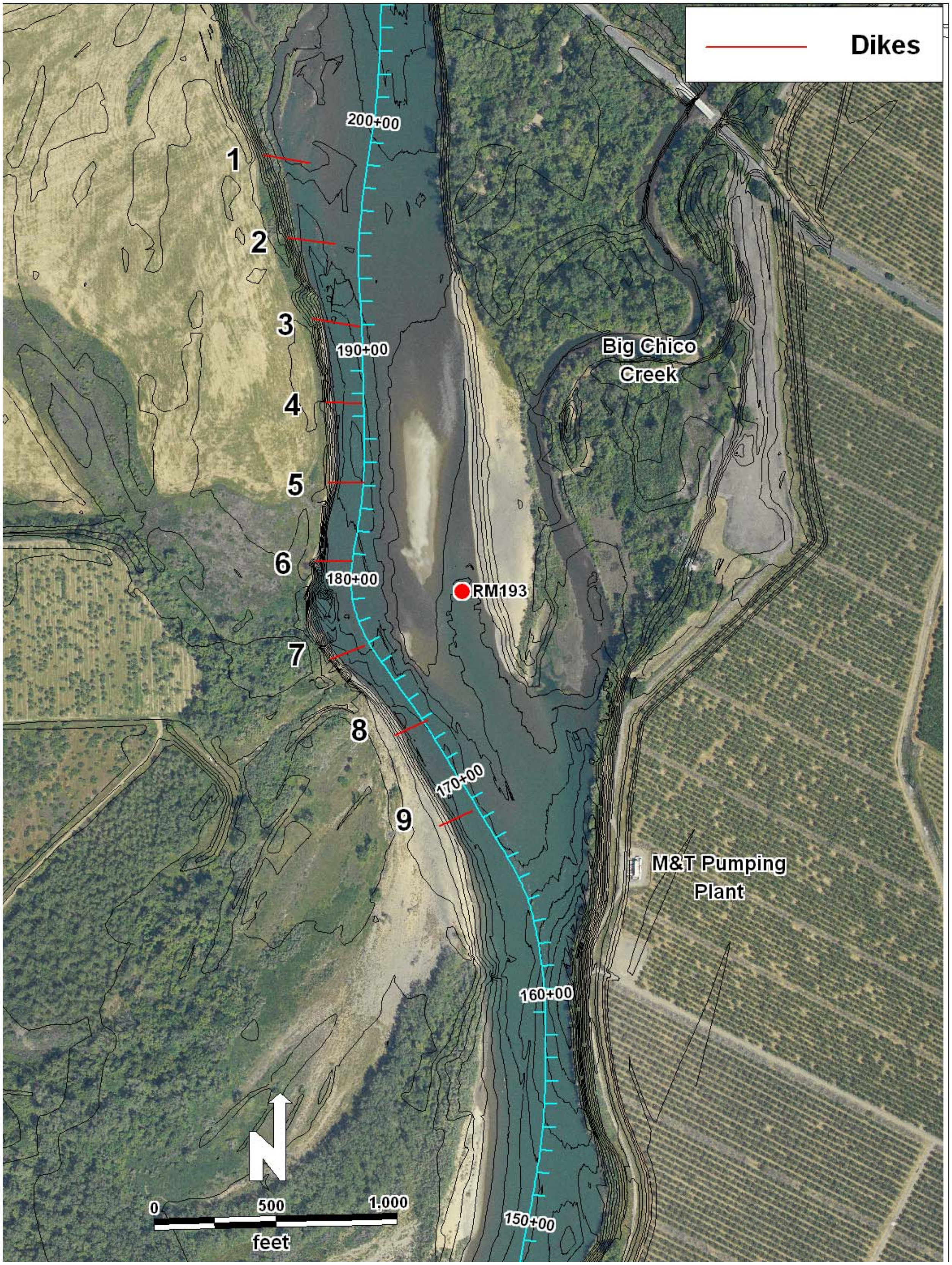

Figure 3.1. Aerial photograph of the study reach showing the dike configurations for Scenario 2 (Dikes 1 through 8) and Scenario 3 (Dikes 1 through 9).

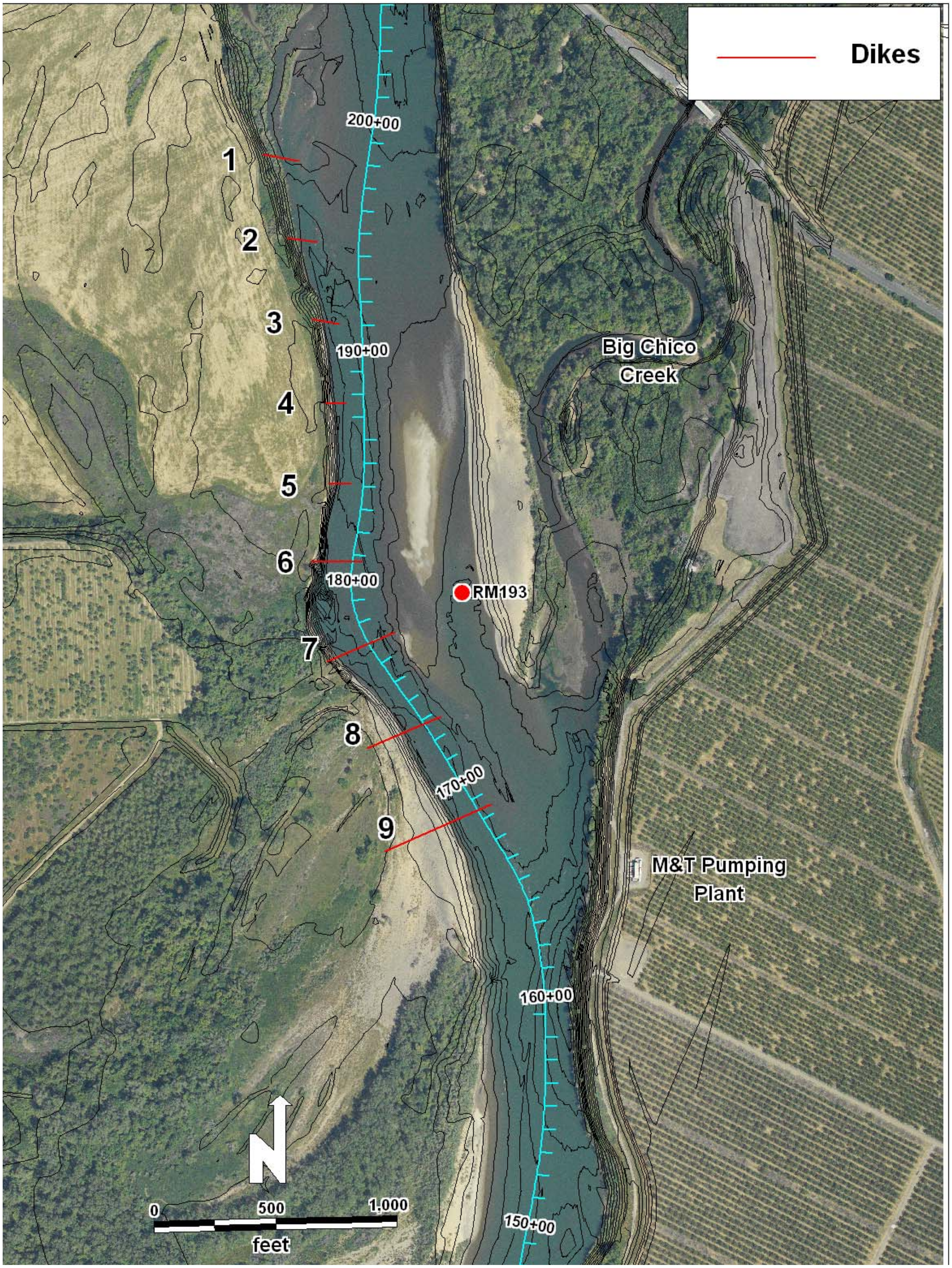

Figure 3.2. Aerial photograph of the study reach showing the dike configurations for Scenario 4.

topography of the modeled reach. Topographic data for the baseline conditions model were derived from the 1996 mapping by Ayres Associates, and updated with bathymetric and topographic data from the December 2005 survey that was conducted by MEI. The update included replacing the in-channel topography with the 2005 data and re-contouring the banks and near-river overbanks to reflect the 2005 channel alignment.

The MEI survey was conducted between December 8 and 12, 2005, when the average discharge was 6,280 cfs (**Figure 3.3**), and it extended from about RM 185.5 near the mouth of Stony Creek to RM 195 (**Figure 3.4**). Control for the survey was based on nearby National Geodetic Survey (NGS) control points because none of the control points that were set by Ayres Associates for the 1996 mapping in the vicinity of the project reach could be located due to repaving of the roads. The hydrographic survey was conducted using a survey grade fathometer (Odom Hydrotrac Echosounder with +/-0.2-foot resolution) and a Leica 1200 series RTK survey grade GPS. Cross sections were run at approximately 150-foot spacing within the limits of the 2-D model mesh, increasing to about 300-foot spacing beyond the 2-D model limits. Additional longitudinal profiles were surveyed to increase survey resolution along the eroding right bank and in the vicinity of the M&T pump. Additional overbank topography was obtained using an RTK roving unit to define the topography of the eroded banks (including the right bank opposite Big Chico Creek) and bars that have changed geometry since the 1996 mapping.

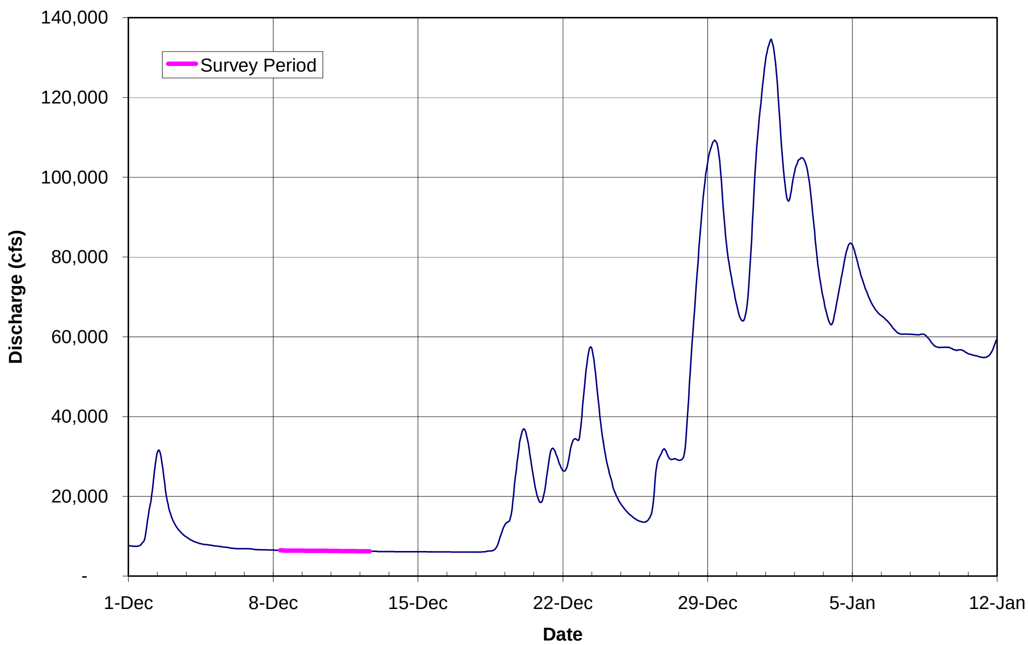

Figure 3.3. Recorded flow in the Sacramento River at the Hamilton City gage between December 1, 2005 and January 11, 2006.

On January 1, 2006, three weeks after the MEI survey, a peak discharge of 134,600 cfs was recorded at the Hamilton City gage (Figure 3.2). The California Department of Water Resources (DWR) re-surveyed the eroding west bank and collected sediment samples in February 2006. Comparison of the DWR and MEI survey along the top of the right bank indicates the bank retreated by an additional approximately 5 feet during the January 2006 flood (**Figure 3.5**). This change is relatively minor in relation to the overall geometry and alignment of the river and the associated effect on hydraulic conditions in the reach is, therefore, insignificant for purpose of the analyses performed for this study. As a result, no adjustment was made to the updated

Figure 3.4. Aerial photograph showing the traces from MEI's December 2005 survey for the channel cross sections and supplementary bathymetry that were used to update the mapping.

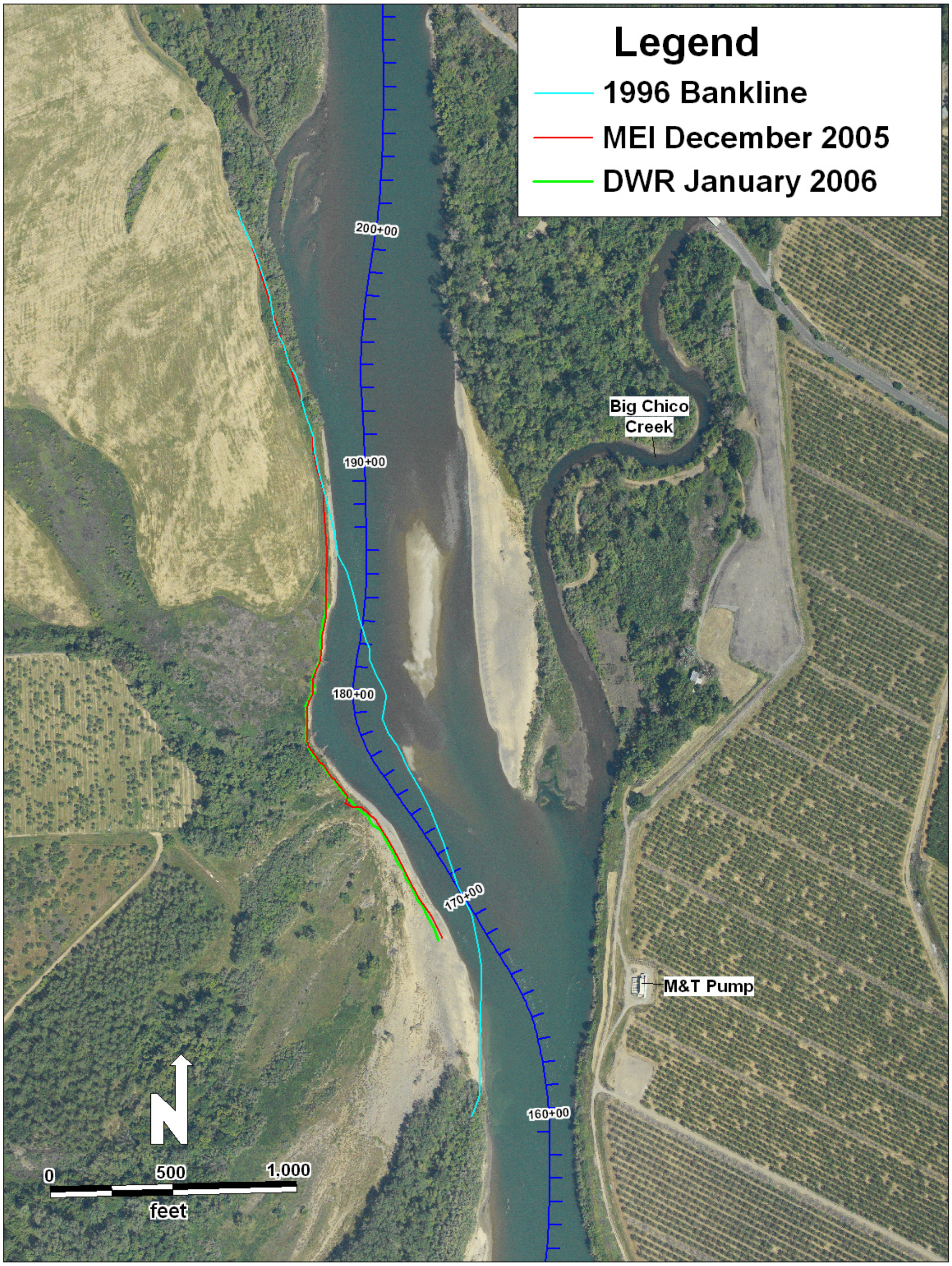

Figure 3.5. Comparison of the right bank alignments in 1996, December 2005 and February 2006. The station-line that was developed to facilitate model development and analysis is also shown.

topography and bathymetry to reflect the changes that have occurred since the 2005 data were collected.

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 confluence of the Sacramento River and Stony Creek (RM189.9). Along this station line, the up- and downstream ends of the 2-D model mesh are located at Sta 245+88 (RM 194.35) and Sta 149+95 (RM192.5), respectively (**Figure 3.6**). The M&T pumping station is located at Sta 164+26 on the left (east) bank of the river.

The topographic data were used to create an initial finite element mesh that consists predominately of quadrilateral elements that are generally perpendicular to the flow direction (Figure 3.5). The modeled reach is approximately 10,000 feet long, with the M&T pump intake located approximately 1,400 feet upstream from the downstream model boundary. The finite element mesh was established to accurately represent the planform geometry and topography of the study reach, with greater mesh density in topographically complex areas where hydrodynamic variability is high (e.g., in the vicinity of the proposed dikes). The resulting mesh that corresponds to the effective flow area at the approximate bankfull discharge of 90,000 cfs contains approximately 7,800 elements and 23,000 nodes.

3.1.2. Downstream Boundary Conditions

The downstream boundary conditions for the 2-D model consist of a specified water-surface elevation for the particular discharge that is being modeled. A rating curve for the downstream end of the model (**Figure 3.7**) was developed using a 1-D HEC-RAS model of the longer surveyed reach that ties into the existing 1-D model at the downstream end that was used in the previous analyses. The 1-D model results indicate that the channel capacity in the vicinity of the M&T intake is approximately 90,000 cfs, and the bar at the mouth of Big Chico Creek becomes submerged at flows greater than 30,000 to 35,000 cfs.

3.1.3. Material Properties and Model Validation

The RMA2 model uses Manning's *n*-values to define boundary friction losses and turbulence exchange coefficients (kinematic eddy viscosity values) to describe energy loss due to internal turbulence. These two parameters are specified as material types for each element in the mesh. Five different material types were used in the models to represent the main channel, side channels, forested areas, banks and dikes (**Figure 3.8**). Main-channel Manning's *n*-values ranging from 0.033 to 0.035 were used for the calibration discharge of 6,280 cfs based on field observations, similar experience with other rivers, and standard references (Chow, 1959; Barnes, 1967; Hicks and Mason, 1991; Julien, 1995).

Manning's *n*-values of 0.12 were used for the overbanks to reflect the roughness of the vegetation in these areas, based on aerial photographs and field observations. An *n*-value of 0.12 was also applied to the dikes to reflect the roughness associated with the riprap. The side channels and banks were assigned *n*-values of 0.05 and 0.06, respectively, to reflect the smaller flow depths (and thus, higher relative roughness) and vegetation that is present in these areas. A constant eddy viscosity of 20 lb/ft^2 was used in the modeling, and the vorticity option was applied to improve correlation between the modeled and observed flow patterns caused by the channel curvature. The agreement between the computed and measured water-surface elevations during the December 2005 survey when the discharge was 6,280 cfs using these parameters is very good (**Figure 3.9**).

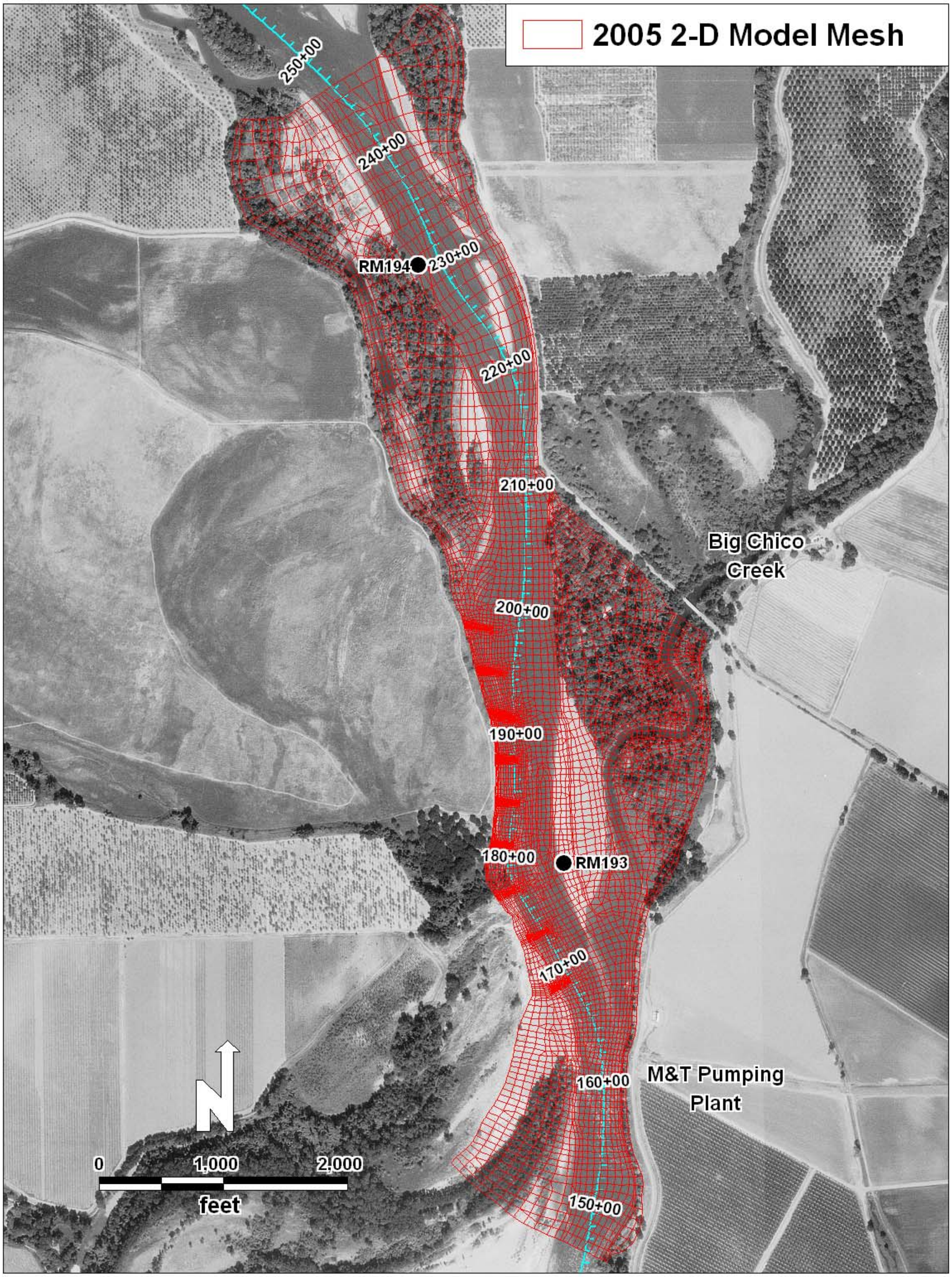

Figure 3.6. Finite element mesh for the 2005 geometry of the M&T pumping plant reach.

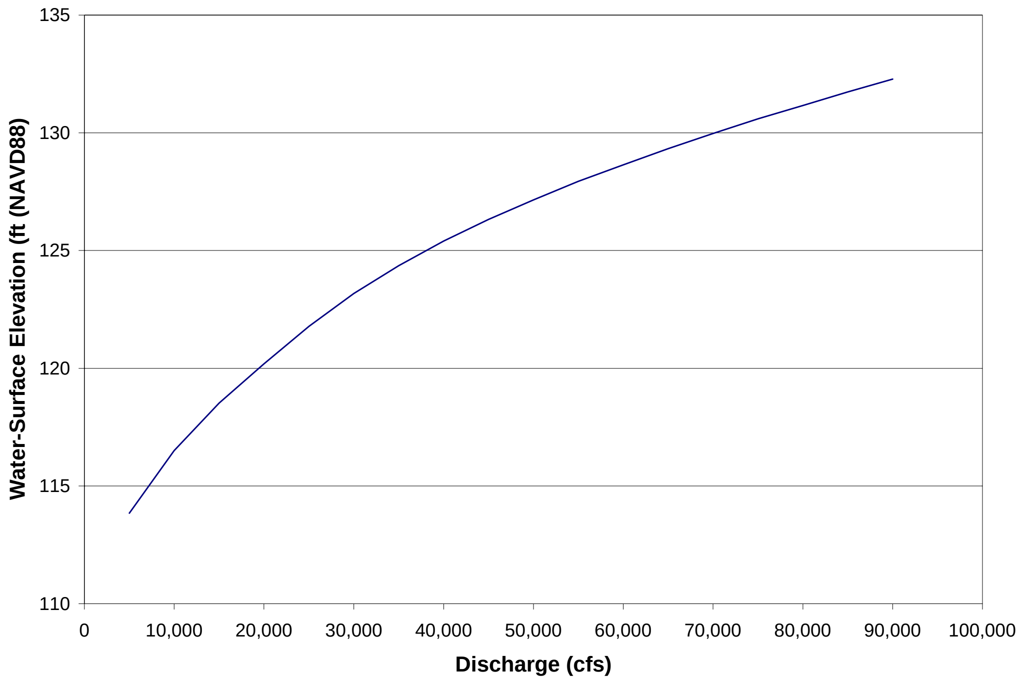

Figure 3.7. Stage-discharge rating curve at the downstream boundary of the 2-D model, based on results from the updated HEC-RAS 1-D model.

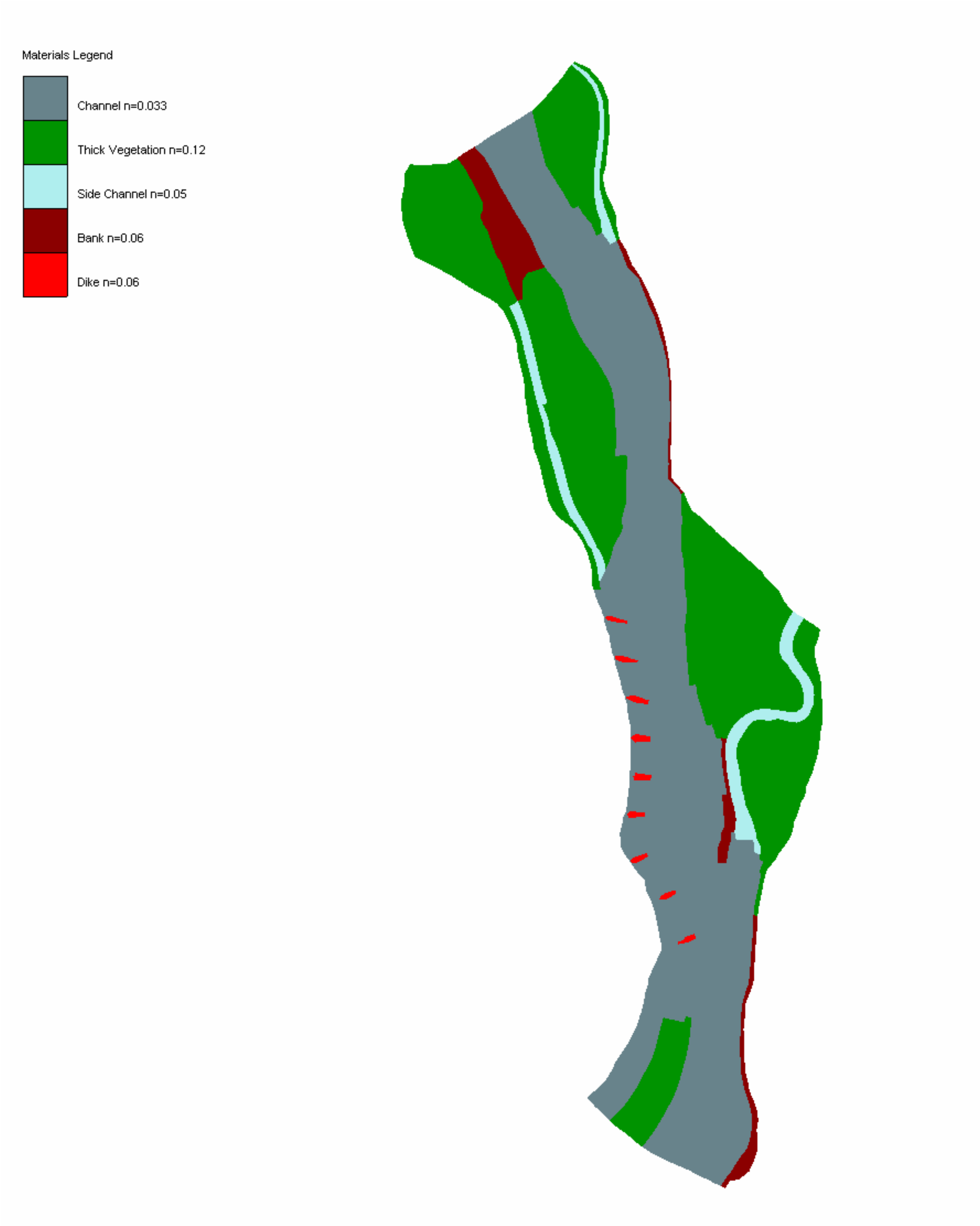

Figure 3.8. Distribution of the material types used to define roughness and turbulence exchange coefficient in the modeled reach.

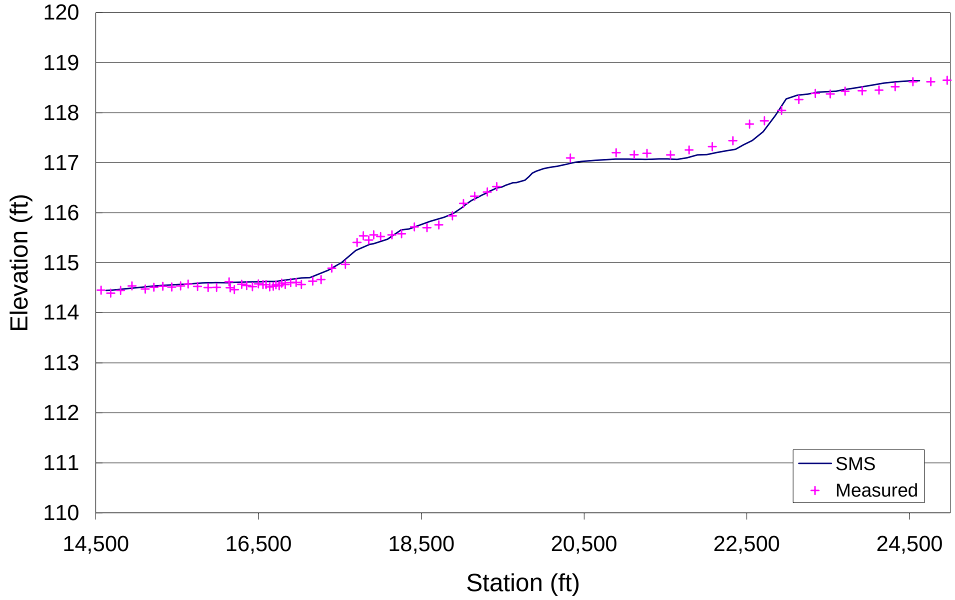

Figure 3.9. Comparison of the predicted and measured water-surface profiles from the December 2005 survey when the discharge was 6,280 cfs.

3.1.4. Effects of Big Chico Creek

A sensitivity analysis was conducted by MEI (2005) to evaluate the potential effects of inflows from Big Chico Creek on hydraulic conditions in the mainstem Sacramento River. Analysis of the concurrent flows indicated that, although there is generally poor correlation between flows, the most likely discharges in Big Chico Creek when the river is near bankfull conditions are in the range of 1,000 to 1,500 cfs. The previous 2-D model was run at a discharge 90,000 cfs with and without a 1,500-cfs inflow from Big Chico Creek. Comparison of the results indicated that Big Chico Creek flows have very little effect on the water-surface profile, flow depths, velocities and sediment-transport characteristics in the vicinity of the M&T intake. Based on these results, Big Chico Creek inflows were not considered in the modeling for this study.

3.1.5. Baseline (2005) Conditions Model Results

To evaluate the hydraulic and sediment-transport conditions in the reach, the baseline conditions model (as well as the dike configuration models) was run for a series of discharges ranging from 5,000 cfs to the approximate bankfull discharge of 90,000 cfs (**Table 3.1**). (The calibration discharge of 6,280 cfs shown in Table 3.1 was only run with the baseline conditions model.)

The three lowest discharges (5,000, 10,000 and 15,000 cfs) were used to evaluate conditions in the river during the period when the pumps are typically in operation. As described in a later section of this report, the top of the dikes for the design scenarios were established using the baseline conditions water-surface profile at 35,000 cfs, which corresponds to about two-thirds of the bank height. Model runs at 23,140 cfs (10-percent exceedence value on the mean-daily flow-duration curve) provide hydraulic conditions for an intermediate discharge between 15,000 cfs and the discharge at the top of the dikes. The two highest discharges were included because some low elevation banks begin to overtop at about 75,000 cfs and, water-surface elevation is near the top of the bank along most of the reach at 90,000 cfs. Discharges above 90,000 cfs were not modeled because nearly all of the additional flow at higher discharges is conveyed in the overbanks, resulting in relatively insignificant changes to the water-surface elevation and hydraulic conditions from the bankfull condition.

Table 3.1. Discharges modeled in the 2-D hydraulic analysis.

Run Number	Discharge (cfs)	Return Period (years)	Percent Exceedence	Comment
1	5,000		95.9	Low flow value
2	6,280		87.5	Baseline calibration discharge
3	10,000		47.8	Low flow value
4	15,000		22.7	Low flow value
5	23,140		11.4	10% exceedence
6	35,000		6.7	Discharge to set top of dike
7	75,000	1.36	1.5	Some overtopping
8	90,000	1.75	0.9	Bankfull

Results from the baseline conditions model indicate that the maximum main channel velocities range from 6 to 10 fps and maximum channel depths range from 17 to 40 feet along the reach at 90,000 cfs (**Figures 3.10 and 3.11**). The highest velocities occur in the riffle area at the upstream end of the reach, and the lowest velocities typically occur in the expansion zone near the downstream end of the gravel bar, which creates the conditions for sediment deposition and further bar development. The maximum flow depth occurs near the M&T pump intake. The velocity over the gravel bar at 90,000 cfs is approximately 4 to 5 fps and the flow depth is approximately 8 feet. The main flow channel upstream from the bar is directed slightly towards the right bank (west bank), but most of the flow is concentrated toward the center of the channel. A flow expansion area occurs at the head of the bar, and the majority of flow is orientated mostly in line with the bar; however, some shoaling occurs towards the left bank over the bar. The velocities in the deep area adjacent to the M&T pump intake are approximately 7 fps at 90,000 cfs. Velocities in the vicinity of the pump at the three lowest modeled discharges (5,000, 10,000 and 15,000 cfs) are approximately 1.2, 1.7 and 1.8 fps, respectively.

3.1.6. Incipient Motion Analysis

An incipient motion analysis was performed to evaluate the mobility of the bed material at the study site under the different modeled discharges and geometries. The analysis was performed by comparing the critical shear stress (τ_C, shear stress required to initiate motion) for the median particle size with the bed shear stress (τ) over the range of flows.

The critical shear stress for each discharge was estimated using the Shields (1936) relation, given by:

$$\tau_c = \tau_{*c}(\gamma_{S-}\gamma)D_{50} \tag{3.1}$$

where τ_c = critical shear stress,

τ_{*c} = dimensionless shear stress,

Υ_S = unit weight of sediment (~165 lb/ft^3),

Υ = unit weight of water (62.4 lb/ft^3), and

D_{50} = median particle size.

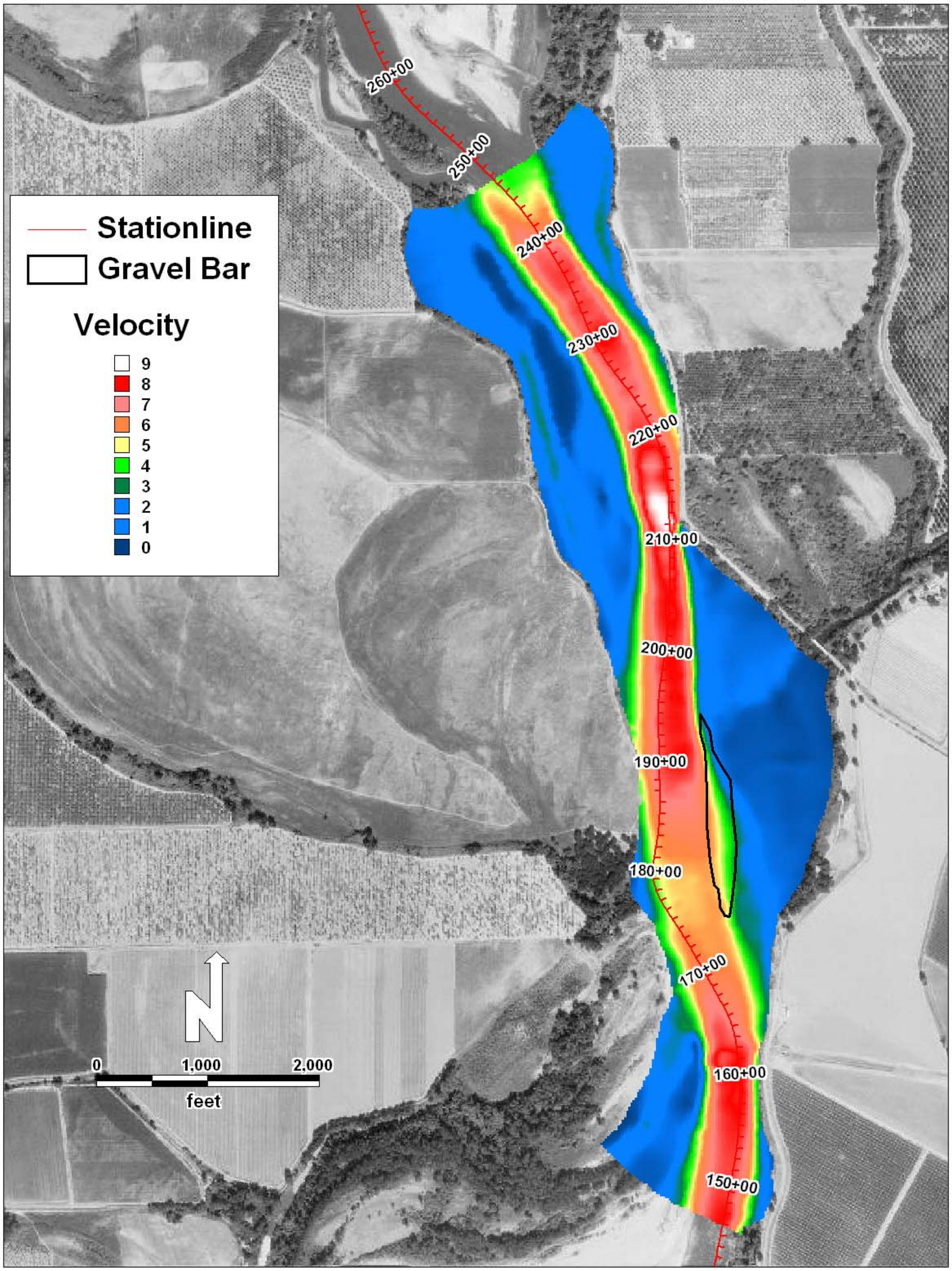

Figure 3.10. Velocity distribution predicted by the baseline conditions model at a discharge of 90,000 cfs.

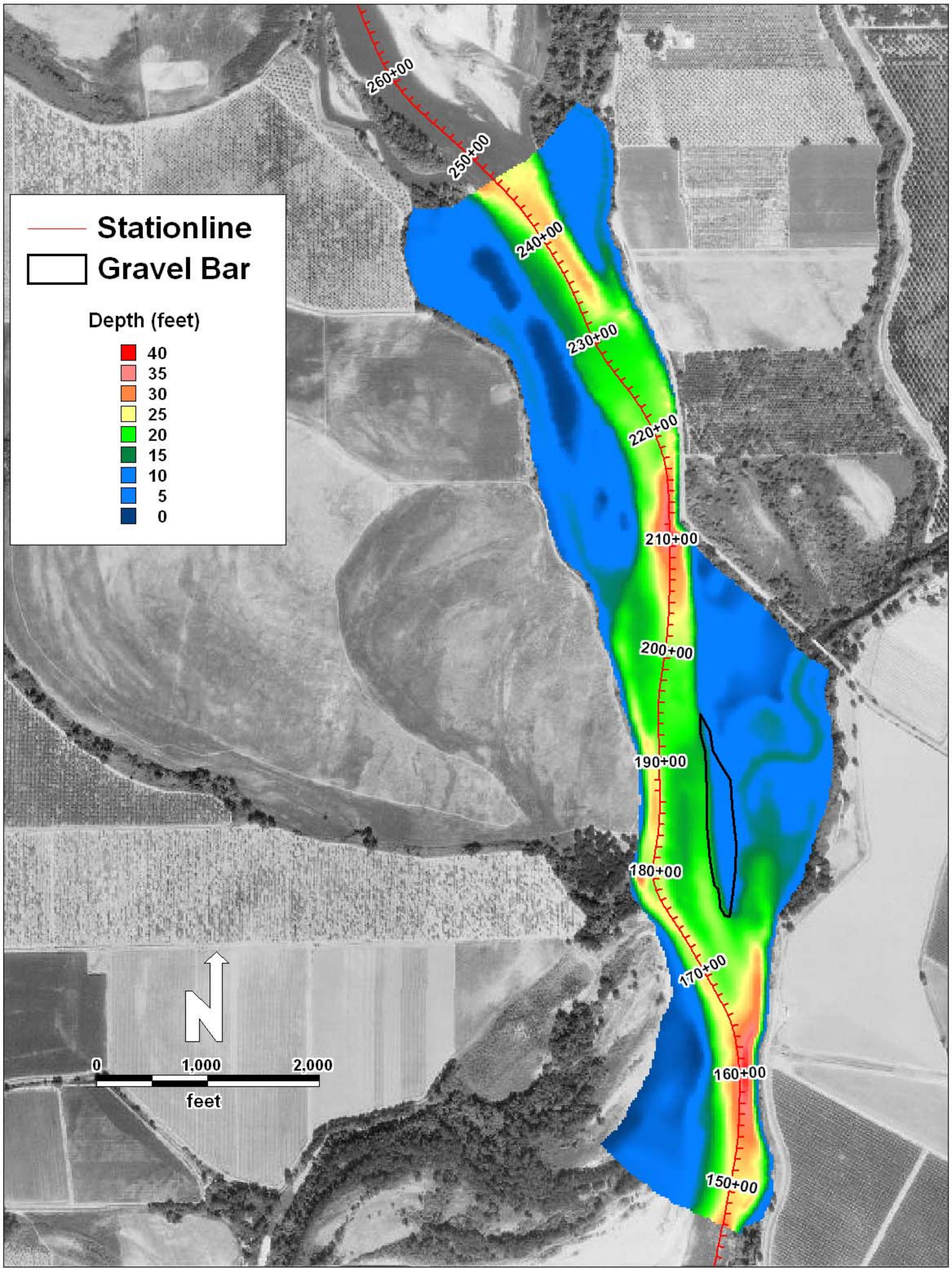

Figure 3.11. Distribution of flow depths predicted by the baseline conditions model at a discharge of 90,000 cfs.

When the critical shear stress for the median particle size is exceeded, the bed is mobilized and all sizes up to about five times the median size are capable of being transported by the flow (Parker et al., 1982; Andrews, 1984). Reported values of τ_{*c} for the median particle size of the surface bed material range from 0.03 (Meyer-Peter and Müller, 1948; Neill, 1968) to 0.06 (Shields, 1936). A value of 0.047 is commonly used in engineering practice based on the Meyer-Peter, Müller bed-load transport equation (Meyer-Peter and Müller, 1948). Detailed evaluation of Meyer-Peter and Müller's data and more recent studies (Parker et al., 1982; Andrews, 1984) indicate that a value of 0.03 is more reasonable for defining incipient motion in gravel- and cobble-bed streams. In fact, Neill (1968) concluded that a dimensionless shear value of 0.03 corresponds to true incipient motion of the bed-material matrix while 0.047 corresponds to a low but measurable sediment-transport rate. Accordingly, a value of 0.03 was used in this analysis.

The bed shear stress due to grain resistance (τ') is normally used in the incipient motion analysis because it is a better descriptor of near-bed hydraulic forces in gravel-bed streams that are responsible for particle motion than the more commonly used total shear stress, because it eliminates the effects of flow resistance due to irregularities in the channel boundary, non-linearity of the channel, variations in channel width, and other factors. The grain shear stress (τ') is computed from the following equation:

$$\tau' = \gamma Y' S \tag{3.2}$$

where: γ = the unit weight of water (62.4 lb/ft^3),
Y' = the portion of the total hydraulic depth associates with grain resistance, (Einstein, 1950), and
S = energy slope.

The local energy slope (S) for each grid location is obtained by rearranging the Manning's equation, as follows:

$$S = \left[\frac{nV}{1.49y^{\frac{2}{3}}}\right]^{\frac{1}{2}} \tag{3.3}$$

The depth due to grain resistance (Y') is then computed by iteratively solving the semilogarithmic velocity profile equation:

$$\frac{V}{V_*'} = 6.25 + 5.75 \log\left(\frac{Y'}{k_s}\right) \tag{3.4}$$

where V = velocity at the node,
k_s = characteristic roughness of the bed, and
V_*' = shear velocity due to grain roughness, given by:

$$V_*' = \sqrt{gY'S} \tag{3.5}$$

The characteristic roughness height of the bed (k_S) is approximately 3.5 D_{84} (Hey, 1979).

For the purposes of evaluating incipient conditions, it is convenient to define the normalized grain shear stress (τ_*), which is the ratio of grain shear stress (τ') to the critical shear stress (τ_C) or:

$$\tau_* = \frac{\tau'}{\tau_C} \tag{3.6}$$

If $\tau_*<1$, the shear stress is insufficient to mobilize the bed material; if $\tau_*>1$, bed mobilization is indicated. Dimensionless grain shear values in the range of 1.3 to 1.5 are necessary for measurable transport rates that would be sufficient to cause significant adjustment of the bed topography over reasonable time-frames.

A representative surface bed-material gradation curve was developed for the study reach based on the average of three measurements that were made during the December 2005 survey using the pebble count method (Wolman, 1954) (**Figures 3.12**). Pebble counts WC1 and WC2 were made on top of the primary gravel bar that is the subject of this analysis, and WC3 was made on the lower elevation mid-channel bar adjacent to the primary bar (**Figure 3.13**). The representative gradation curve has median (D_{50}) and D_{84} sizes of 39 and 60 mm, respectively. A bulk sample of the subsurface material was also collected at the same location as WC1 after removing the coarser surface layer. The material in this sample is representative of the size-gradation of the sediment that was being transported at the time the bar was being formed, and it had a median size (D_{50}) of 9.5 mm and a D_{84} of 32.9 mm (Figure 3.11).

The normalized grain shear stress (NGS) was estimated at each node in the finite element model for each of the modeled discharges using a dimensionless critical shear stress (i.e., Shields parameter) of 0.03 and the representative median (D_{50}) size at the site of 39.0 mm. For the hydraulics associated with the 90,000 cfs model run, the normalized grain shear stresses in the main channel range from about 0.7 between the pump intake and the downstream portion of the bar near Sta 180+00 to about 1.4 near the upstream end of the bar (**Figure 3.14**). This indicates that the river is capable of transporting gravel- and cobble-sized material through the reach along the upstream half of the bar, but the area between about the downstream half of the bar and the pump intake is depositional (Figure 3.14). At this discharge, there is insufficient shear stress to mobilize the material on the gravel bar. The results also indicate that the NGS increases back to the 1.2 to 1.5 range downstream from the intake. At discharges of 75,000 cfs and below, the NGS values are generally less than 1.3, indicating that significant sediment transport does not occur at the lower modeled flows.

3.1.7. Sediment-transport Calculations

The variation in sediment-transport capacity along the reach was quantified by estimating the transport capacity at 28 cross sections that were developed from the 2-D mesh using the continuity line option in SMS (Figure 3.13). The sediment-transport capacity across each element of the continuity line was computed using the Parker (1990) surface-based bed load equation with the corresponding hydraulic conditions at the nodes and discharges across each element of the cross section. The results indicate that the transport capacity in the vicinity of Cross Sections 7 through 15 (Sta 168+20 to Sta 182+60), upstream from the pump intake to midway along the gravel bar, is very low compared to upstream reaches (**Figure 3.15**). This condition strongly favors the continued development and downstream migration of the bar.

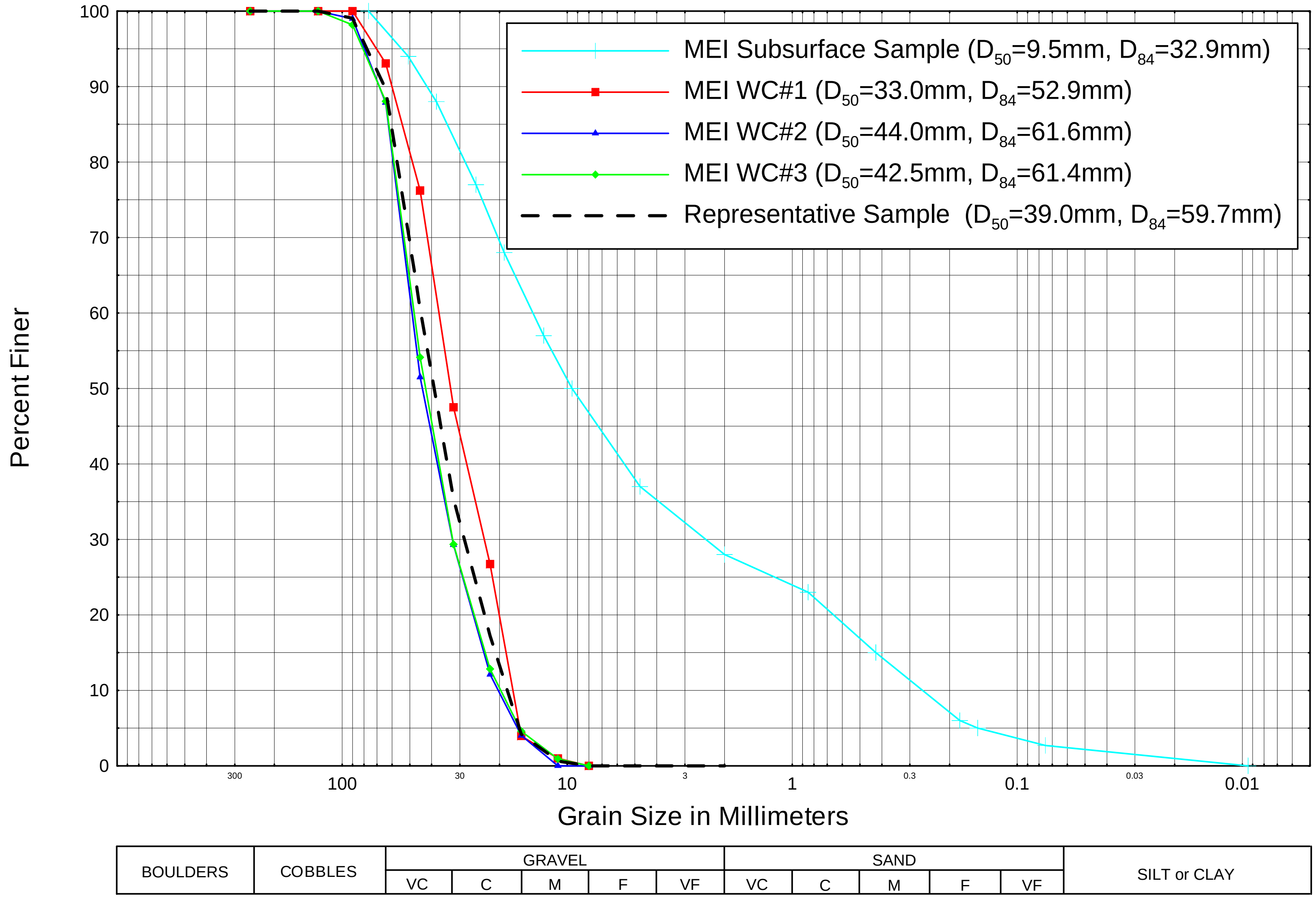

Figure 3.12. Bed-material gradation curves for samples collected by MEI in conjunction with the December 2005 surveys. Also shown is the representative surface-gradation curve that was used in the incipient motion and in the sediment-transport analysis.

Figure 3.13. Location of the surface sediment samples that were collected by MEI using the pebble count method (Wolman, 1954) in conjunction with the December 2005 survey. The subsurface sample shown in Figure 3.11 was taken at location WC1.

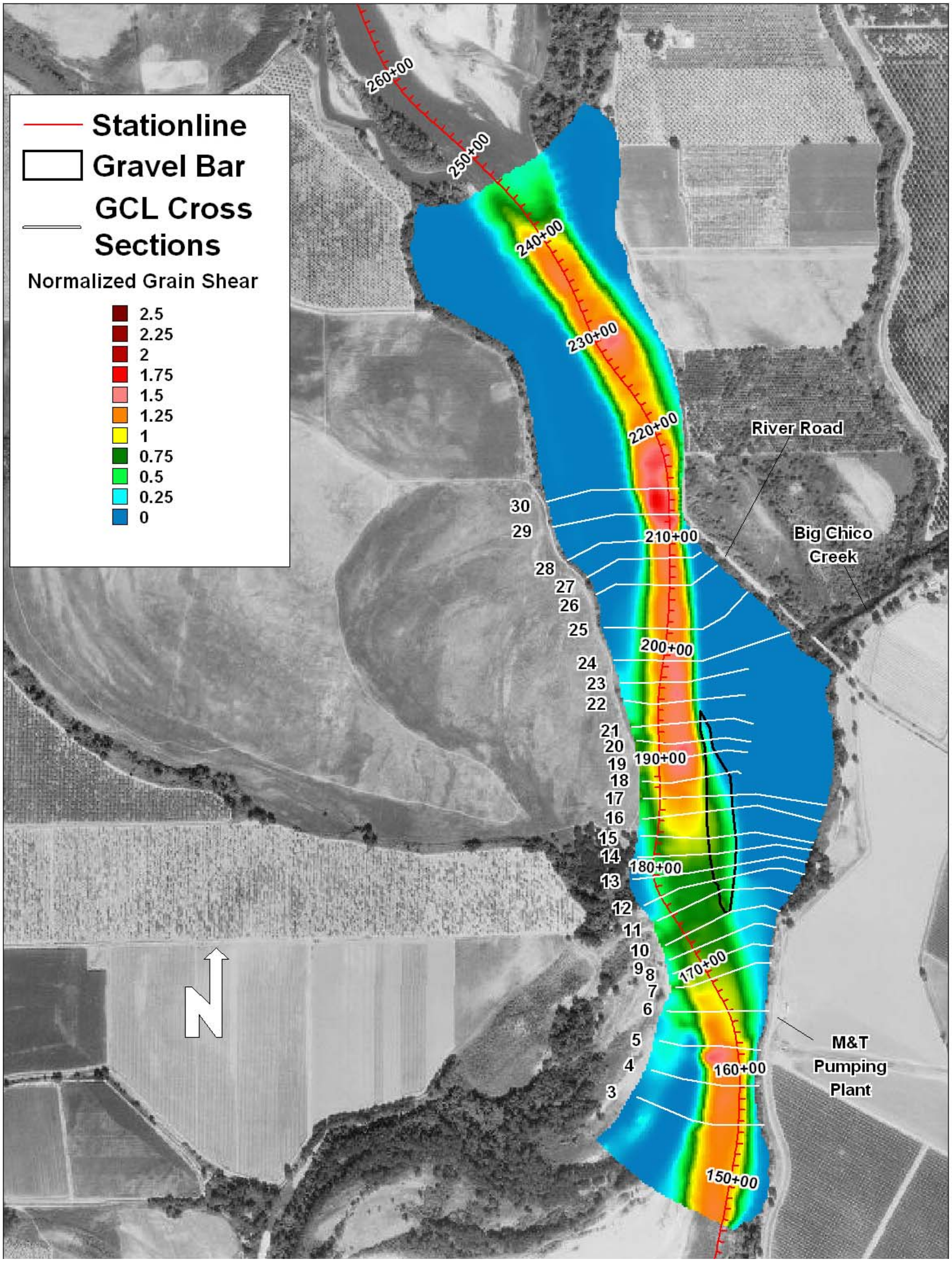

Figure 3.14. Distribution of normalized grain shear predicted by the baseline conditions model at 90,000 cfs. Also shown are the cross-section lines used for the sediment-transport calculations.

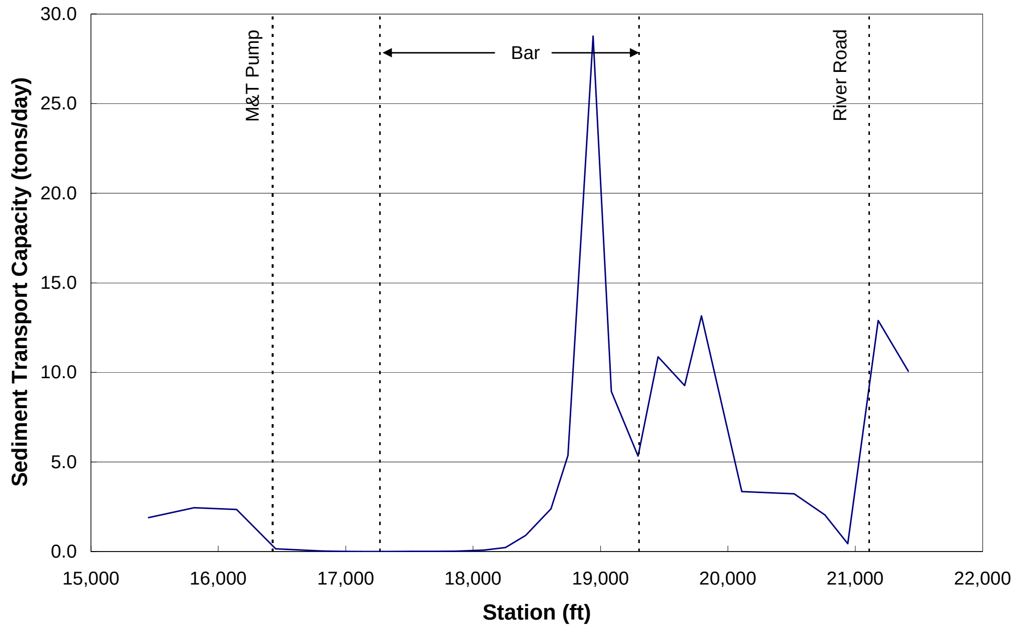

Figure 3.15. Predicted sediment-transport capacities at the 28 cross sections identified in Figure 3.13 for the baseline conditions.

3.2. Design Conditions Models

3.2.1. 8-Dike Configuration (Scenario 2)

The effects of the dike field that was proposed in the original Steering Committee report (Harvey et al., 2004) were re-evaluated using the updated model with the 2005 topography, modified to reflect the presence of the dikes (Figure 3.1). The proposed dike field consists of eight 150- to 200-foot long spur dikes that are spaced at about 360-foot intervals from near the nose of the primary gravel bar to about 500 feet upstream from the head of the bar. The top of the dikes was set at the water-surface elevation associated with a flow of 35,000 cfs, which corresponds to approximately two-thirds of the existing bank height (**Figure 3.16**). At this height, the dikes will be overtopped about 7 percent of the time (about 24 days per year), on average, based on the mean daily flow-duration curve (Figure 2.2). The longitudinal profile along each dike consists of the 2H:1V sloping face from the nose to the intersection with the river bed, and a crest profile that slopes upward at 5 percent from the nose to the intersection with the existing bankline (**Figure 3.17**). In cross section, the dikes have a topwidth of 5 feet, with 2H:1V sideslopes.

Model results at 90,000 cfs for this scenario indicate that the shear stress along the gravel bar will increase significantly compared to baseline conditions due primarily to narrowing of the channel (**Figures 3.18 and 3.19**). Under baseline conditions (Figure 3.12), the NGS along the right (west) edge of the gravel bar is generally between 0.7 and 0.9, and the upper surface of the bar is at or below 0.8. This indicates that the material on the bar is not mobile under these conditions. With the proposed dike field, the NGS along the right edge and upper portion of the bar increases to 1.1 to 1.3, indicating that surface material on the bar would be mobilized. The NGS in the main channel along the bar is also somewhat higher, ranging from about 1.0 near the downstream end to about 1.3 near the head of the bar. This configuration tends to shift the

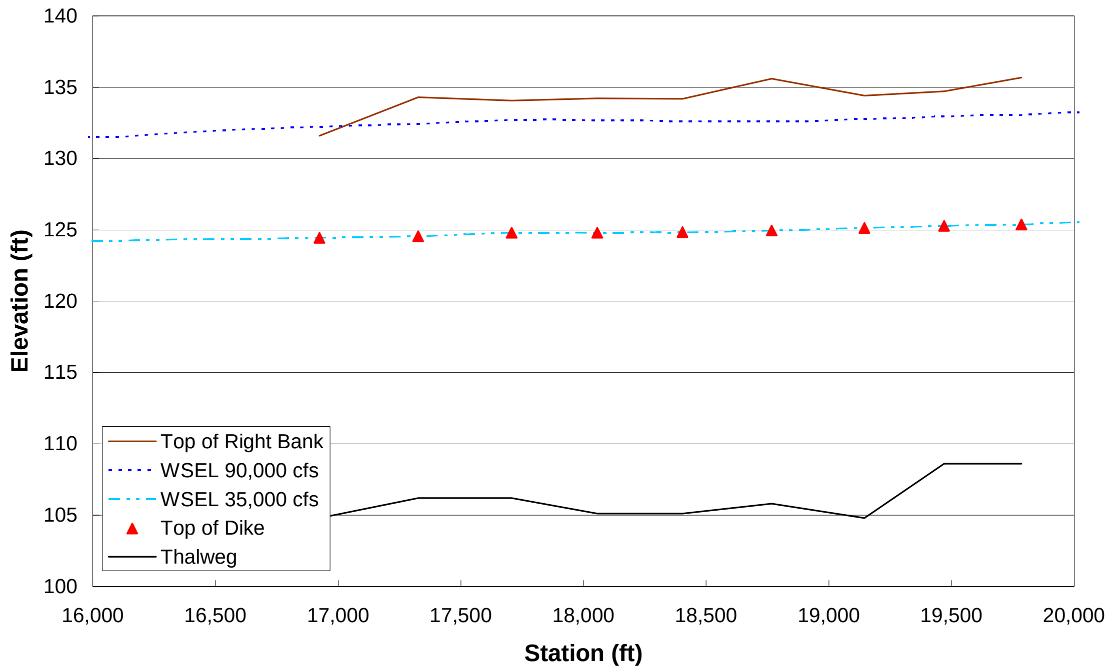

Figure 3.16. Predicted water-surface profiles at 35,000 and 90,000 cfs. Also shown are the tops of the dikes in relation to the water-surface profiles.

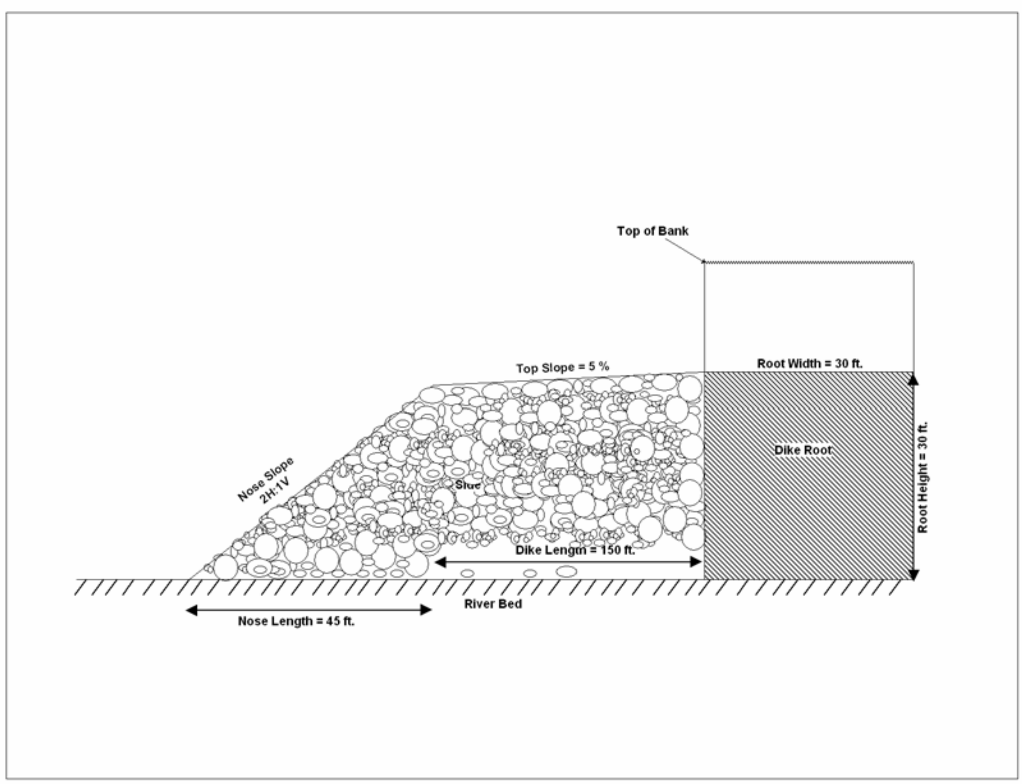

Figure 3.17. Schematic profile of the proposed spur dikes.

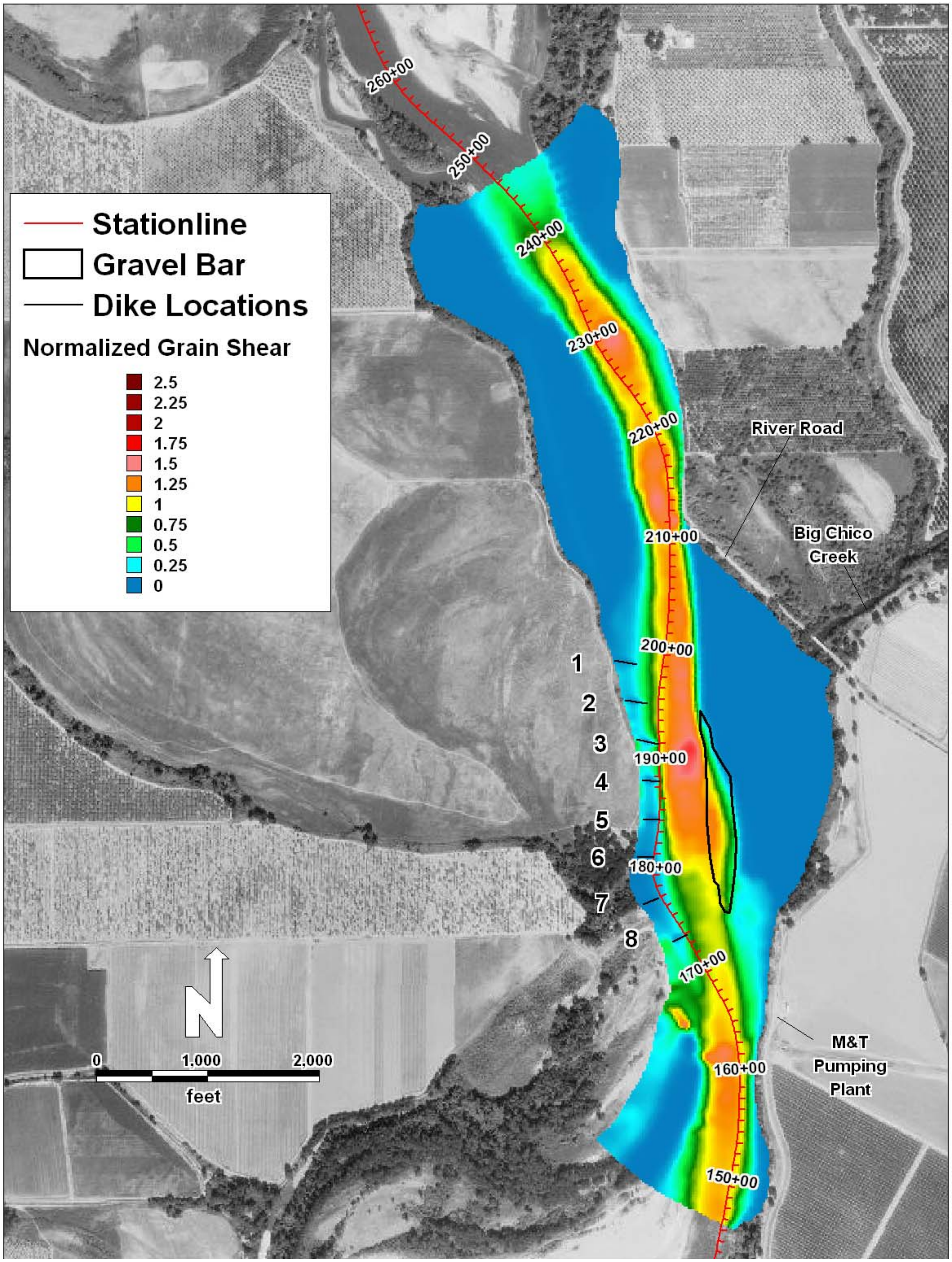

Figure 3.18. Normalized grain shear stress distribution at 90,000 cfs for the 8-dike configuration (Scenario 2).

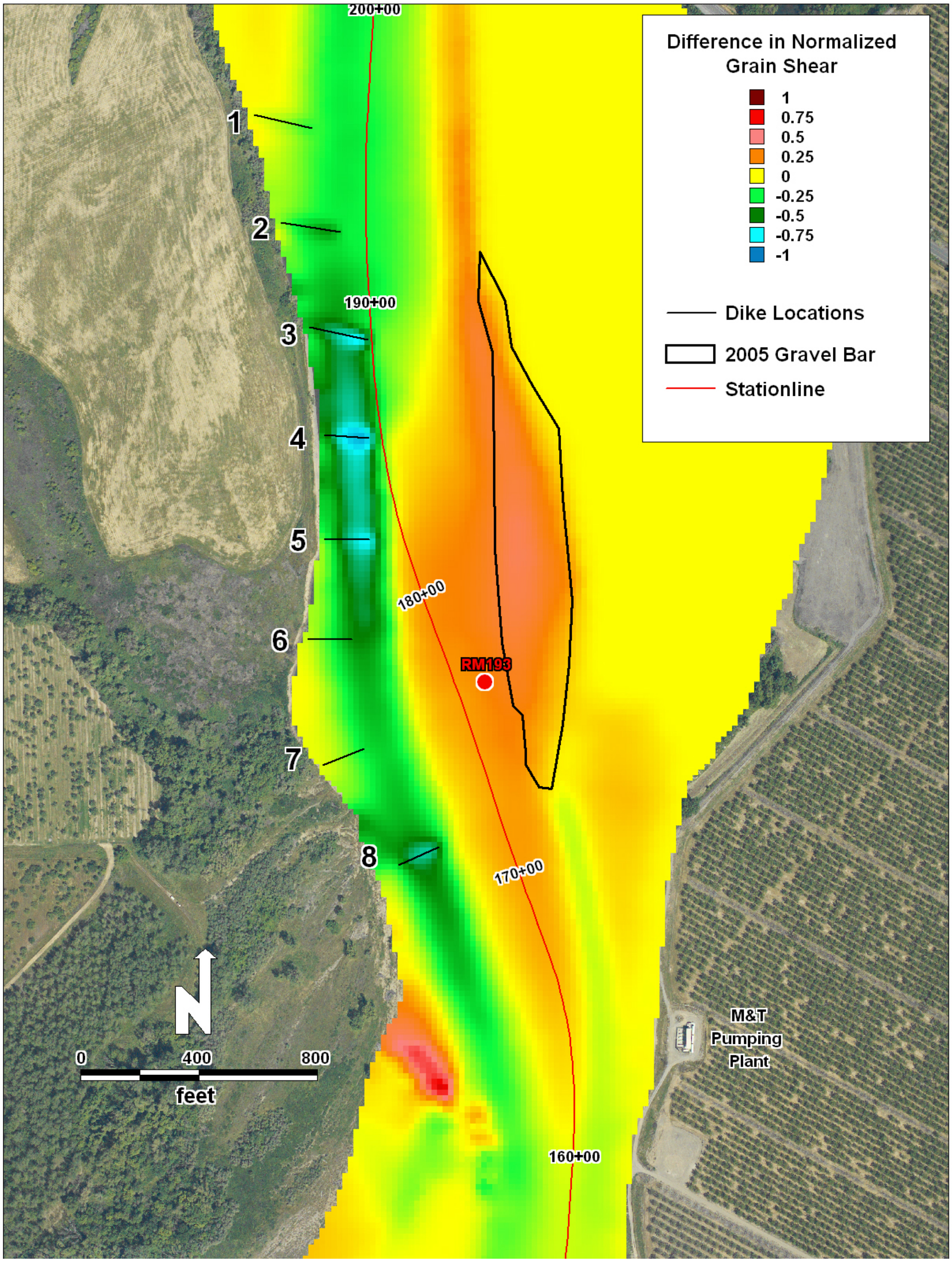

Figure 3.19. Difference in normalized grain shear stress between baseline conditions and the 8-dike configuration (Scenario 2) at 90,000 cfs.

location of highest shear toward the left side of the channel compared to baseline conditions, but there is little or no change in shear in the vicinity of the intake (**Figure 3.20**).

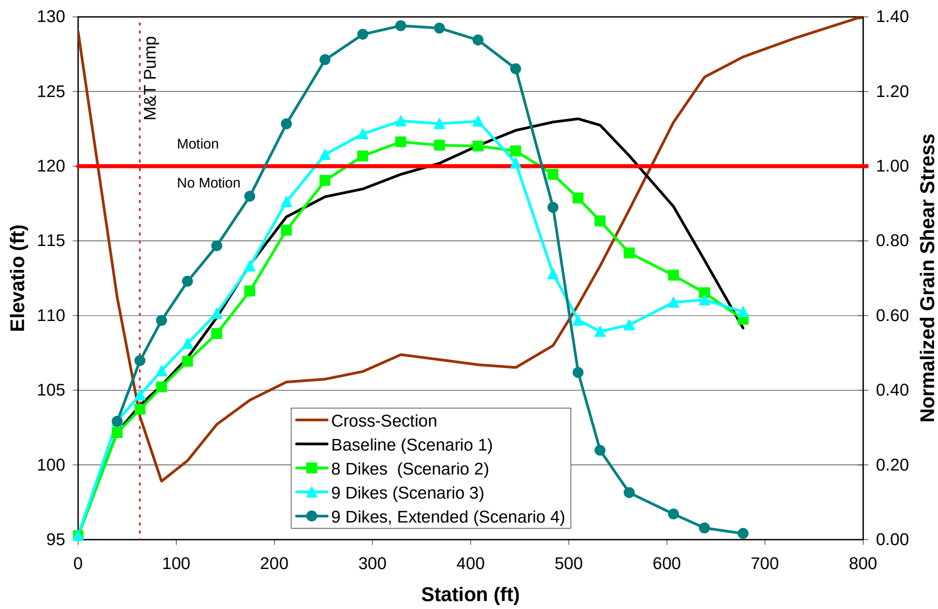

Figure 3.20. Normalized grain shear (NGS) distribution at M&T pump (Cross Section 6 identified in Figure 3.13).

The estimated transport capacity for the gravel- and cobble-sized bed material is considerably higher along the bar with the dikes in place than under baseline conditions (**Figure 3.21**). In the reach between the downstream end of the bar and the pump intake, however, there is little or no transport, similar to baseline conditions. Downstream of the intake the sediment transport capacity increases by a modest amount due to the change in flow alignment associated with the upstream dikes.

Continued downstream migration of the gravel bar will occur primarily through changes in channel geometry associated with continued erosion of the west bank adjacent to and downstream from the bar. The proposed dike field will prevent further bank erosion at this location; thus, it should also be effective in preventing further downstream growth of the bar. The higher mobility of the gravels along the right edge of the bar compared to baseline conditions also indicates that this portion of the bar would likely erode under this configuration. Based on the low bed material-transport capacities in the reach downstream from the bar relative to the upstream reaches, however, the channel will likely remain depositional in the vicinity of the intake, and deposition problems may continue to occur, particularly under low to moderate flow conditions.

Under low to intermediate flow conditions, the bed shear stress at the pump intake with the 8-dike configuration will increase by 30 to 50 percent over baseline conditions, which should increase the capacity to transport sand-sized material, thereby decreasing the potential for sand deposition at this location. An analysis was conducted to further quantify the potential for reducing the tendency for deposition of sand-sized material in the vicinity of the intake by identifying the preferred path of a typical particle that would pass over the intake and then computing the bed shear stress and relative transport capacity along that path. The bed shear stress was computed from the 2-D model output, and the relative transport capacity was

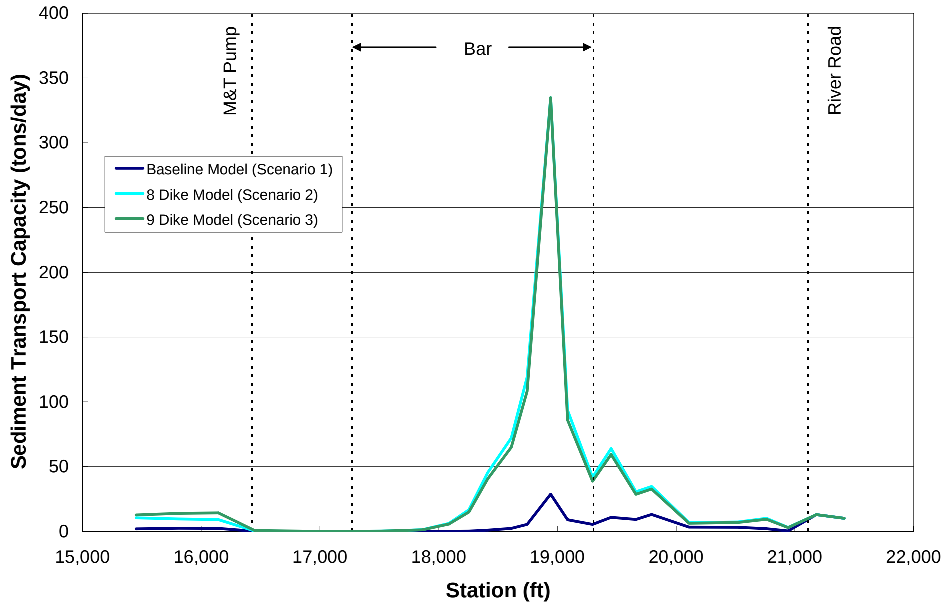

Figure 3.21. Predicted sediment-transport capacities at the 28 cross sections identified in Figure 3.13 for the baseline conditions, 8- and 9-dike scenarios.

estimated based on the difference between the bed shear stress and the critical shear stress raised to the 1.5 power, consistent with bed-load transport equations that are often applied for sand-bed conditions (e.g., Meyer-Peter, Müller). A typical example of the particle paths indicated by the 2-D model for baseline conditions and the project conditions scenarios at a discharge of 10,000 cfs, which is slightly less than the median flow in the study reach, is shown in **Figure 3.22**. Under baseline conditions, the transport capacity for 1-mm sand along this path for a distance of about 1,000 feet up- and downstream from the intake is very low (**Figure 3.23**). In fact, the bed shear is below incipient conditions over nearly all of this portion of the path. For the 8-dike configuration, the transport capacity increases substantially in the area from about 500 to 1,000 feet upstream from the intake, but there is little or no change in the vicinity of, and downstream from, the intake.

Reanalysis of the hydraulic and sediment mobility characteristics of the M&T reach has produced somewhat different results from those presented in the previous analysis (MEI, 2005). There are two primary reasons for the differences. First, the previous analysis of 2003 baseline conditions was conducted with 1996 Ayres Associates topography that was modified, as far as possible, to account for the 2000 bar dredging and retreat of the west bank between 1996 and 2003 (>200 feet). Additionally, bank erosion and retreat have continued since 2003, further altering the reach hydraulics. Second, and probably more significant, the sediment size information that was used in the 2003 conditions analysis was derived from a pebble count conducted in 1990 at the head of the Bidwell State Park bar (WET, 1990) that had a median (D_{50} and D_{84}) sizes of 21.5 and 39 mm, respectively (MEI, 2005). In contrast, the three pebble counts conducted on the bar upstream of the M&T pumps by MEI in December 2005, had an average D_{50} and D_{84} of 38 and 60 mm, respectively.

In combination, these two factors have a significant impact on the mobility of the bed material as expressed in terms of the normalized grain shear (NGS). Comparison of Figure 3.12 from MEI (2005) with Figure 3.14 in this report shows the extent of the differences (note the differences in the color scales on the figures). In general terms, the 2005 figure shows much higher NGS

Figure 3.22. Predicted particle path of sediment over the M&T pump at 10,000 cfs.

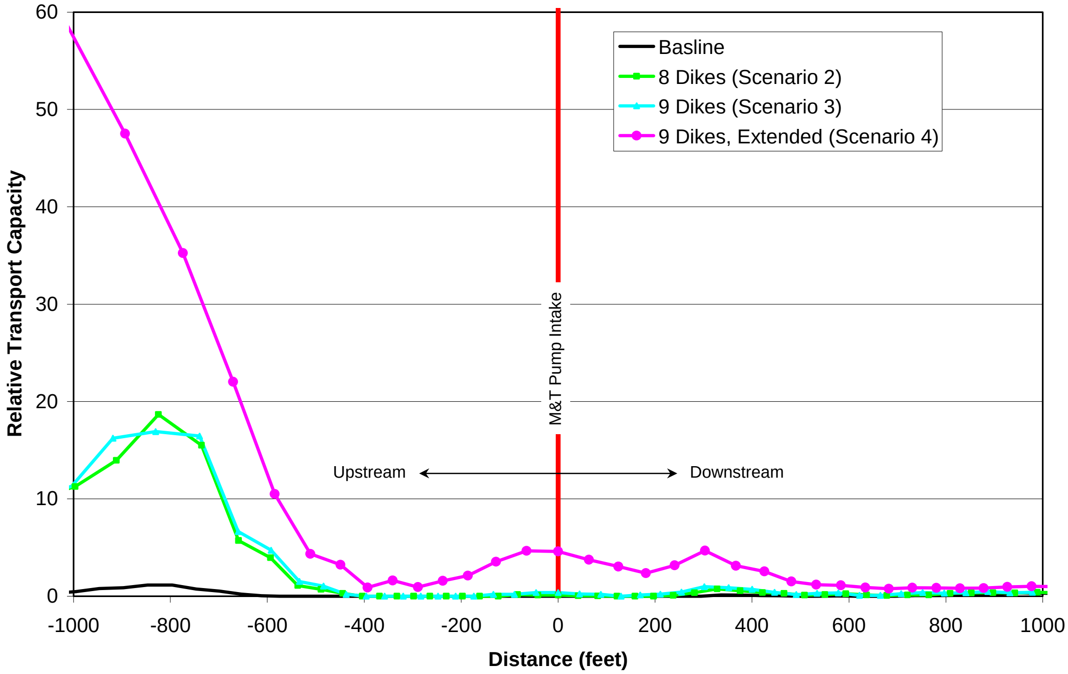

Figure 3.23. Relative transport capacity for 1.0 mm sand along the path of a typical particle that passes over the M&T pump intake at 10,000 cfs for the baseline (Scenario1), 8-dike (Scenario 2), 9-dike (Scenario 3) and extended-dike (Scenario 4) configurations.

values at 90,000 cfs at the head and middle of the bar and somewhat higher values throughout the rest of the modeled area. In short, the current analysis using updated data indicates that under current baseline conditions, the sediments are less mobile than was indicated by the previous analysis.

The impacts of the changes in hydraulic conditions and sediment gradation on the effectiveness of the dikes can be seen by comparing Figure 3.16 in the initial analysis (MEI, 2005) with Figure 3.18 (note the differences in the color scales on the figures). With the 8-dike scenario, the initial analysis showed that the NGS values along the margin of the bar at 90,000 cfs were in the range of 1.5 to 2.5, and hence the bed materials would have been mobile. In contrast, Figure 3.18 now shows that the NGS values are only in the range of 1.1 to 1.3, and therefore, the bed material is less mobile and the 8-dike configuration is less effective.

3.2.2. 9-Dike Configuration (Scenario 3)

Based on the information presented in the previous section, the originally proposed 8-dike configuration should be effective in preventing further lateral erosion of the west bank and downstream migration of the primary gravel bar, but this configuration would do little to reduce the tendency for sand deposition in the vicinity of the pump intake during low to moderate flow conditions. As a result, an additional 150-foot dike was added to the model approximately 350 feet upstream from the intake to further constrict the river (Figure 3.1), under the hypothesis that the additional constriction would force more flow toward the left bank, increasing the hydraulic energy and transport capacity in the vicinity of the intake. The additional dike has the same design geometry as the other eight dikes, and the top was also placed at 35,000 cfs water-surface elevation.

Model results for this configuration indicate that the hydraulic conditions over most of the site, including the primary gravel bar, are identical to those under the 8-dike configuration (Scenario 2) (**Figures 3.24 and 3.25**). The additional dike, however, creates an obstruction that shifts a portion of the flow toward the left bank and the pump intake, causing a small increase in shear stress in the vicinity of the intake (Figure 3.20). In spite of the increase in shear, the NGS for the gravel- and cobble-sized material between the downstream end of the primary bar and the intake is still below critical conditions at the bankfull flow of 90,000 cfs, and the transport capacity for sand-sized material also remains very small at low to intermediate flows.

3.3. Extended 9-Dike Spur Model (Scenario 4)

The dike lengths for Scenarios 2 and 3 were established with the intent of narrowing the river to improve continuity of gravel transport through the reach, while limiting the amount of rock that would be required to construct the dikes. For these configurations, the nose of the spurs essentially parallels the existing, eroded bankline, which does not eliminate the expansion zone that was created by the bank erosion that has occurred over the past several years (Figure 3.1). While these configurations would be effective in preventing further migration of the right bank and further downstream migration of the gravel bar, they do not substantially improve the flow conditions that lead to sand deposition in the vicinity of the intake during low to intermediate flows. An additional model configuration was, therefore, evaluated with the three downstream dikes for the 9-dike configuration extended to the approximate location of the 1996 bankline, and the five upstream dikes shortened to approximately 100 feet (the minimum length that would provide effective bank protection) (Scenario 4, Figure 3.2). The three downstream dikes were also raised to full bank height to maximize the amount of flow constriction at the downstream end of the gravel bar and in the vicinity of the pump intake. Due to the considerable length of the extended dikes, it was not possible to maintain the 5-percent top slope. As a result, the crest of the dikes was sloped upward at 5 percent from the nose until it reached the top-of-bank elevation, and then maintained at that elevation to the point of intersection with the bank. The extended 9-dike configuration was then modeled at 10,000 and 90,000 cfs to evaluate the low- and high-flow sediment-transport characteristics.

Model results for this configuration indicate that the bed shear stress at 90,000 cfs would increase significantly between about Dike 7, near the downstream end of the gravel bar, and about 500 feet downstream from the pump intake, compared to the other scenarios that were analyzed **(Figures 3.26 and 3.27**). Under Scenarios 2 and 3, the bed shear at 90,000 cfs is near or below critical conditions for mobilization of the typical gravel- and cobble-bed material in this portion of the reach (Figures 3.18 and 3.24), while this scenario would increase the NGS to well above critical. The increase in shear stress occurs mostly along the main flow path adjacent to the pump intake, but the shear stress also increases at the intake (Figure 3.20). The dike configuration for this scenario would also cause a significant increase in the transport capacity of sand-sized material in the vicinity of the intake at low to intermediate flows (Figure 3.23).

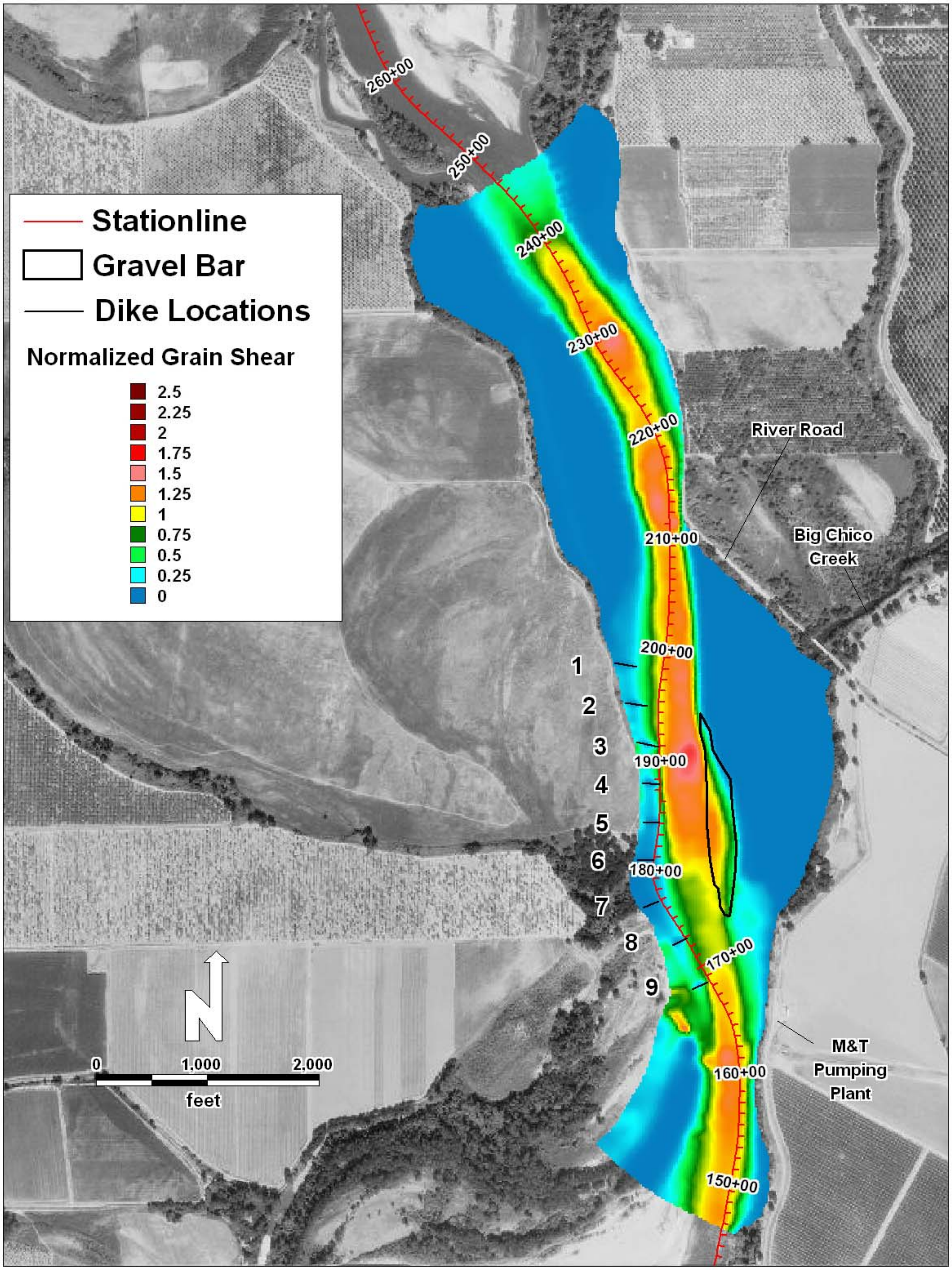

Figure 3.24. Normalized grain shear stress distribution at 90,000 cfs with the 9-dike configuration (Scenario 3).

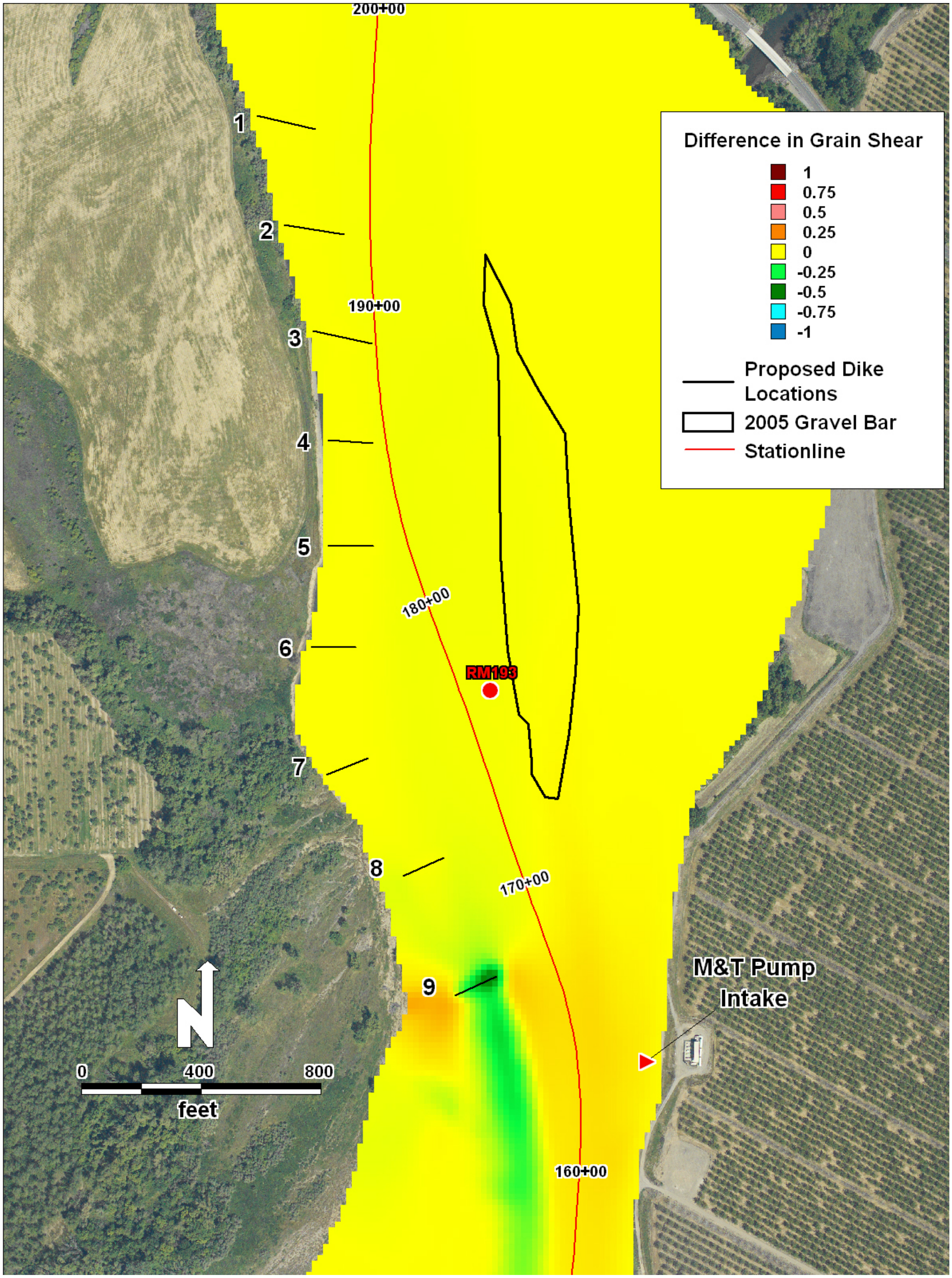

Figure 3.25. Difference in normalized grain shear at 90,000 cfs between the 9-dike configuration (Scenario 3) and 8-dike configuration (Scenario 2).

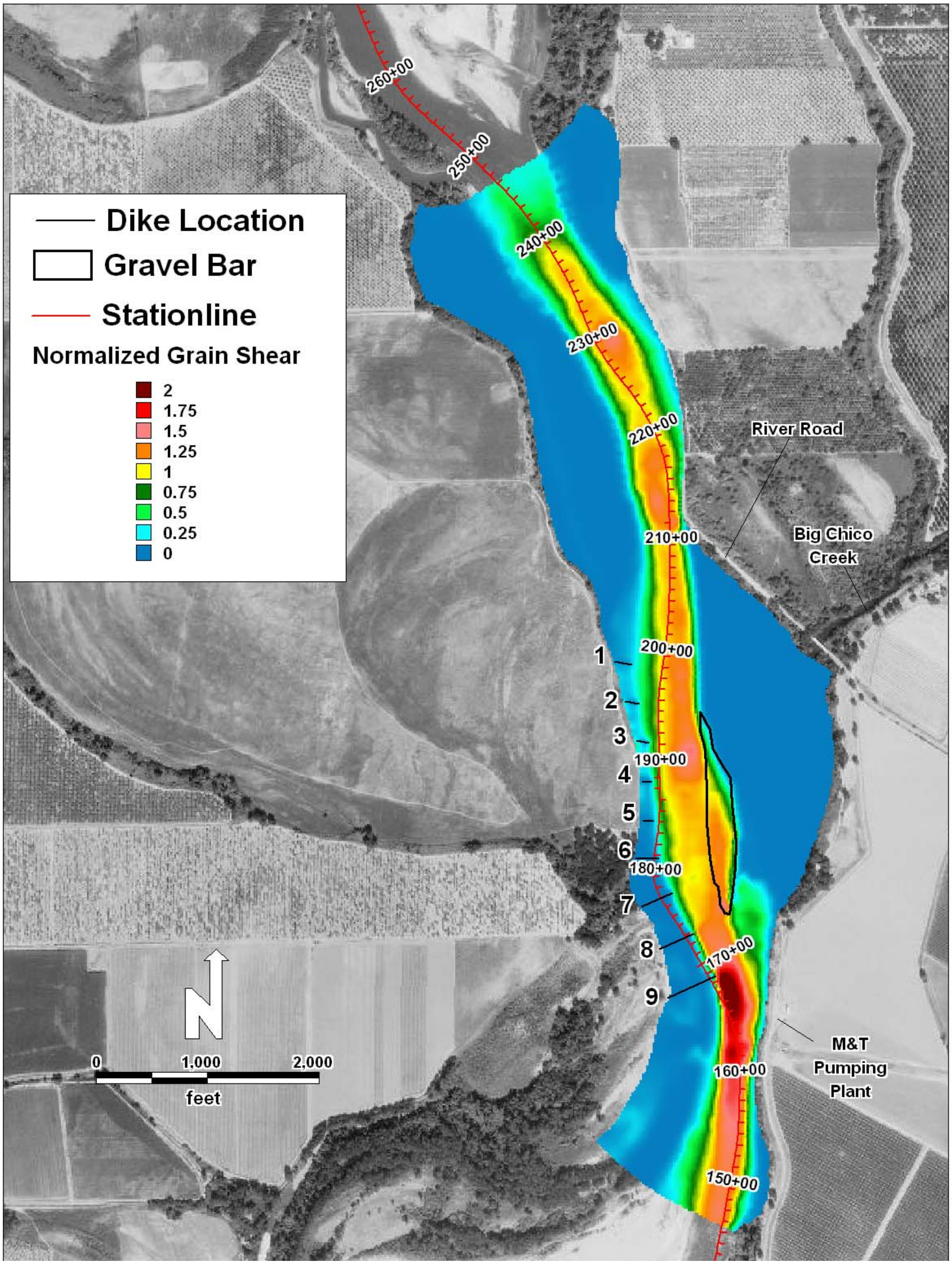

Figure 3.26. Normalized grain shear stress distribution at 90,000 cfs with the extended 9-dike configuration (Scenario 4).

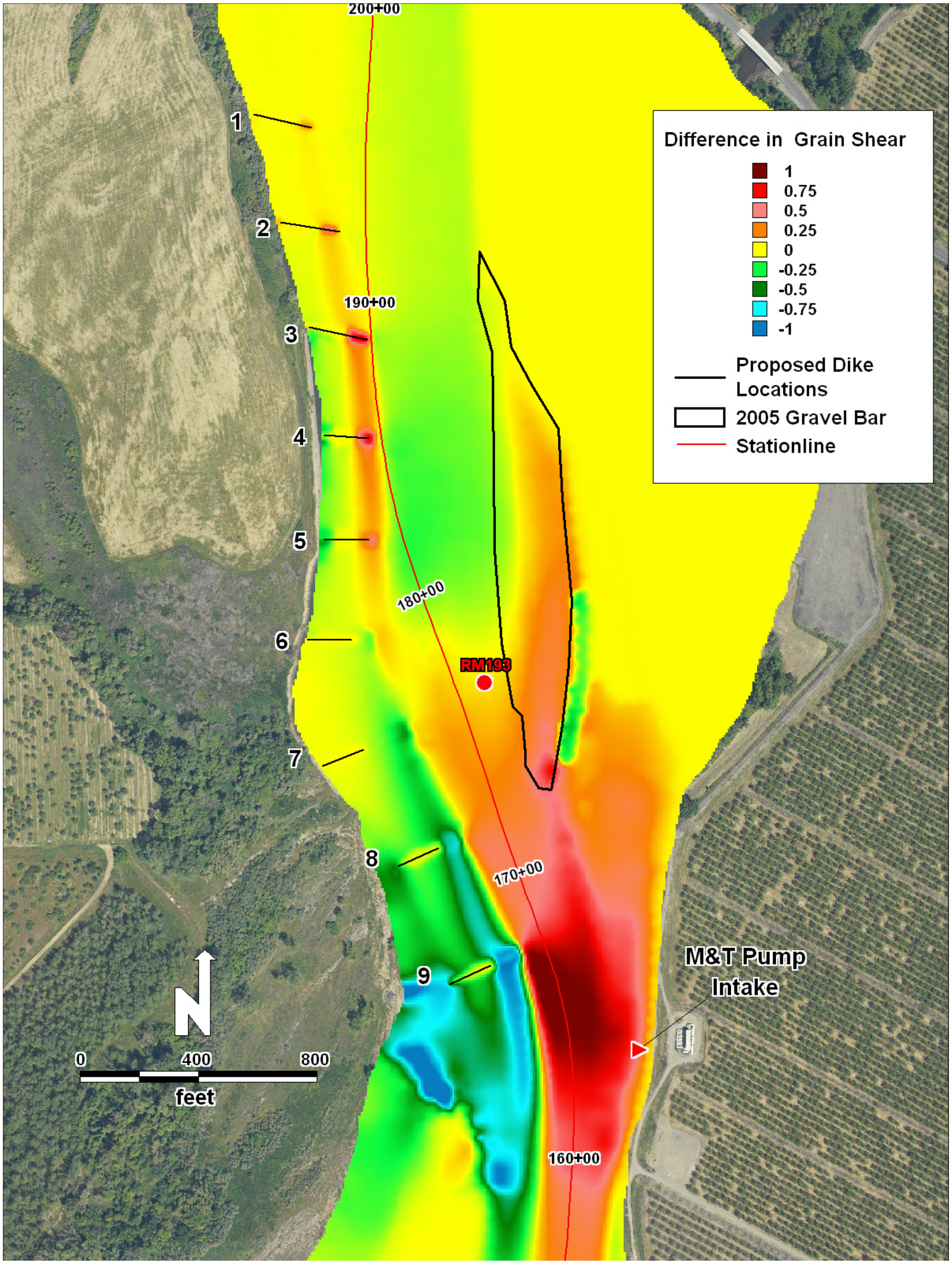

Figure 3.27. Difference in normalized grain shear at 90,000 cfs between the extended 9-dike configuration (Scenario 4) and 9-dike configuration (Scenario 3).

4. ADDITIONAL INFORMATION

4.1. Construction Cost Estimates

Preliminary construction cost estimates for Scenarios 2, 3 and 4 were developed for purposes of assessing construction feasibility and comparison with three other alternatives that were previously provided to the Steering Committee by MWH (2004) (**Table 4.1**). Two different estimates were prepared for each scenario, using different assumptions regarding the mitigation requirements and source of the rock. For the first estimate (**Table 4.2**), the mitigation costs (Item 7) were assigned on a per unit length basis of disturbed bank length along the entire reach of affected bankline, assuming that all of the rock would be obtained from a local quarry. The project will likely be treated as full bank revetment for mitigation purposes, and the unit cost of mitigation is estimated to be about $800 per foot of bankline, based on costs incurred in other recent Sacramento River projects. For the second estimate (**Table 4.3**), it was assumed that the mitigation requirements (Items 7) can be offset by removing a similar length of riprap from the M&T Ranch property at Golden State Island, hauling this rock to the M&T site, and incorporating the rock into the dikes. This would also likely result in a small decrease in the cost of the rock.

Table 4.1. Summary of estimated construction costs for three alternatives provided to the Steering Committee by MWH (2004).

Alternative	Description	Cost[1]
1	Install additional tee fish screen	$ 6,391,800
2	Groundwater extracted with production wells	$ 5,984,400
3	Groundwater extracted with Ranney wells	$ 15,376,200

[1]*The estimate of construction costs are at the feasibility level, and therefore very preliminary in nature. They exclude operations, maintenance and replacement costs* (MWH, 2004).

The total project cost for each case that was evaluated includes lump sum estimates for design and pre-construction tasks (Items 1 through 5) that were previously provided by MWH (2004), as well as a $50,000 budget for a water quality monitoring program (Item 6), based on the Butte City Dike project. The actual construction cost (Item 8) was estimated based on anticipated quantities for each of the significant line-items, plus a lump sum mobilization cost of $100,000. Detailed breakdowns of the quantities, approximate unit costs, and total construction costs for the three scenarios under each of the two potential mitigation strategies are provided in **Appendix B**. Unit costs used in the estimates for new riprap and fill gravel were provided by Carl Woods Construction in Yuba City CA (personal communication, April 2006), and other unit costs were based on information from R.S. Means (2006) and known costs of similar work in the region for site clearing and preparation, revegetation and clean-up. Operations and Maintenance (O&M) costs were based on the Corps of Engineers Planning Study–level estimate of 1 percent of the construction cost per year (Mr. Dan Tibbitts, Hydraulics Section, Sacramento District, Corps of Engineers, personal communication, April 2006). Since the rate of return and the rate of inflation are both assumed to be 4 percent (Olen Zirkle, DU, personal communication, April 2006), the present value of the O&M estimates for the various scenarios were developed by multiplying the annual rate (1%) by the agreed upon project life (40 years).

Other assumptions used in preparing the estimates included the following:

- Approximately 20 acres would be cleared to provide sufficient access to the site and working area.
- All new riprap would be delivered to the site by the quarry.
- A long-reach excavator would be used to remove the dike root material, and this material would be deposed of at the site.
- Site revegetation and cleanup will be required.

For the second mitigation strategy, it was assumed that the riprap along the left bank of Golden State Island is appropriate for dike construction, and the available quantity was estimated assuming that the riprap extends over 2,500 feet of bankline at a height of 20 feet and thickness of 3 feet, which provides approximately 5,600 yd^3 of riprap (6,975 tons).

Using the above assumptions, the total project cost with all new rock and complete mitigation for the site is estimated to be about \$7.1M for the 8-dike scenario (Scenario 2), \$7.8M for the 9-dike scenario at the originally proposed dike length, and \$11.4M for the extended 9-dike scenario (Scenario 3). The estimates decrease to \$4.3M, \$4.6M, and \$8.2M for the three scenarios, respectively, if the second mitigation strategy can be used.

Table 4.2. Estimated cost for construction of Scenarios 2, 3 and 4, assuming all new rock and mitigation costs of $800 per foot of bankline.

Item	Item Description	Total		
		Scenario 2 (8 Dikes)	Scenario 3 (9 Dikes)	Scenario 4 (9 Dikes Extended)
1	30% Engineering Cost	$ 150,000	$ 150,000	$ 150,000
2	Physical and Computer Modeling	$ 150,000	$ 150,000	$ 150,000
3	Environmental Document	$ 400,000	$ 400,000	$ 400,000
4	Final Design	$ 500,000	$ 500,000	$ 500,000
5	Construction Management	$ 300,000	$ 300,000	$ 300,000
6	Water Quality Monitoring	$ 50,000	$ 50,000	$ 50,000
7	Mitigation	$ 2,160,000	$ 2,480,000	$ 2,480,000
8	Construction	$ 1,960,000	$ 2,210,000	$ 5,060,000
9	Contingency @ 25%	$ 1,420,000	$ 1,560,000	$ 2,270,000
10	Operations & Maintenance (1% per year PV)	$ 784,000	$ 884,000	$ 2,024,000
	Total	**$ 7,870,000**	**$ 8,680,000**	**$ 13,380,000**

Table 4.3. Estimated cost for construction of Scenarios 2, 3 and 4, assuming mitigation can be offset by removing rock from Golden State Island and using this rock at the M&T site.

Item	Item Description	Total Scenario 2 (8 Dikes)	Total Scenario 3 (9 Dikes)	Total Scenario 4 (9 Dikes Extended)
1	30% Engineering Cost	$ 150,000	$ 150,000	$ 150,000
2	Physical and Computer Modeling	$ 150,000	$ 150,000	$ 150,000
3	Environmental Document	$ 400,000	$ 400,000	$ 400,000
4	Final Design	$ 500,000	$ 500,000	$ 500,000
5	Construction Management	$ 300,000	$ 300,000	$ 300,000
6	Water Quality Monitoring	$ 50,000	$ 50,000	$ 50,000
7	Mitigation	$ -	$ -	$ -
8	Construction	$ 1,900,000	$ 2,150,000	$ 4,990,000
9	Contingency @ 25%	$ 860,000	$ 930,000	$ 1,640,000
10	Operations & Maintenance (1% per year PV)	$ 760,000	$ 860,000	$ 1,996,000
	Total	**$ 5,070,000**	**$ 5,490,000**	**$ 10,180,000**

4.2. Adaptive Management Experiment

Other than the recent installation of spur dikes at the Butte City Bridge by CALTRANS, spur dikes have not been extensively used as a form of bank protection on the Sacramento River (Harvey et al., 2004). However, spur dikes have been used on many other rivers, including the Yuba and American Rivers, to successfully prevent bank erosion and retreat Examples of successful installations are provided in Appendix A. Laboratory and field studies of a dike field on the Willamette River in Oregon have demonstrated that the dikes provide acceptable levels of bank protection and induce significant between-dike sedimentation that forms the substrate for riparian vegetation growth (Klingeman et al., 1984). Dikes have also been used extensively on the Mississippi, Missouri, and Red Rivers to change channel alignments to favor navigation (Lindner, 1969; Winkley, 1994; USACE, 2002b). Review of the literature (Klingeman et al., 1984; Shields et al., 1995; Lacey and Millar, 2004) suggests that spur dikes can have beneficial environmental effects, and they are a more ecologically/biologically acceptable form of bank stabilization when it is necessary to prevent river meandering to protect identified riverside infrastructure as envisaged in the House Bill 1086 process. Since the effectiveness for erosion control and environmental impacts of spur dikes have not been investigated on the Sacramento River, the Steering Committee recommended that an Adaptive Management-based experiment be developed for the proposed spur dikes that would enable their physical and ecological/biological effects to be assessed. The Adaptive Management experiment requires development of hypotheses and identification of quantitative performance measures for both erosion control and ecological/biological effects that can be used to test the hypotheses.

It is anticipated that the experiment will involve comparison of physical and ecological/biological characteristics in the spur dike reach between RM 192.7 and RM 193.3 with those characteristics in a geomorphically similar reach which will continue to erode. A candidate site for the comparison is located at approximately RM 173. Information derived from the experiment can then be used to inform decisions regarding future bank protection and its ecological/biological impacts at other required locations on the Sacramento River system.

Furthermore, the results of the experiment will provide a means of quantifying the ecological/biological effects of dikes, which in turn will provide a sound basis for establishing mitigation requirements.

The experiment will test the following hypotheses:

1. The spur dikes will stabilize the west bank of the Sacramento River and prevent further bank retreat,
2. The spur dikes will prevent the bank-attached bar upstream of the M&T pumps from migrating downstream,
3. The spur dikes will provide adequate velocities and shear stresses at the fish screens and the pump inlets to prevent sand build up during the irrigation season when flows in the river are between about 4,000 and 14,000 cfs,
4. The spur dikes will increase the area of low velocity habitat for fry and juvenile salmonid species during high flows,
5. The spur dikes will increase areas of greater depth and, therefore, adult habitat during low-flow periods due to high-flow scour around the ends of the dikes, and
6. The spur dikes will not increase the level of predation of juvenile salmonids by native and non-native predator fish species.

To test the hypotheses, a 5-year data collection and analysis program is required, the results of which will determine the effectiveness of the dikes in providing both physical and biological benefits or detriments. Hypotheses 1, 2, and 3 can be addressed with a monitoring program that would be implemented after high flow events. As-built surveys would be conducted following completion of the dike installation. Following each high flow event, visual inspection of the dikes will provide an initial evaluation of any erosion of the west bank or downstream migration of the bank-attached bar. If sufficient change is observed, the magnitude of the change will be quantified by resurveys. Before and after the pumping season, diver inspection can be used to determine if sand has accumulated at the fish-screens and pump inlets, and velocity measurements can be made to test the accuracy of the 2-D model predictions. The amount of sand deposition can be quantified by the divers, or if the deposition is significant, with a bathymetric survey. At the time of the as-built survey of the dike field, a bathymetric and topographic survey of the river at RM 173 will also be completed to determine the baseline condition. Surveys of the RM 173 site will be conducted after each high-flow event to document physical changes, including bank erosion and retreat and downstream migration of the opposing bank-attached bar.

Hypotheses 4, 5 and 6 can be tested with a biological monitoring program at both the spur dike and RM 173 sites. Physical and biological monitoring can be used to identify the amount of low velocity habitat and use of available habitat by fry and juvenile salmonids during high flows at each site. Similarly, physical and biological monitoring can be used to evaluate the presence or absence of deeper pools during low flow periods at the spur dike site, and to assess usage by various life stages of the salmonids and the presence of native and non-native predators.

At the end of the 5-year monitoring period, assuming that there have been a reasonable number of high flow events, statistical analysis of the physical and biological monitoring data at the two sites will allow the hypotheses to be tested. The results of the adaptive management experiment can then be used to inform future decisions regarding the use of dikes to prevent bank erosion at critical sites, and to provide a quantifiable basis for assessing the biological mitigation requirements for this form of bank stabilization.

Estimated costs for baseline surveys and subsequent monitoring are:

Item	Estimated Cost
Baseline surveys of RM 193 and RM 173 sites	$40,000
Five resurveys of RM 193 and RM 173 sites	$125,000
Five dive surveys at RM 192.75	$30,000
5 years of biological monitoring at RM 193 and RM 173 sites	$150,000
TOTAL	**$345,000**

4.3. Physical Modeling

If one of the dike options is ultimately selected for further consideration, it is strongly recommended that additional analysis be conducted using a physical model to validate the numerical model results. The primary focus of the model would be to obtain more detailed information about the potential for the dikes to eliminate sand deposition at the pump intakes, however, the model could also be used to validate the flow patterns and bed shear stress distributions that are important to understanding the effects of the project on instream habitat and evolution of the gravel bar. MEI coordinated with the Hydraulic Engineering Laboratory at Colorado State University to develop the following preliminary workplan and budget-level cost estimate for the physical modeling.

4.3.1. Scope of Work

The specific objectives of the physical model study would be as follows.

1. Construct a physical model of the proposed dike field and pump intake. It is anticipated that the model would extend from approximately River Sta 160+00 upstream to Sta 200+00 (Figure 3.26).
2. Quantify the three dimensional flow conditions within the proposed dike field.
3. Verify the proposed dike layout.
4. Introduce appropriately scaled sediment to represent the sand fraction of the sediment load into the model and quantify deposition patterns within the study area.

To meet these objectives, a Froude scale physical model will be constructed of the project reach. At this time, it is anticipated that an indoor river modeling facility that consists of an existing flume that is 100 feet long by 25 feet wide and 4 feet deep will be used (**Figure 4.1**). Water is supplied to the facility by a re-circulating pump system capable of providing discharges up to 45 cfs. A mobile data acquisition cart spans the flume permitting data to be collected at any location throughout a given model.

Flow data collected during each test can include water-surface elevations, 1-, 2- or 3-dimensional point velocities, shear stress measurements using a Preston Tube and sediment concentrations. Photographic and video records of each test segment will be made with digital pictures and 8mm video.

Figure 4.1. Photograph of river modeling facility.

4.3.2. Model Scale

Based on the dimensions of the study reach and flume, the model could be built at an undistorted 1:50 Froude scale, to provide optimization between construction costs and model resolution. It is the opinion of Colorado State University that this scale will be adequate to quantify the hydraulic variables of interest and qualitatively assess the effect of the dike field on sand-sized sediment moving through the project reach. A summary of approximate prototype and corresponding model scale values for variables of interest for the recommended model scale is provided in **Table 4.4**.

Table 4.4. Summary of model variables.

	Prototype	Model Scale 1:50
Max discharge	374,100 cfs	21.2
Min discharge	10,000 cfs	0.6
Model length	4,000 feet	80.0
Model width	1,250 feet	25.0
Sediment size	1.0 mm	0.16
Dike length	120-210 feet	2.4-4.2

Several methods are available to scale sediment in a physical model. Typically, similitude is determined in models that investigate sediment transport using the Rouse Number (z), which is defined by the following equation:

$$z = \frac{w_s}{ku_*} \tag{4.1}$$

where: w = particle settling viscosity
k = Von Karman's constant = 0.4
u_* = shear velocity

The Rouse Number is a dimensionless parameter relating the rate of fall of a suspended particle to the strength of turbulence acting on the particle. Holding the prototype and model Rouse numbers constant, scaling of the particle fall velocity (w) becomes a factor of the square root of the model length ratio. Variations in particle size and density can then be examined to determine an appropriate material to place in the model.

Assuming the appropriate prototype sediment size for the sand load is approximately 1.0 mm, Rouse Number scaling produces a required model sediment size of approximately 0.16 mm. Material of this type is commercially available and can be used for the proposed model study.

4.3.3. Test Program

Following construction of the model, a series of initial shakedown runs will be conducted prior to the actual data collection runs. If the modeling is done at CSU, they will likely request that representatives from the Technical Advisory Committee (TAC) and the owners be present during these runs to verify that the completed model represents the intended conditions prior to the commencement of the actual test runs.

For the purposes of preparing a cost and schedule estimate, it is assumed that model operations will occur over an approximately three (3) weeks period. During that time, shop personnel will be available to modify and adjust the model, as necessary. Again, if the testing is done at CSU, they will likely request that MEI and/or representatives of the owner be on-site to direct modifications to the structure geometry.

4.3.4. Estimated Modeling Costs

The estimated cost to construct the model, conduct the testing and prepare a project report is approximately $190,000 (**Table 4.5**).

Table 4.5. Summary of estimated project costs.

Construct 1:50 Model and remove following testing	$47,500
Model Testing	$33,300
Project supervision, administration and final report preparation	$20,600
Subtotal	$101,200
Normal University overhead @ 46%	$46,600
Total Laboratory Cost	**$147,800**
MEI Oversight	$15,000
Subtotal	**$163,000**
Contingency (15%)	**$25,000**
TOTAL ESTIMATED COST	**$188,000**

5. SUMMARY AND CONCLUSIONS

5.1. Summary

Reanalysis of the hydraulic and sediment mobility characteristics of the M&T reach of the Sacramento River to evaluate the potential of a series of spur dikes along the west bank to solve existing and future sedimentation problems at the M&T fish screens and pump intake was carried out in this study to fully represent the existing topography and bathymetry of the site and the existing gradation of the sediments that comprise the bank-attached bar that is currently located upstream of the inlet. The previous investigation (MEI, 2005) was based on modifications to the topography and bathymetry that were collected by Ayres Associates in 1996, and a bed-material sample that was collected in 1990 from the head of the bar at Bidwell State Park (WET, 1990).

The specific objectives of the study were to:

1. Evaluate whether the spur dikes would prevent further erosion of the west bank of the Sacramento River upstream of the M&T pumps because the 330 feet of erosion and bank retreat that has occurred between 1996 and 2006 has been identified as the fundamental cause of the sedimentation problems at the pumps (Harvey et al., 2004),
2. Evaluate whether the spur dikes would prevent downstream migration of the bank-attached gravel bar located on the east side of the Sacramento River upstream of the M&T pump inlet, and
3. Evaluate whether the spur dikes would cause high enough velocities during the normal range of pumping flows (4,000 to 14,000 cfs) to prevent sand accumulation around the fish screens and pump inlets.
4. Prepare conceptual-level cost estimates for the dike scenarios that can be compared to other alternatives for addressing the problems at the M&T intake.
5. Develop an Adaptive Management Experiment that will enable the physical and ecological/biological effects of the dikes to be assessed.
6. Prepare a workplan and budget estimate for a physical model study further evaluate the selected dike alternative.

To address the objectives of the investigation, the 2-D hydrodynamic model (RMA2) of the site was modified to represent the existing (late 2005) topography and bathymetry of the Sacramento River from RM 192.5 (Sta 149+95) to RM 194.35 (Sta 245+88). The M&T pumps are located at RM 192.75 (Sta 164+26) (Figure 1.1). Models were developed for the following scenarios:

1. 2005 channel alignment and geometry (baseline conditions—Scenario 1),
2. Proposed 8-dike configuration (Scenario 2, Figure 3.1),
3. Proposed 9-dike configuration (Scenario 3, Figure 3.1), and
4. Proposed extended 9-dike configuration (Scenario 4, Figure 3.2).

Modeled discharges ranged from 5,000 to 90,000 cfs which is the approximate bankfull discharge for the reach. Incipient motion and sediment transport calculations were conducted with output from the models and an average sediment gradation (Figure 3.12) with D_{50} and D_{84} of 39 and 60 mm, respectively, that was developed from three pebble counts on the bank-attached bar (Figure 3.13) in December 2005. A particle tracking analysis was used to better define the shear stress distribution and relative transport capacity of 1.0-mm sand at low to intermediate discharges in the vicinity of the fish screens and pump inlets.

5.2. Conclusions

This investigation led to the following conclusions:

1. With the current channel geometry and the existing bar material surface gradation, the bar sediments are less mobile than was indicated by the MEI (2005) analysis which tends to be supported by the observed rebuilding of the bar that was dredged in 2000.
2. Regardless of the number of dikes used, or their configuration, the reduced velocities and shear stresses along the west bank should prevent further erosion and retreat of the bank (Figures 3.18, 3.24, and 3.26).
3. All of the dike configurations [Scenario 2 (8 dikes), Scenario 3 (9 dikes) and Scenario 4 (extended 9 dikes)] will prevent downstream migration of the bank-attached bar on the east bank of the river upstream of the M&T pumps.
4. Based on the analysis of the relative transport capacity for 1.0-mm sand at the pumps at a discharge of 10,000 cfs, only the Scenario 4 dike configuration (extended 9 dikes) will prevent sand accumulation at the fish screens and pump inlets (Figure 3.23).
5. The total estimated project cost to construct the dikes for Scenario 2, 3 and 4, assuming that all new rock is used and environmental mitigation would be required for the entire approximately 3,200 feet of bankline, is $7.9M, $8.7M and $13.47M, respectively. If the mitigation can be offset by removing an equivalent length of bankline on the M&T property at Golden State Island, and using the removed rock at the M&T pump site, the estimated project costs for the three scenarios decreases to $5.1M, $5.5M, and $10.2M, respectively.
6. It would be necessary for the Adaptive Management experiment to extend over at least a 5-year period to obtain sufficient information, and the cost would be approximately $345,000.
7. A physical model study to evaluate the selected dike alternative could be conducted for approximately $190,000.

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