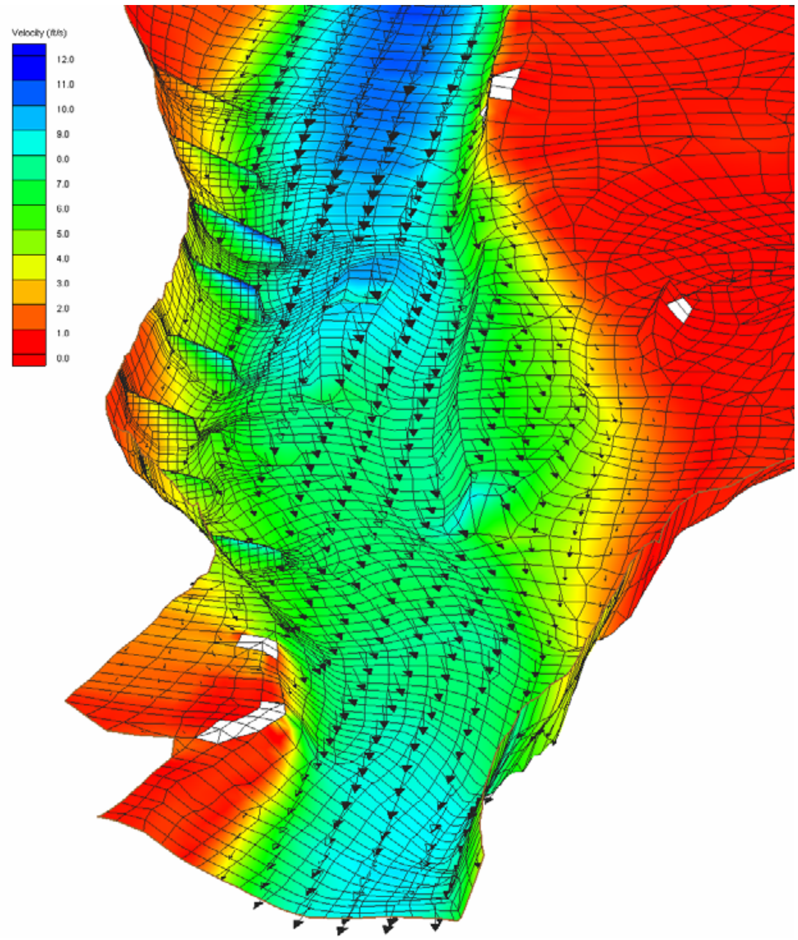
# Two-dimensional Modeling to Evaluate Potential River Training Works at M&T Pumping Plant Sacramento River, RM192.5



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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 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 facility. Based on the available aerial photography, the west bank eroded up to 270 feet just upstream from the intake between 1996 and 2003, and this bank may have migrated an additional 40 to 50 feet in response to high flows that have occurred over the past few months (Olen Zirkle, personal communication, 2005). 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, Larsen, Mussetter and Cui, 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 currently eroding riverbank to the east, preventing further erosion and potentially increasing flow velocities sufficiently to prevent further deposition in the vicinity of the intake. The analyses that were conducted for the original steering committee report were performed using hydraulics from an existing 1-dimensional (1-D) step-backwater model. Because these results are cross sectionally-averaged, they do not provide sufficient resolution to clearly evaluate the effects of the spur dikes on flow patterns and the lateral distribution of velocity and shear stress. As a result, the steering committee recommended that the spur dike alternative be evaluated in more detail using a 2-dimensional (2-D) model. This report presents the results of the 2-D analysis.

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 performed for the original steering committee report (Harvey, Larsen, Mussetter and Cui, 2004).

2. A 2-D hydraulic model 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), with the 1996 bathymetric survey and aerial photography performed by Ayres Associates (1996). The 2-D model was used in conjunction with the existing 1-D model to validate Manning's *n*- values and turbulence exchange coefficients (kinematic eddy viscosity values). The validated 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.

3. A model was developed to represent conditions in the reach that existed in 2003, to the extent possible given the available data, by modifying the 1996 conditions model based changes in the bank lines and bars that are visible in the 2003 aerial photography. This model was used to evaluate hydraulic and sediment-transport conditions in the vicinity of the M&T intake under existing (2003) topographic conditions.

4. An initial project conditions model was developed by modifying topography in the 2003 model to reflect the spur dikes that were proposed in original steering committee report, and this model was used to evaluate the potential effect of the spur dikes.

5. The 1996 conditions model was modified to reflect, to the extent possible given the available data, conditions that existed in 1979 prior to significant erosion of the west bank when width of the river through the reach was more uniform.

6. The 2003 baseline conditions model was modified to realign the left (east) bank revetment along River Road upstream from the M&T intake to determine whether this change would improve the flow alignment and transport capacity through the reach.

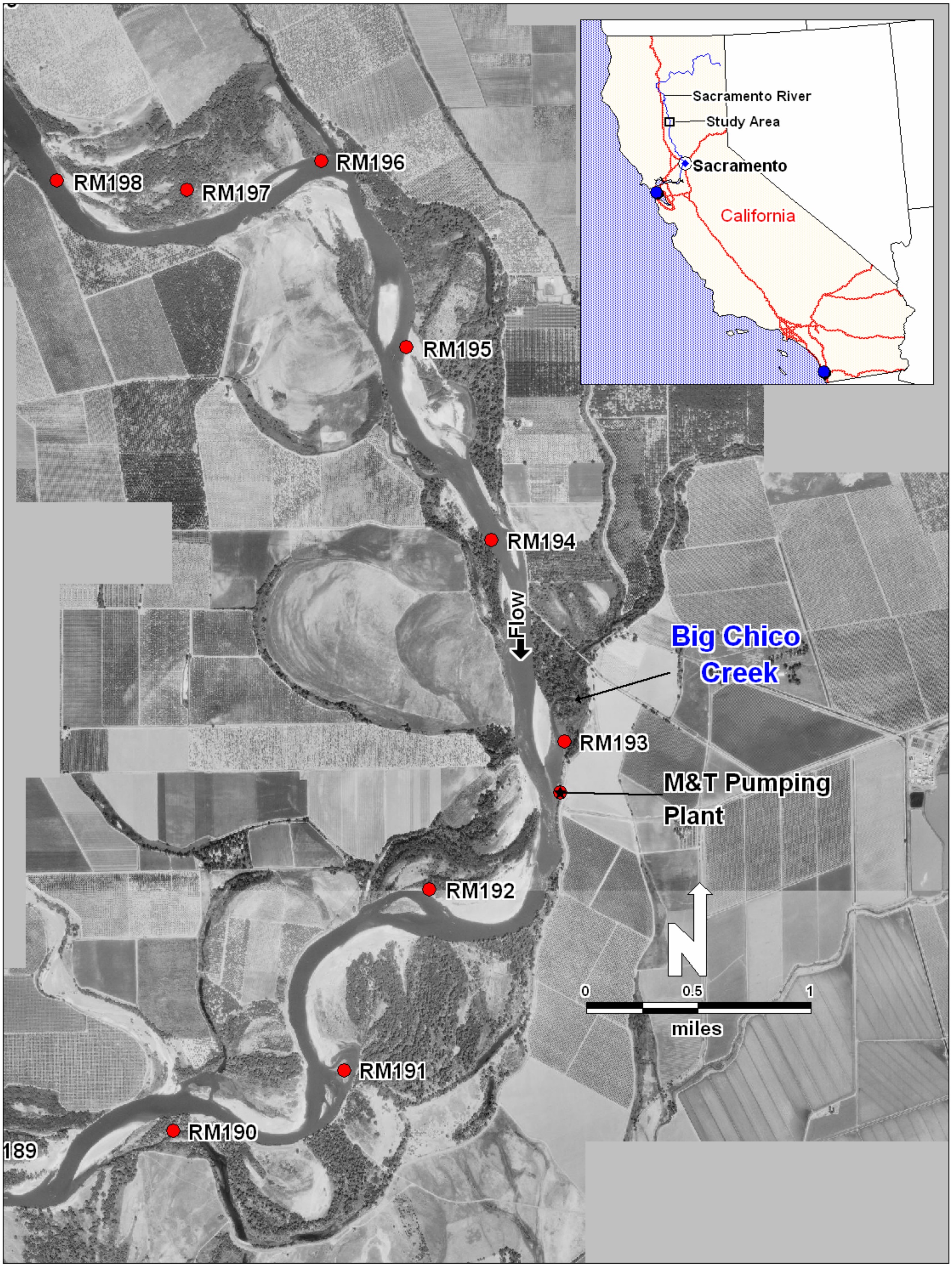


Figure 1.1. Location of the Sacramento River and the study site.

## 1.1. Authorization

This was carried out by 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 discharge regime in the study reach was evaluated based on recorded flows 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, and 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.

A flood-frequency curve for the study site was developed from the data at the Hamilton City gage using Weibull plotting positions for the annual flood peaks for the post-Shasta Dam period (1946-2003). [The standard log-Pearson Type III (LPIII) analysis that is typically used for flood-frequency analyses was not used because flow regulation by Shasta Dam causes the curve to deviate significantly from the LPIII frequency distribution.] Peak discharges used in the analysis for the period prior to 1980 were obtained directly from the USGS records. The USGS discontinued operation of the gage after 1980, and no data exist for the gage for the period from 1981 to 1994. The State of California reinstated the gage in 1995, and annual peak flows for the period from 1995 through 2003 were estimated from hourly data that is available from the California Data Exchange (CDEC) web site. Based on the resulting 45 data points (1946-1980, 1994-2003) the recurrence interval of the largest recorded flow is 46 years. Discharges for recurrence intervals greater than 46 years were obtained from USACE (1997a), which included estimates for floods to the 500-year event that were developed based on hypothetical storm modeling. The resulting flood frequency curves that represents the combination of the Weibull plotting positions for the recorded data and the estimated values is presented in **Figure 2.1**, and the peak discharges associated with various return period events are summarized in **Table 2.1**.

The curve in Figure 2.1 indicates that the 1.5- and 2-year peak flows are about 72,000 and 90,000 cfs, respectively. Based on results from the available hydraulic models, the average bankfull capacity of the channel in the vicinity of the M&T intake is about 90,000 cfs (see Chapter 3).

A mean daily flow-duration curve was also developed for the Hamilton City gage for the water years for which a complete data set was available (WY1946 through WY1980 from the USGS data, WY1997-WY2000 and WY2003 from the CDEC data, **Figure 2.2**). Based on the flow-duration curve, the median flow (flow that is equaled or exceeded 50 percent of the time) at the gage during the period was about 9,000 cfs, and the 10- and 90-percent exceedence flows were 23,160 and 5,460 cfs, respectively.

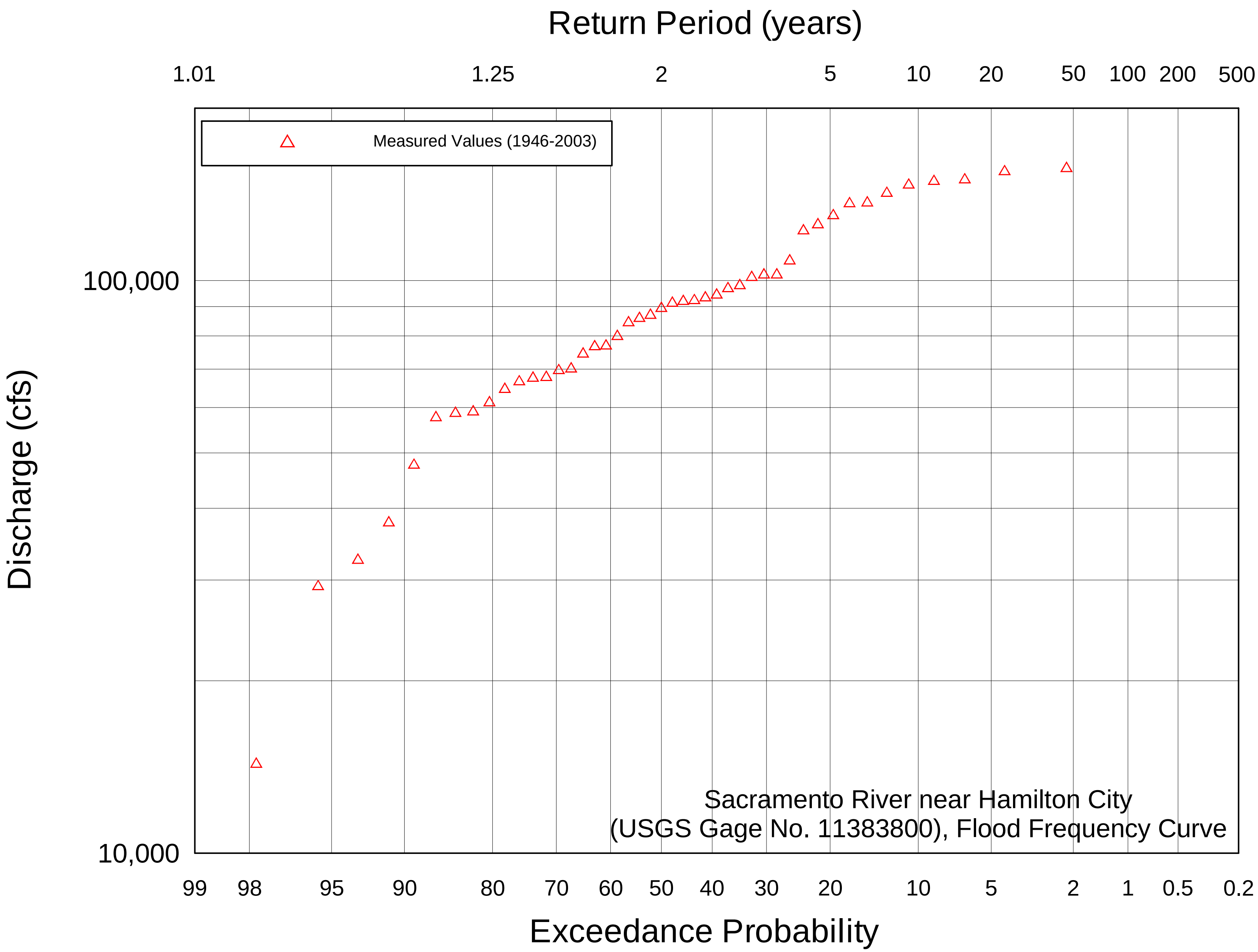


Figure 2.1. Post-Shasta Dam flood-frequency curve for Hamilton City gaging station.

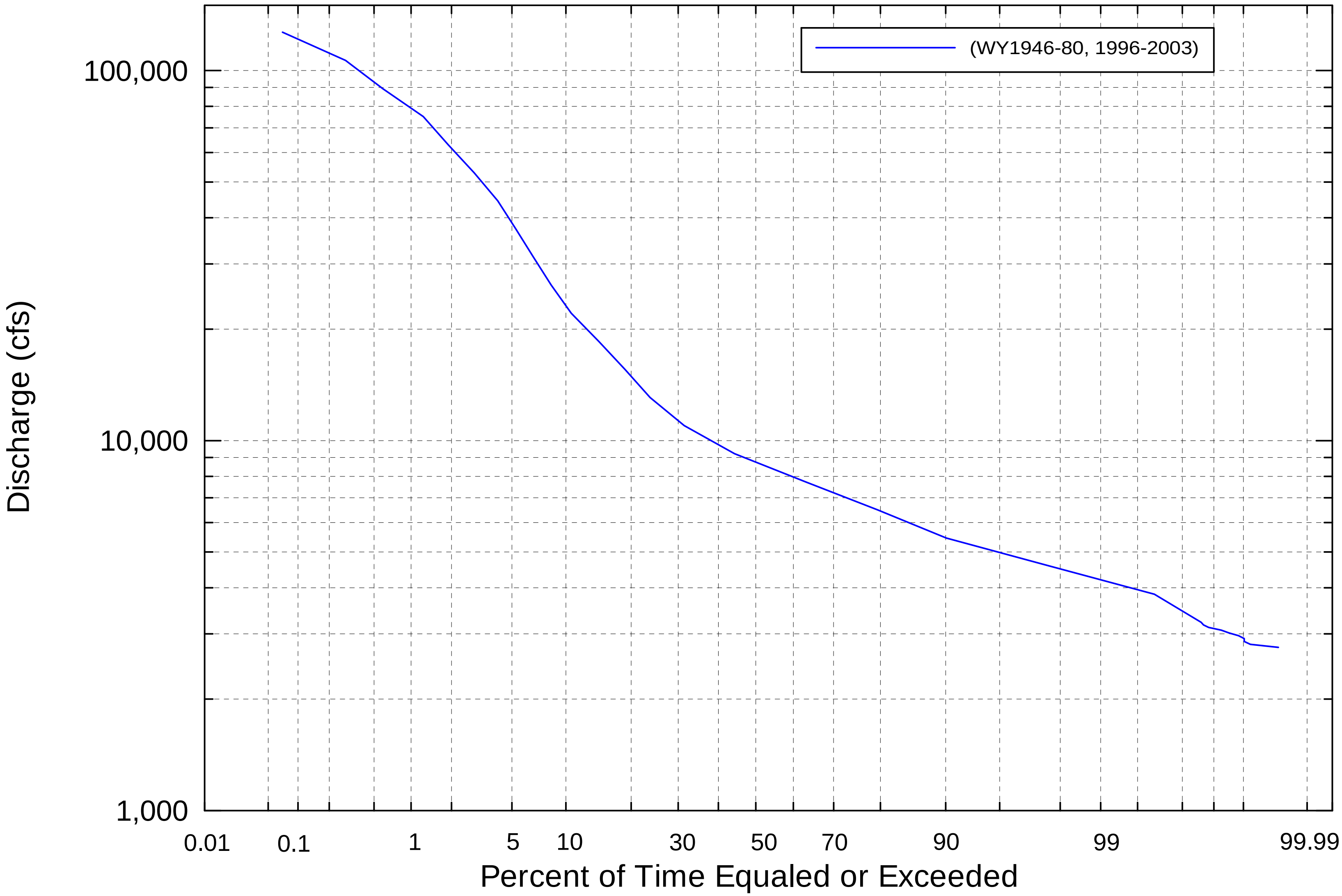


Figure 2.2. Post-Shasta flow-duration curve for Hamilton City gaging station.

| 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) |
| 59,300 | 1.20 |
| 72,000 | 1.5 |
| 90,000 | 2. |
| 129,700 | 5 |
| 148,600 | 10 |
| 179,500 | 50 |
| 374,100 | 100 |

Annual runoff past the Hamilton City gage during the 22-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 10.5M 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, 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.

An analysis of the flows from Big Chico Creek was conducted to aid in evaluating the hydraulic and sediment-transport effects on the Sacramento River. The confluence of Big Chico Creek with the Sacramento River is approximately 0.2 miles upstream of the M&T pumping station. Flow from the mouth of Big Chico Creek is directed towards the left side of the bar, and probably prevents the bar from attaching directly to the left bank upstream from the M&T Diversion pump. The flows from Big Chico Creek were evaluated to determine if any backwater effects were occurring in the vicinity of the bar, and if so, to evaluate any related effects on the location, geometry, and stability of the bar.

The gage on Big Chico Creek is located approximately 11 miles from mouth on the northeast side of the City of Chico and at the base of the Sierra Nevada mountains. Because flows from the Big Chico Creek drainage are largely unregulated and the drainage area is much smaller that the Sacramento River drainage upstream from the study reach, correlation between flows in the Sacramento River and Big Chico Creek is low (**Figure 2.4**). To facilitate the hydraulic modeling, a frequency analysis was conducted to identify the most likely range of flows in the creek when the discharge in the river is in the range of bankfull (85,000 cfs to 95,000 cfs) (**Figure 2.5**). The analysis indicates that the discharge in Big Chico Creek is most often in the range of 1,000 to 1,500 cfs under these conditions.

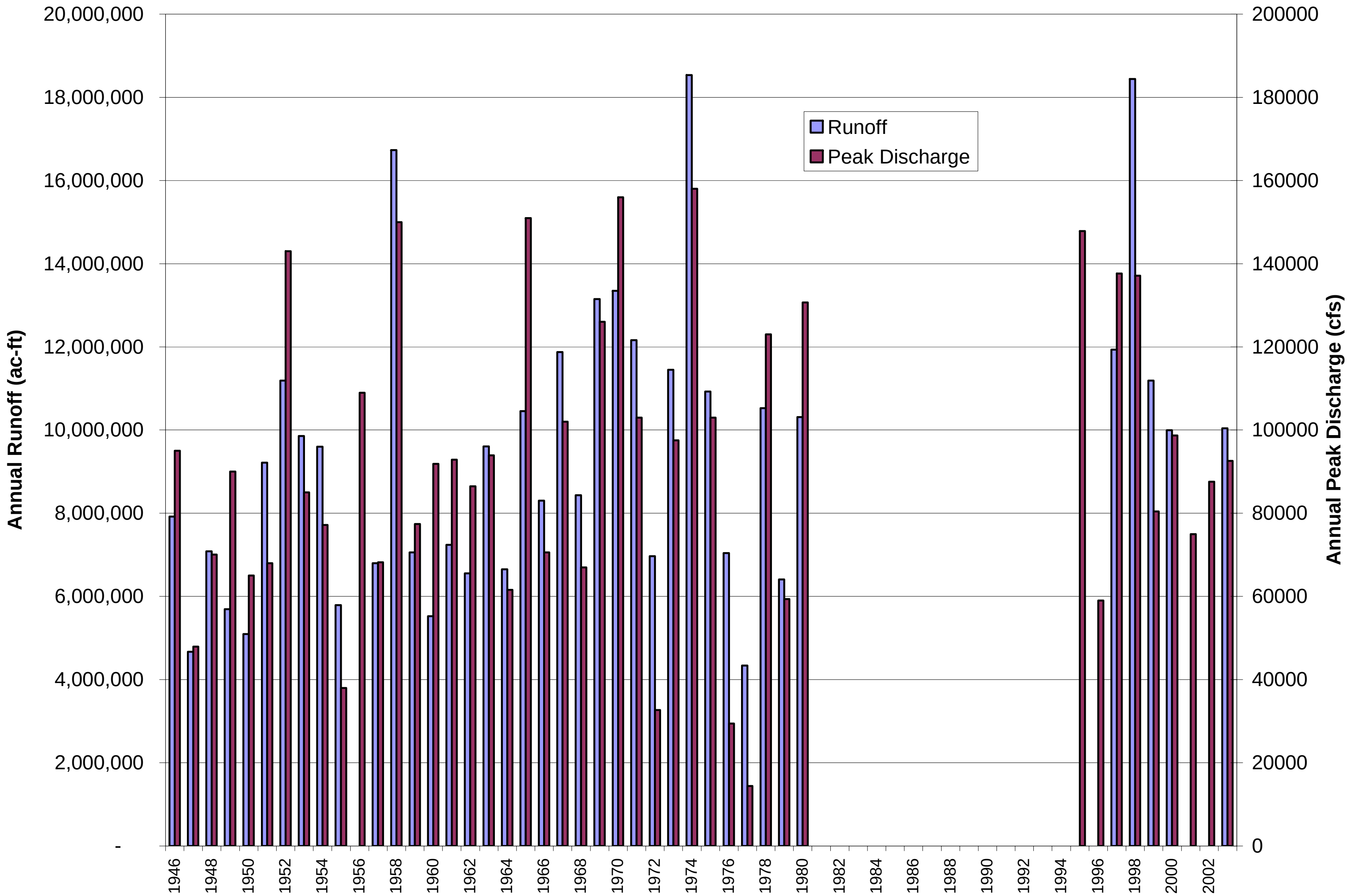


Figure 2.3. Annual runoff volumes and peak discharges at the Hamilton City gaging station from 1946 to 2003. No data are available for the 1981 to 1984 period.

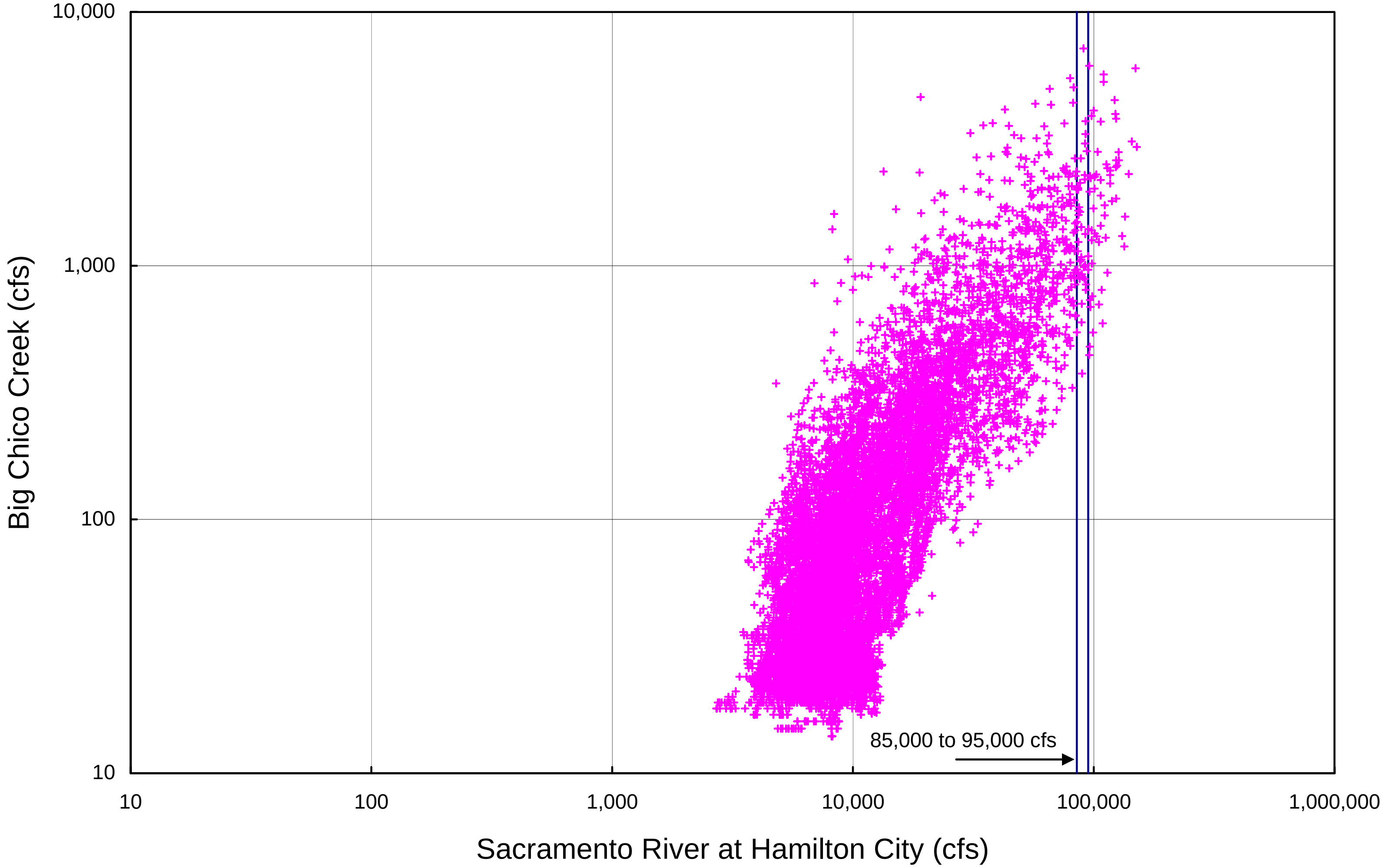


Figure 2.4. Concurrent mean daily flow values between the Sacramento River at Hamilton City and Big Chico Creek.

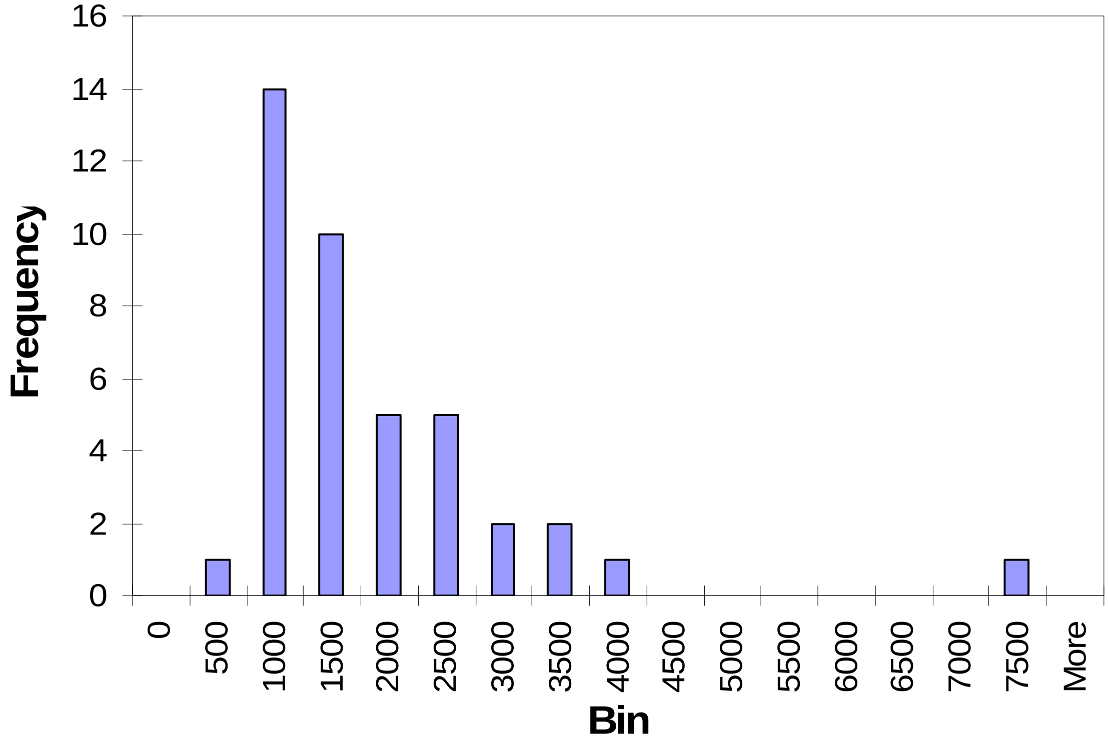


Figure 2.5. Histogram of discharges measured at Big Chico Creek when the concurrent discharge at the Hamilton City gage is between 85,000 and 95,000 cfs.

# 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 station 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 bank migration and development 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 project 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 initially developed and executed for the following scenarios:

1. 1996 channel alignment and geometry (1996 baseline conditions),
2. 2003 channel alignment and geometry (2003 baseline conditions), and
3. 2003 channel alignment with the proposed dikes (2003 design conditions).

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 of the 2003 design conditions model indicate that the area in the vicinity of the pump intake would remain depositional, even in the presence of the dikes. A model that represents, to the extent possible given the available data, conditions that existed in 1979, prior to erosion of the west bank, was developed to illustrate the changes in hydraulic and bed material transport conditions that occurred as a result of the channel changes. The 1979 conditions scenario was used for the historical analysis because the 1979 aerial photography showed a relatively uniform width through the reach than existed in the 1996 and later photographs, indicating that the hydraulic conditions and sediment-transport capacity was probably more uniform through the reach. In addition to the above, the 2003 baseline conditions model was also modified to realign the revetment on the left (east) bank along River Road, upstream from the mouth of Big Creek to determine if such realignment would decrease the likelihood of continued deposition in the vicinity of the pump intake.

## 3.1. 1996 Conditions Model

The 1996 conditions 2-D model was developed to provide a means of validating channel roughness and eddy viscosity values by comparison with the available 1-D model that has been calibrated for the reach. The results from this model was also compared to the 2003 conditions and 2003 design conditions scenarios to evaluate the magnitude of changes in hydraulic and sediment-transport conditions that occurred during the intervening 7 year period.

### 3.1.1. Topographic Data

A 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 geometry of the modeled reach, with the channel topography represented by bed elevations assigned to each node in the mesh. Topographic data for this scenario was derived from topographic and hydrographic surveys conducted by Ayres Associates in 1996, and obtained from the Corps of Engineers (USACE) through a Freedom of Information Act (FOIA) request. The data were obtained in Microstation DGN and DTM formats, with the horizontal datum referenced to State Plane Coordinate System, North American Datum of 1983 (NAD83 1992) and the vertical datum was based on the National Geodetic Vertical Datum of 1929 (NGVD29). To facilitate the 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 downstream end of the 1996 2-D model is located at Sta 92+50 (RM191.5) and the upstream end of the reach is located at Sta 250+00 (RM194.35, **Figure 3.1**). The M&T pumping station is located at Sta 164+00.

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 3.2**). The reach of river represented by the model is approximately 15,200 feet in length, extending from RM 191.5 (approximately 7,150 feet downstream from M&T pump intake) to RM 194.35 (approximately 8,100 feet upstream from the intake), and resulting mesh contains approximately 9,500 elements and 28,000 nodes. The density of the finite element mesh was based on previous experience with similar rivers and designed to accurately represent the topography of the reach. A higher density of elements was used in topographically complex areas and where significant hydrodynamic variability occurs.

### 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 was developed from the 1-D model of the reach between RM187 to RM195.25 that was used for the previous study to estimate to estimate the water-surface profiles for flows between 20,000 and 200,000 cfs (**Figure 3.3**) (Harvey et al., 2004). Results from the 1-D model indicated 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.

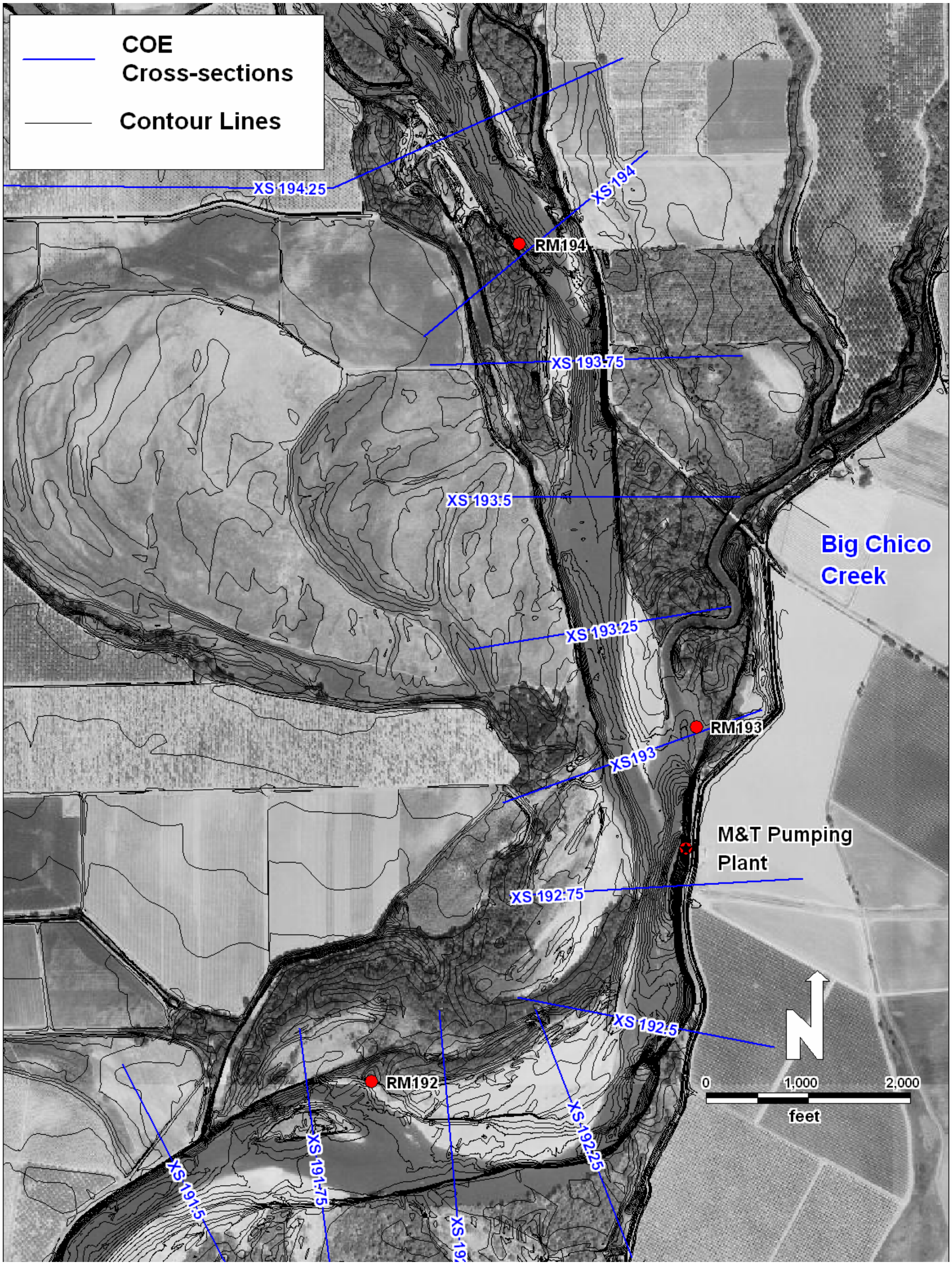


Figure 3.1. Map showing topographic mapping used for the development of the 2-D models of the M&T pumping plant reach.

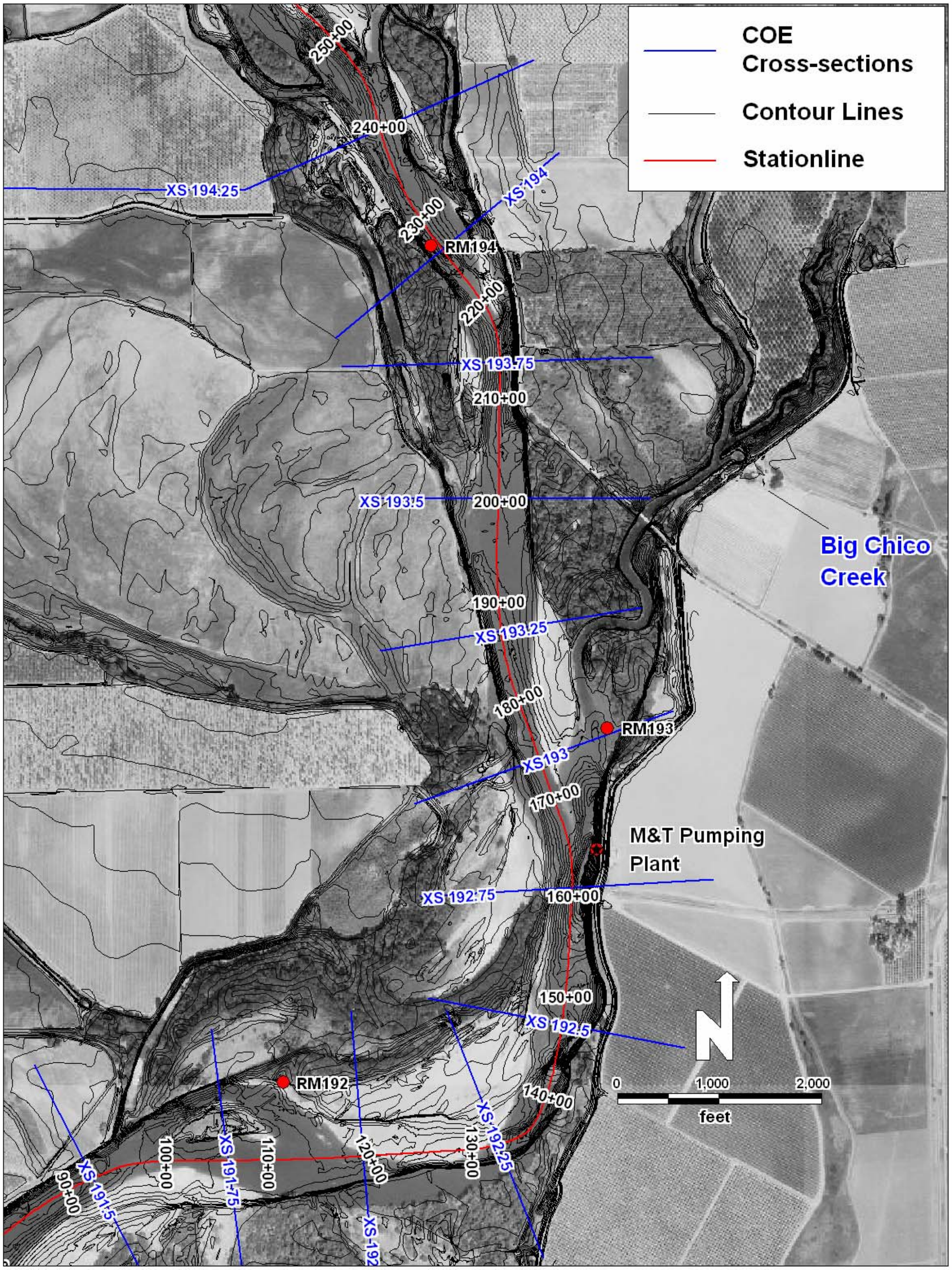


Figure 3.2. Finite element mesh for the 1996 geometry of the M&T pumping plant reach.

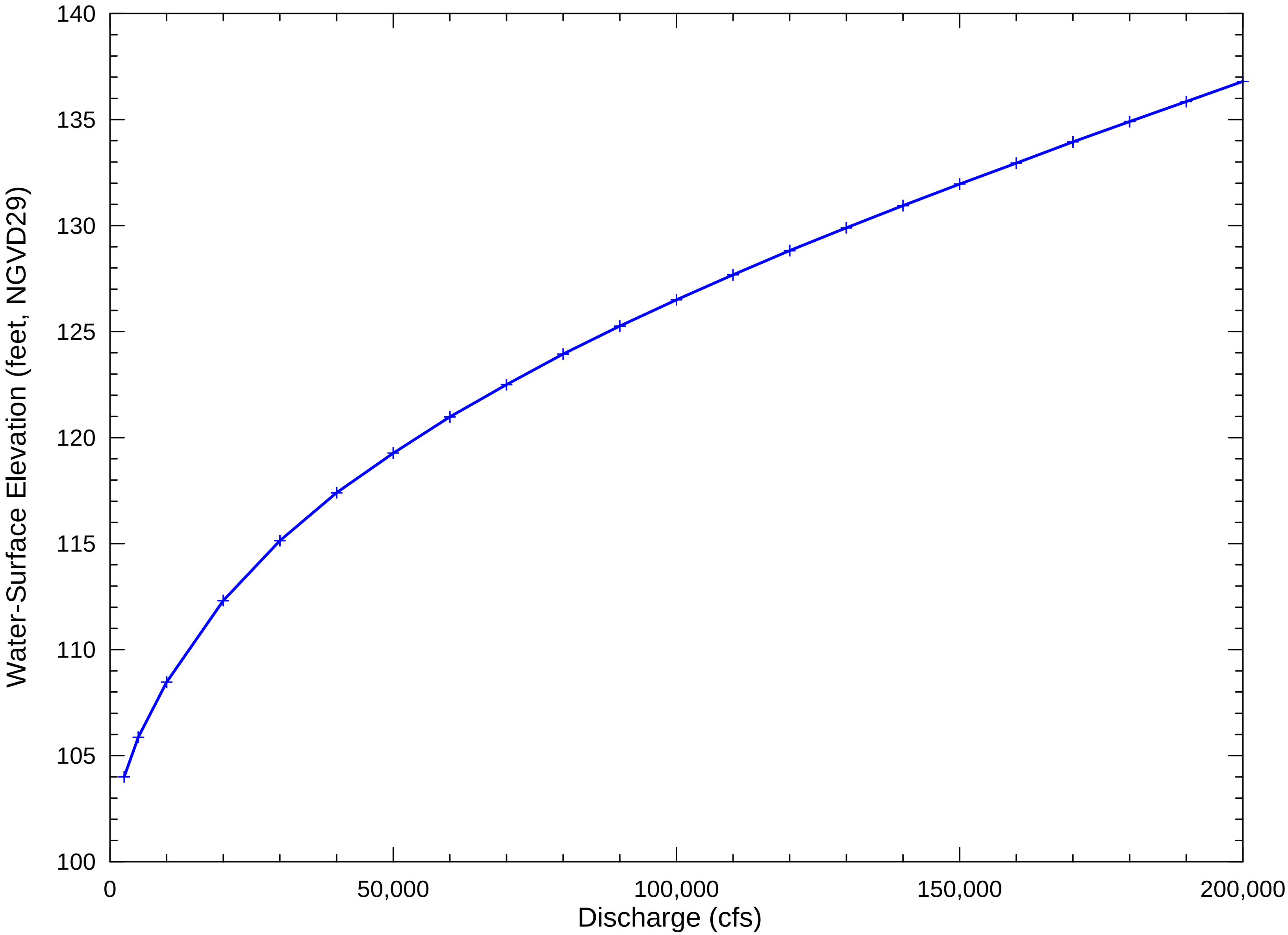


Figure 3.3. Rating curve computed by the HEC-RAS 1-D model at RM 191.5 that is used at the starting water-surface elevation for the 2-D model.

### 3.1.3. Material Properties and Model Validation

The RMA2 hydrodynamic 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 finite element mesh. Three different material types were used in the models to represent the main channel, the overbanks, and side channels, respectively (**Figure 3.4**). A main-channel Manning's *n*-value of 0.023 was established based on field observations, similar experience with other rivers, and standard references (Chow, 1959; Barnes, 1967; Hicks and Mason, 1991; Julien, 1995), and validated by comparing computed water-surface elevations at 90,000 cfs from the 2-D model with results from the previously applied 1-D model that originated from the model developed by the USACE for the Sacramento and San Joaquin River Basins Comprehensive Study (USACE, 1997b). The 1-D model was calibrated to field measurements and is based on the same topography used to develop the 2-D 1996 model. Manning's *n*-values of 0.12 were used for the overbanks to reflect the roughness of the vegetated in these areas, based on aerial photographs and field observations, and the *n*-values of 0.06 were used for the side channels to reflect the reduced flow depths 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 prediction of the flow patterns due to channel curvature. Agreement between the validated 1-D model and the 2-D model at the approximate bankfull discharge of 90,000 cfs using these input values was excellent (**Figure 3.5**).

### 3.1.4. Effects of Big Chico Creek

An analysis was also conducted to evaluate the relative effect of flows from Big Chico Creek on flow patterns and transport rates in the study reach. As discussed in Chapter 2, the available flow records indicate that Big Chico Creek discharges that have occurred when the discharge in the Sacramento River was near bankfull ranged from a few hundred cfs to as much as 7,500 cfs, and were most often in the range of 1,000 to 1,500 cfs (Figure 2.5). A sensitivity analysis was conducted by running 2-D model at 90,000 cfs with and without a 1,500 cfs inflow from Big Chico Creek. The results of the analysis indicate that Big Chico Creek flows have very little effect on the water-surface profile through the study reach, or the computed depths and velocities in the vicinity of the M&T intake. Based on these results, Big Chico Creek inflows were neglected in the remainder of the modeling.

### 3.1.5. 1996 2-Dimensional Modeling Results

To evaluate the hydraulic and sediment-transport conditions in the reach, all of the models for this study, including the 1996, model were run with the approximate bankfull discharge of 90,000 cfs, with the boundaries of the 2-D mesh corresponding approximately to the area of inundation at this discharge. 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 water-surface elevation and hydraulic conditions above bankfull.

Model results indicate that maximum main channel velocities range from 8 to 12 fps and maximum channel depths range from 17 to 40 feet along the reach at 90,000 cfs (**Figures 3.6 and 3.7**). The peak 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. At 90,000 cfs, the velocity over the gravel bar is approximately 6 to 7 fps and the flow depth is approximately 10 feet. The

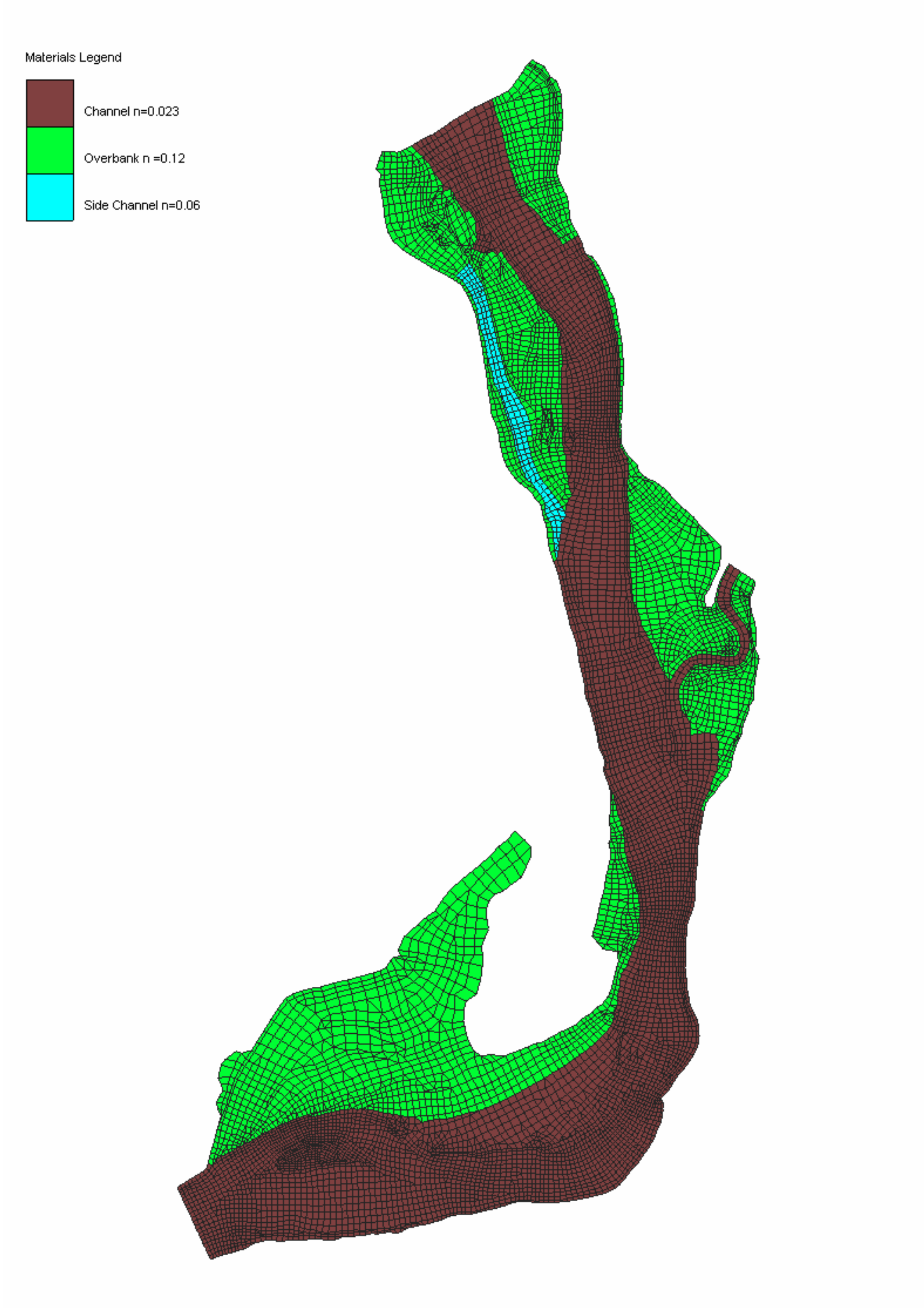


Figure 3.4. Finite element mesh of the M&T pumping plant reach showing the distribution of element material types.

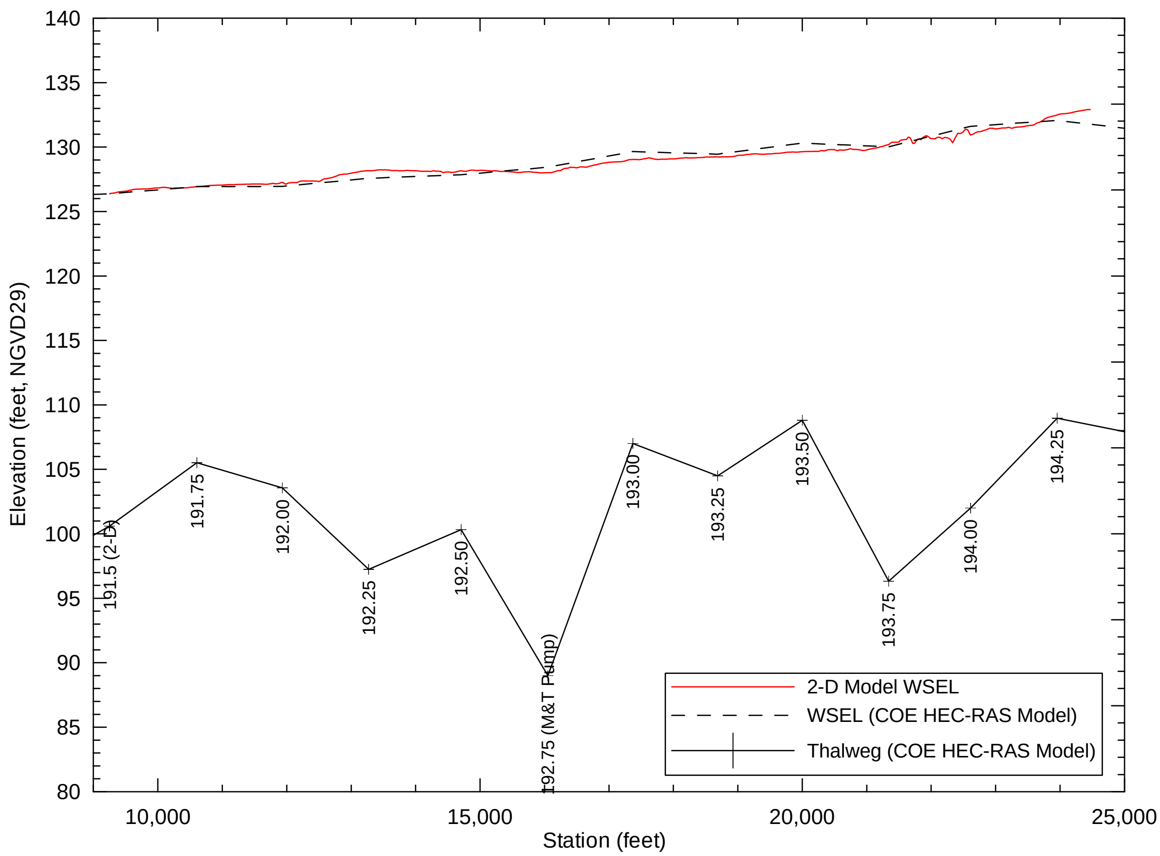


Figure 3.5. Comparison of water-surface elevations computed from the 1-D and 2-D models.

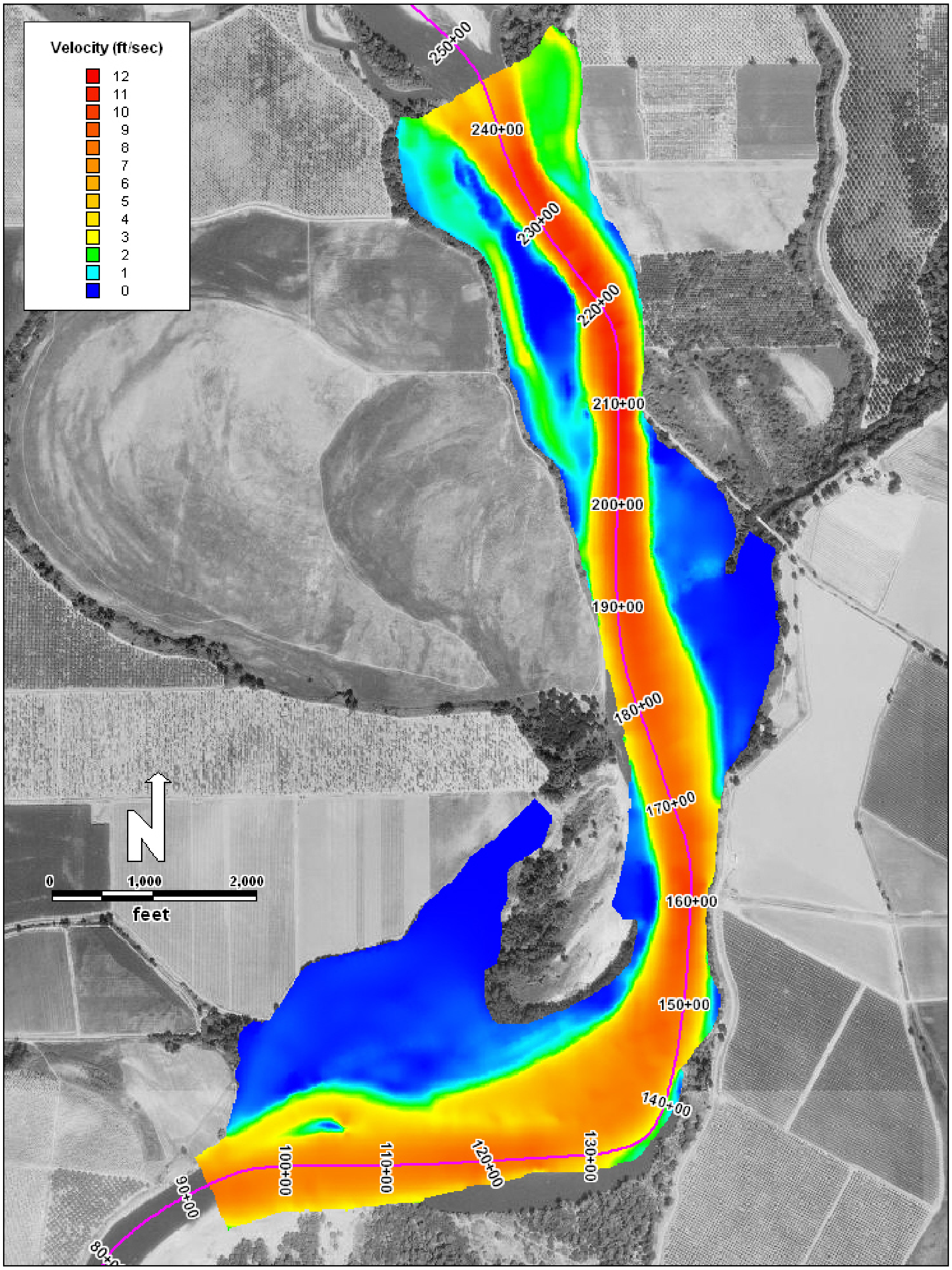


Figure 3.6. Velocity distribution predicted by the 1996 conditions model at a discharge of 90,000 cfs.

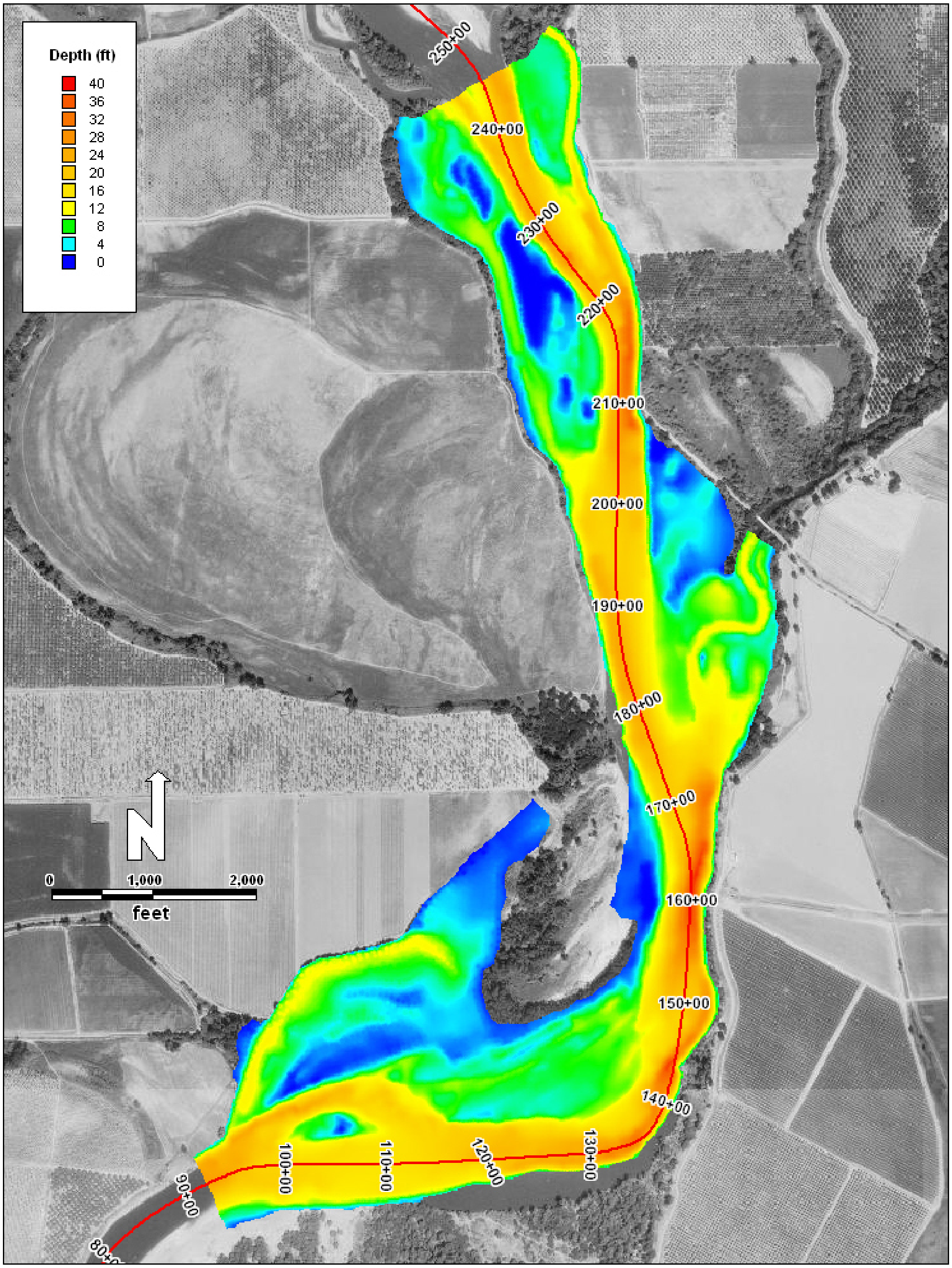


Figure 3.7. Distribution of flow depths predicted by the 1996 conditions model at a discharge of 90,000 cfs.

orientation of the main flow channel upstream of 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.

### 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 model geometries. The analysis was performed by comparing the critical shear stress ($\tau_C$, shear stress required to initiate motion) for the median particle size with the bed shear stress ($\tau$) at the approximate bankfull discharge of 90,000 cfs.

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

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

where
$\tau_C$ = critical shear stress,
$\tau_{*C}$ = dimensionless shear stress,
$\Upsilon_S$ = unit weight of sediment (~165 lb/ft$^3$),
$\Upsilon$ = unit weight of water (62.4 lb/ft$^3$), and
$D_{50}$ = median particle size.

When the critical shear stress for the median particle size is exceeded, the bed is mobilized and all sizes up to 5 times the median size are capable of being transported by the flow (Parker et al., 1982; Andrews, 1984). Reported values of $\tau_{*C}$ 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.

The bed shear stress due to grain resistance ($\tau'$) is normally used in the incipient motion analysis because it is a better descriptor of near-bed hydraulic energy in gravel-bed streams than the more commonly used total shear stress. This is 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 ($\tau'$) is computed from the following equation:

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

where:
$\gamma$ = 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) is obtained by rearranging the Manning's equation, as follows:

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

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 ($\tau_*$), which is the ratio of grain shear stress ($\tau'$) to the critical shear stress ($\tau_C$) 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.

The normalized grain shear stress (NGS) was estimated at each node in the finite element model using a dimensionless critical shear stress of 0.03 and the average median ($D_{50}$) size of the bed material at the site of 21.5 mm (**Figure 3.8**). Normalized grain shear stresses in the main channel from the analysis range from 2.2 to 4.0 at 90,000 cfs, indicating that significant sediment-transport is occurring over the entire reach. The highest NGS values occur in the reach upstream from the head of the bar (Sta 190+00) and the lowest values occur at the downstream end of the bar in the vicinity of the M&T intake.

### 3.1.7. Sediment-transport Calculations

The variation in sediment-transport capacity along the reach was quantified by estimating the transport capacity at nine cross sections that were developed from the 2-D mesh using the continuity line option in SMS (Figure 3.8, Cross Section XS0 through XS7). The sediment-transport capacity across each element of the continuity line was computed using the Parker

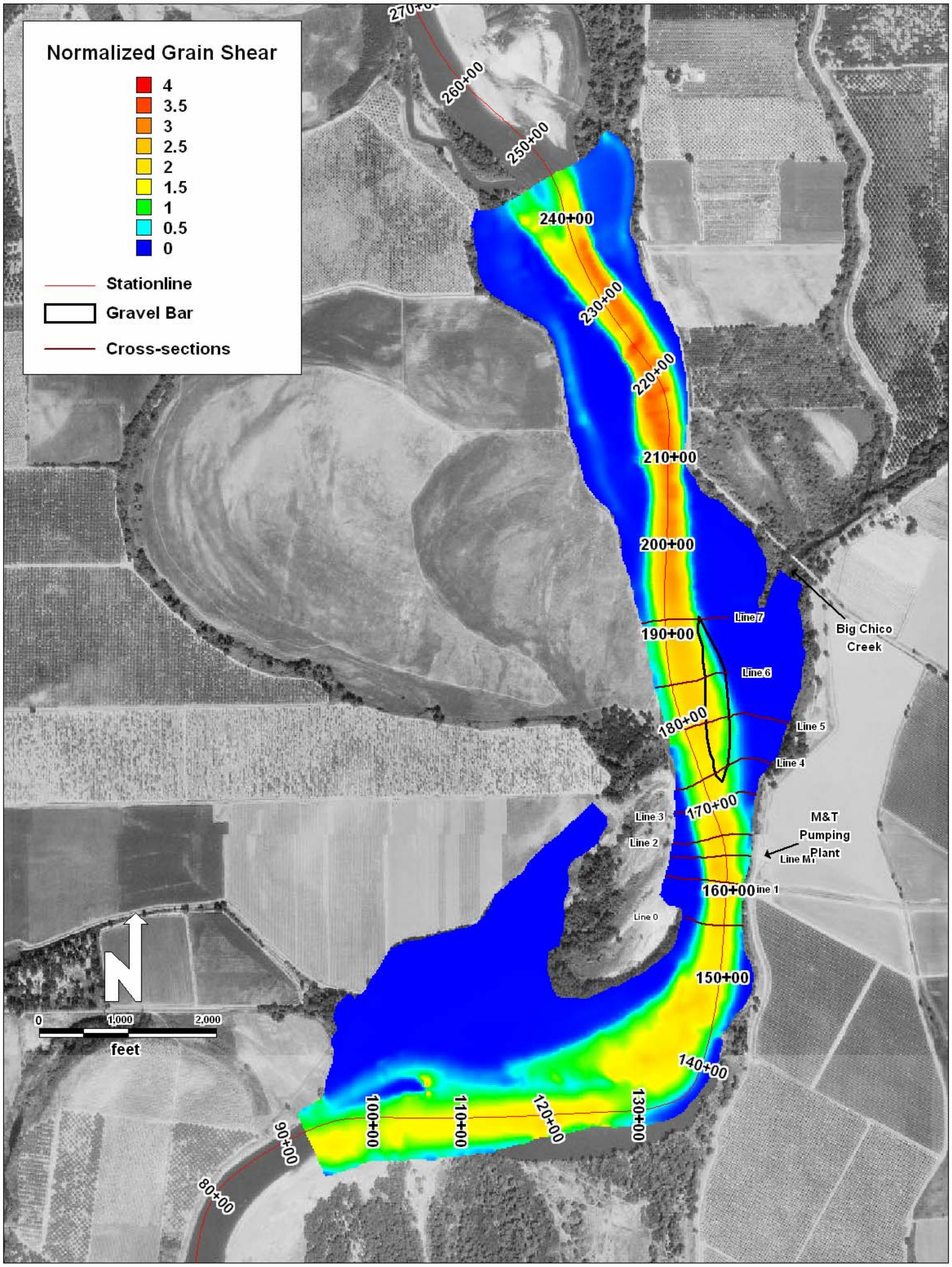


Figure 3.8. Distribution of normalized grain shear predicted by the 1996 conditions model at 90,000 cfs. Also shown are the cross section lines used for the sediment-transport calculations.

(1990) surface-gradation-based bed load equation with the corresponding hydraulic conditions at the nodes and discharges across each element of the cross section (**Figure 3.9**). The results indicate that the transport capacity in the vicinity of XS3 and XS4, which are located at the downstream end of the bar and upstream of the M&T pump is very low compared to both the up- and downstream reaches. These conditions strongly favor continued development of the bar.

## 3.2. 2003 Baseline Conditions Model

Comparison of the available aerial photography of the study reach indicates that the west bank eroded up to 270 to the west over an approximately 0.5 mile reach immediately upstream from the M&T intake between 1996 and 2003 (**Figure 3.10**). The aerial photographs also indicate that the configuration of the gravel bar and other bed features changes during this period. Because the reach was not remapped in 2003, the topography for the 2003 conditions model was estimated by making appropriate modifications to the 1996 mapping to reflect the changes that are visible in the aerial photography (**Figure 3.11**), and elevations from the post-dredging survey that was conducted by NorthStar in March, 2003. These changes included realigning the west bank to correspond to the 2003 bankline. Since the overall profile of the river does not appear to have changed between 1996 and 2003, the toe of the realigned bank in the 2003 model was held at the same elevation as the toe of the bank in the 1996 mapping, which roughly corresponded to the surveyed water-surface elevation. The bank slope was also held the same as that in the 1996 mapping. The subaqueous contours in the vicinity of the modified bank were contoured so that the thalweg in the main channel was located along the right approximately one-third of the channel width to account for the upstream flow orientation and deflection of flow off the mid-channel bar toward the right bank. The height of the mid-channel bar that is visible in the 2003 aerial photograph (Figures 3.10 and 3.11) was set 2 feet above the water-surface elevation, based on field observations at a discharge similar to that at the time of the photo, and the subaqueous contours between the mid-channel bar and the primary gravel bar were modified to reflect elevations from the final excavation contours shown on the NorthStar plans. The size and topography of the subaerial portion of the gravel bar were also modified to match the NorthStar plans.

### 3.2.1. 2003 Baseline Conditions Incipient Motion and Sediment-transport Results

The model results for 2003 baseline conditions indicate that the overall range of velocities, depths and normalized shear stresses are similar to that under 1996 conditions, but the distribution is quite different in the reach along and downstream from the gravel bar (**Figure 3.12**, Figure 3.8). From the head of the bar at about Sta 190+00 to the upstream end of the model, the shear stresses are similar between the two models. Along and downstream from the gravel bar (Sta 160+00 to Sta 190+00), however, the predicted shear stresses are generally higher and more uniform in the 1996 model than in the 2003 model. In fact, shear stresses well above the critical shear are indicated in the main channel in the entire reach along the bar and for about 2000 feet downstream, including the vicinity of the M&T intake. For 2003 conditions, the shear stress between about Sta 160+00, just downstream from the intake, and Sta 172+00, near the downstream end of the bar is at or only slightly above critical. The indicated changes in shear stress result in a significant reduction in bed material transport capacity throughout the reach from about Sta 160+00, about 300 feet downstream from the M&T intake to the upper end of the bar at about Sta 185+00 (Figure 3.9). Since the transport capacity through this reach is relatively low, compared to the upstream supply, the reach is clearly depositional under these conditions, and the bar will likely continue to grow in the downstream direction.

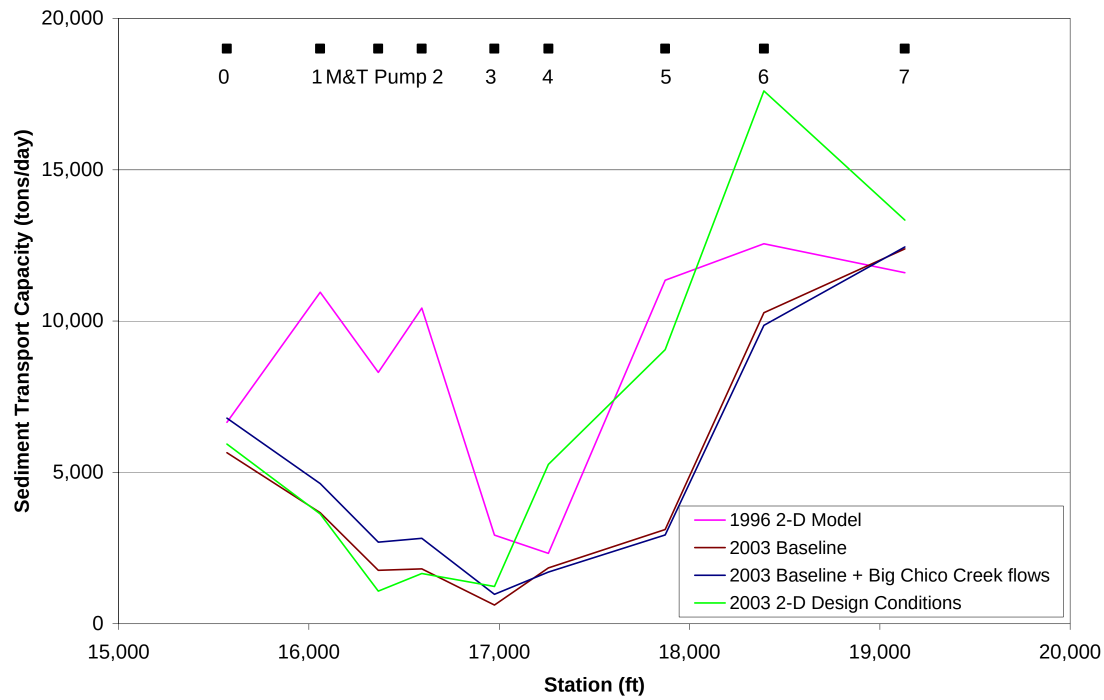


Figure 3.9. Estimated sediment-transport capacities at the nine cross sections identified in Figure 3.8 for 1996 conditions, 2003 baseline conditions, and 2003 conditions with the proposed dike field.

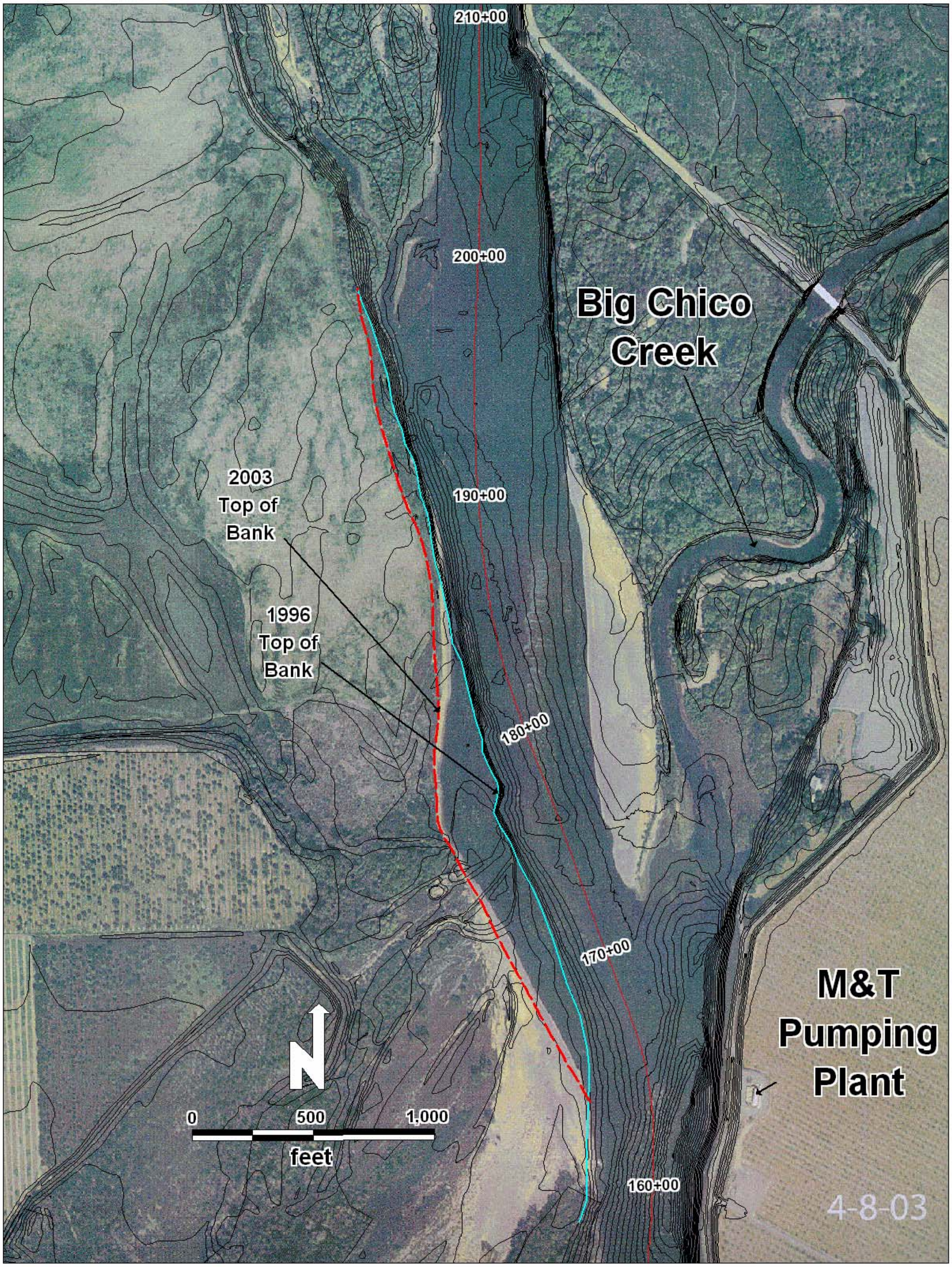


Figure 3.10. 1996 2-foot contour mapping overlain on the 2003 aerial photography.

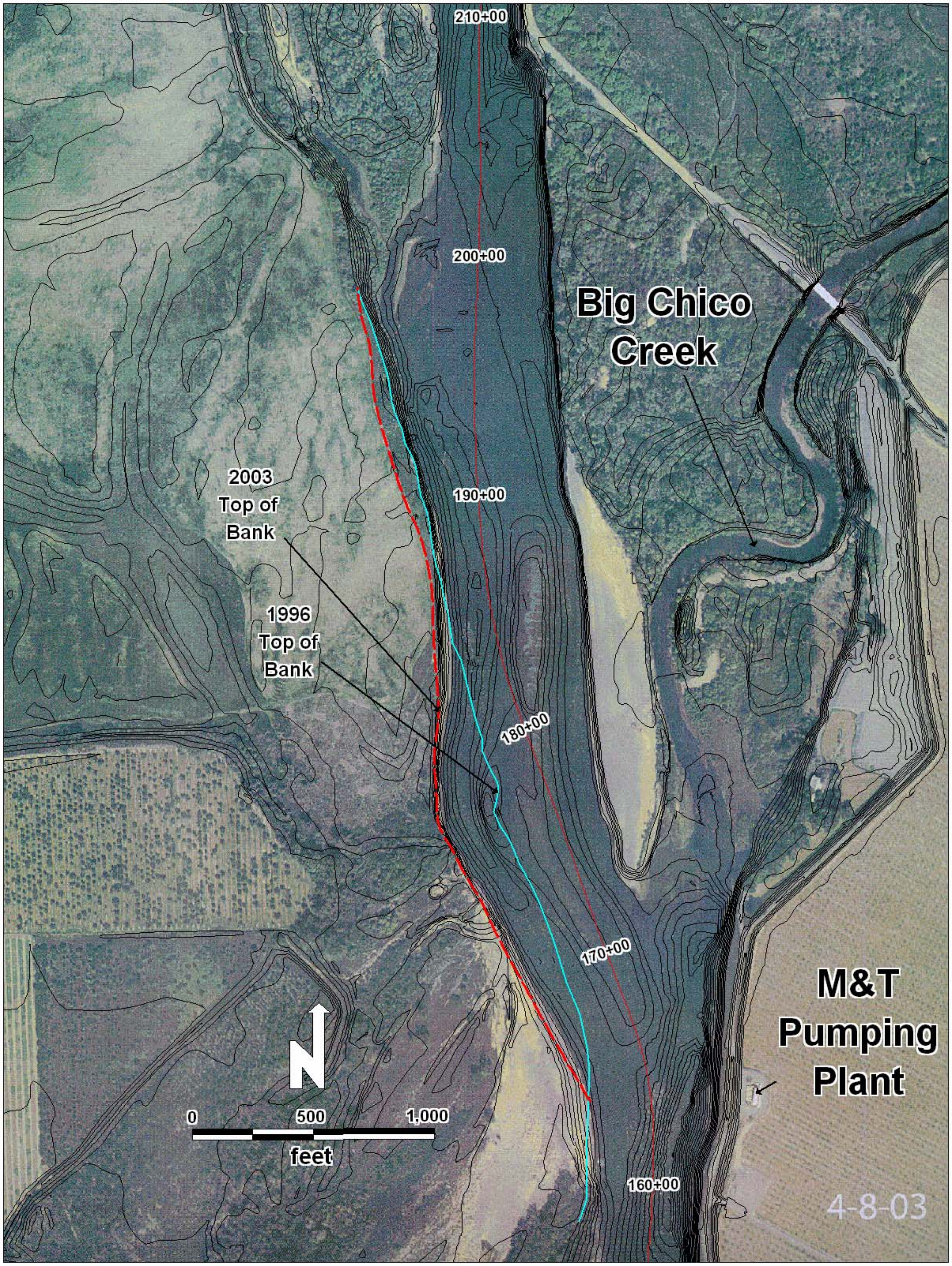


Figure 3.11. Aerial photograph and estimated contours for 2003 conditions.

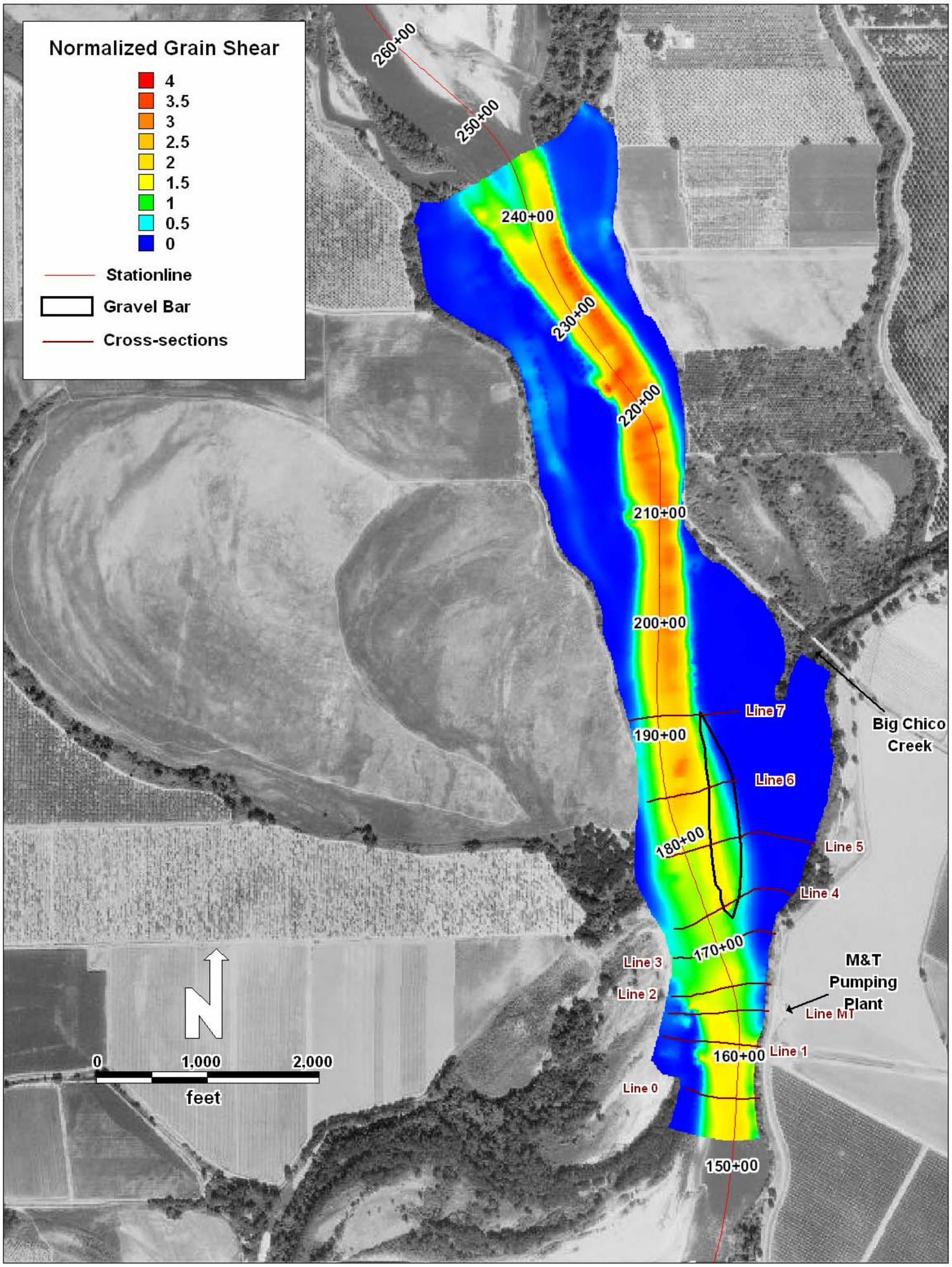


Figure 3.12. Normalized grain shear stress distribution predicted by the 2003 baseline conditions model at 90,000 cfs.

## 3.3. 2003 Model with Proposed Spur Dikes

The effects of the spur dike field that was proposed in the original Steering Committee Report (Harvey et al., 2004) were evaluated by modifying the topography in the 2003 model to reflect the presence of the dikes, and reanalyzing the hydraulic and sediment-transport conditions in the study reach (**Figure 3.13**). The proposed dike field consists of eight 150- to 200-foot long spur dikes that are spaced at about 380-foot intervals from just upstream from the M&T intake to about 500 feet upstream from the head of the gravel bar. The height of the most downstream dike (Dike 8) was set to two-thirds of the bank height where the dike intercepts the bank, and the profile slopes downward at 5 percent for the 150-foot length of the dike (**Figure 3.14**). The nose of the dike then slopes downward at 2H:1V to the channel bed. The top elevation of the other seven dikes was determined by running a line parallel to the 90,000 cfs water-surface profile upstream from the top of Dike 8 (**Figure 3.15**), to ensure the hydraulic performance of each dike would be similar over the range of flows. The proposed dikes have a topwidth of 5 feet, with 2H:1V sideslopes. With the proposed design, the top of the dikes are inundated at approximately 37,000 cfs, which is equaled or exceeded about 5.3 percent of the time, or approximately 20 days per year, on average, based on the available flow records.

### 3.3.1. 2003 Dike Scenario Incipient Motion and Sediment-transport Results

Model results for the proposed dike configuration indicate that the velocities and shear stresses along the gravel bar will increase significantly compared to 2003 baseline conditions (**Figures 3.16 and 3.17**) due primarily to narrowing of the channel. Under baseline conditions (Figure 3.12), normalized shear stresses along the right (west) edge of the gravel bar at the modeled discharge were generally between 1.0 and 1.6, and the upper surface of the bar was at or below 1.3. This indicates that the edge of the bar would be slightly above the threshold of motion and the top of the bar would be relatively stable under these conditions. With the proposed dike field, the normalized shear stresses along the right edge of the bar increase to 1.7 to 2.3 and the upper surface of the bar is typically in the range between 1.1 and 1.8, indicating that surface material on the bar would be mobilized under these conditions. The main channel shear stresses are also somewhat higher (normalized shear stresses between 1.0 and 1.5) from the downstream end of the bar through about Sta 170+00, about 800 feet upstream from the M&T intake. In the vicinity of the intake, however, the shear stresses are relatively low (1.0 or slightly above) under both scenarios.

The estimated bed material transport capacities along most of the length of the bar and for about 300 feet downstream are considerably higher (and similar to 1996 conditions) with the dikes in place than occurs under baseline conditions (Figure 3.9). In the reach between the downstream end of the model and about Sta 170+00, including the reach through the M&T intake, the transport capacities are relatively low, and very similar to those under baseline conditions.

The results obtained from this analysis indicate that the proposed spur dikes may be effective in preventing further growth of the gravel bar and continued lateral erosion of the west river bank away from the mouth of Big Chico Creek and the M&T intake. 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 M&T intake, and deposition problems may continue to occur.

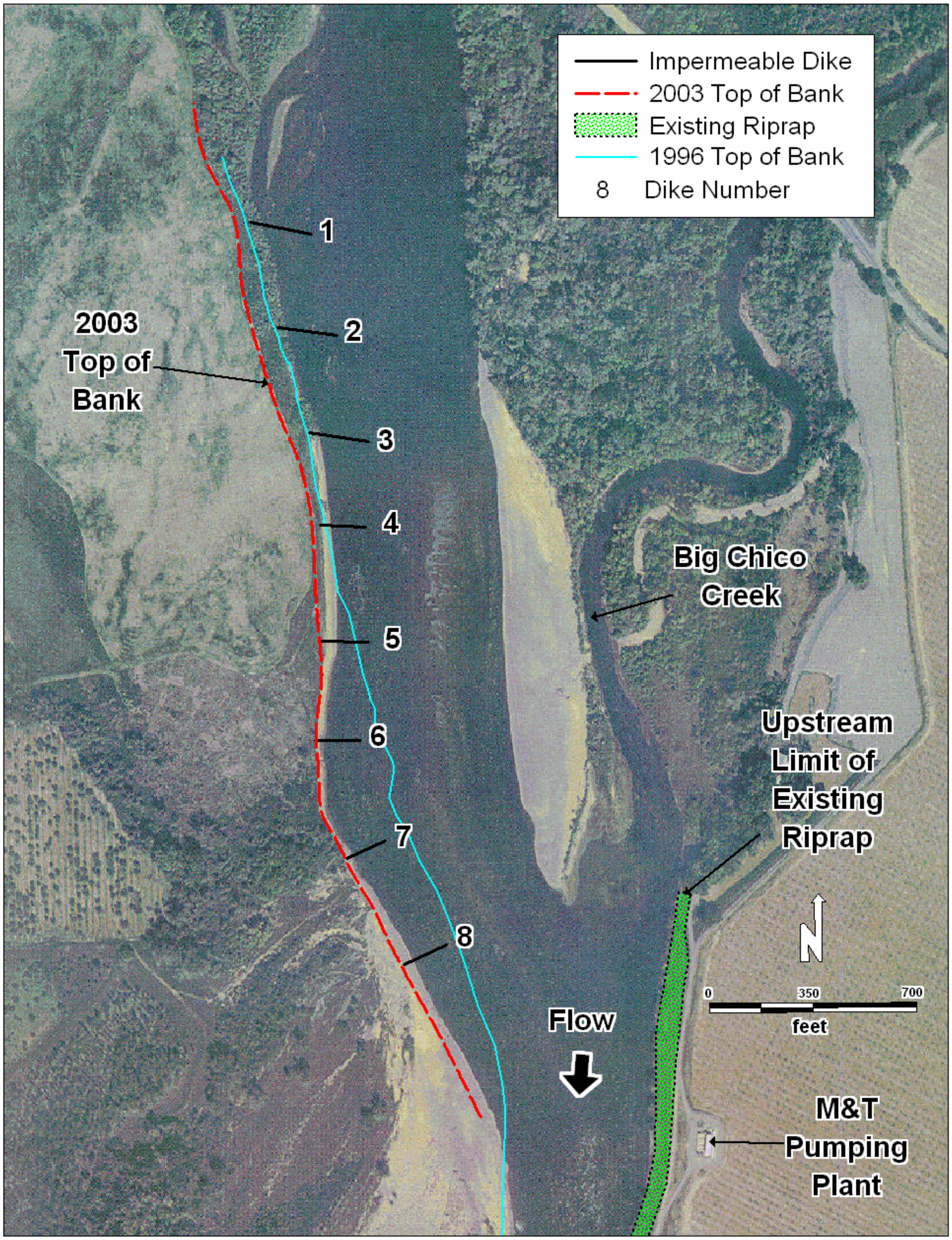


Figure 3.13. Preliminary dike design and layout, RM193R.

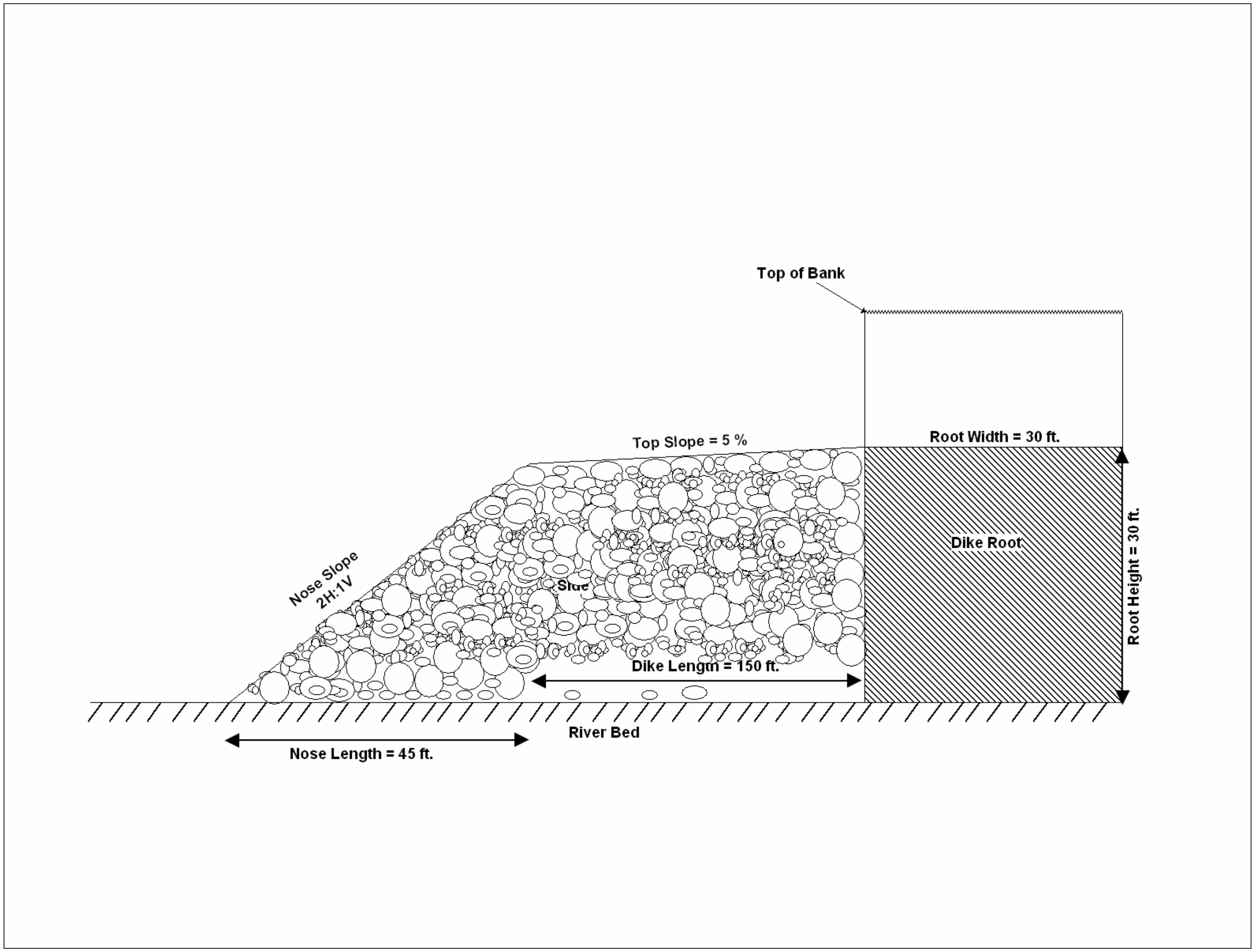


Figure 3.14.     Schematic profile of the proposed spur dikes.

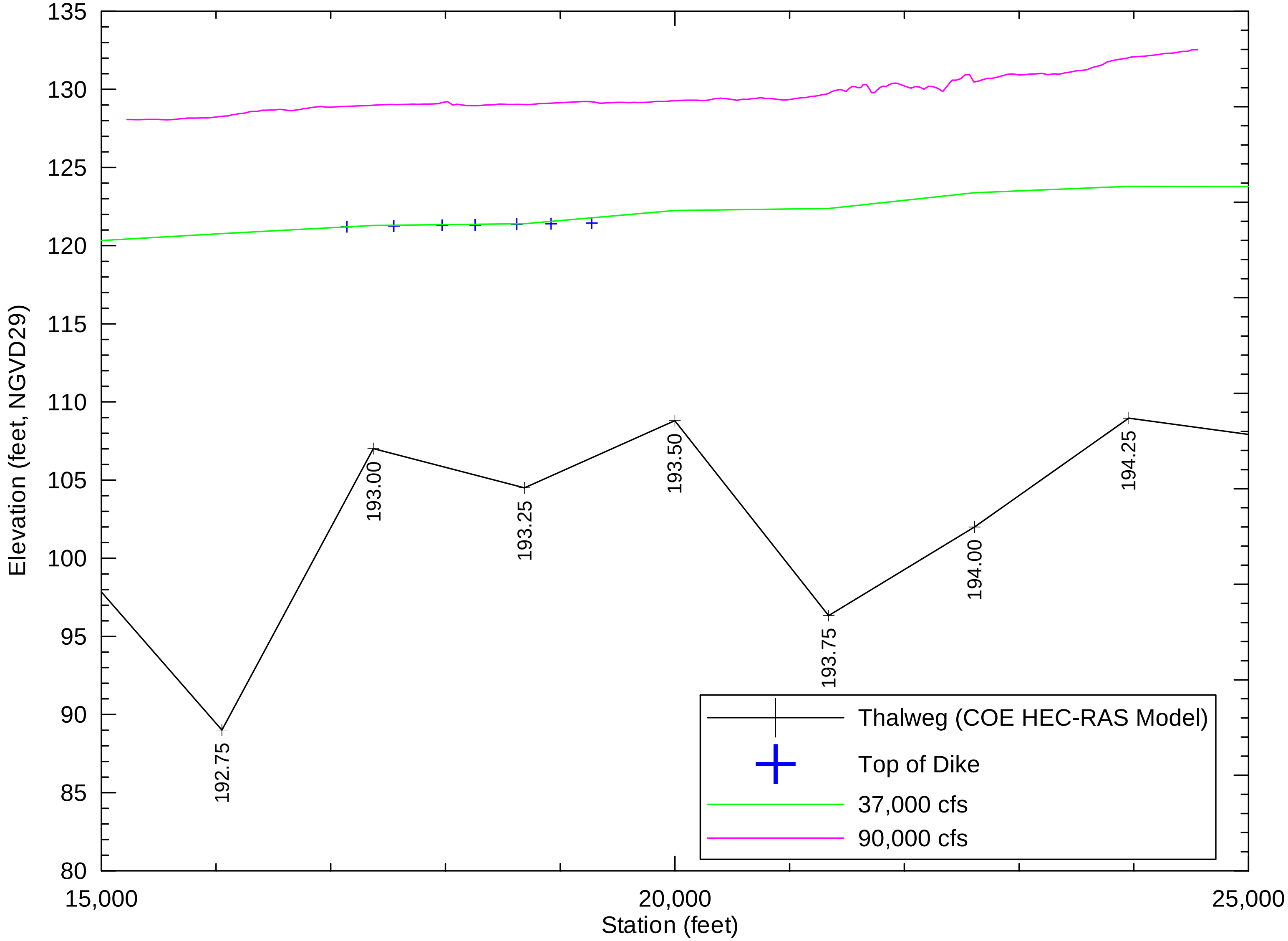


Figure 3.15. Estimated water-surface profiles at 37,000 and 90,000 cfs. Also shown are the top of the dikes in relation to the water-surface. profiles.

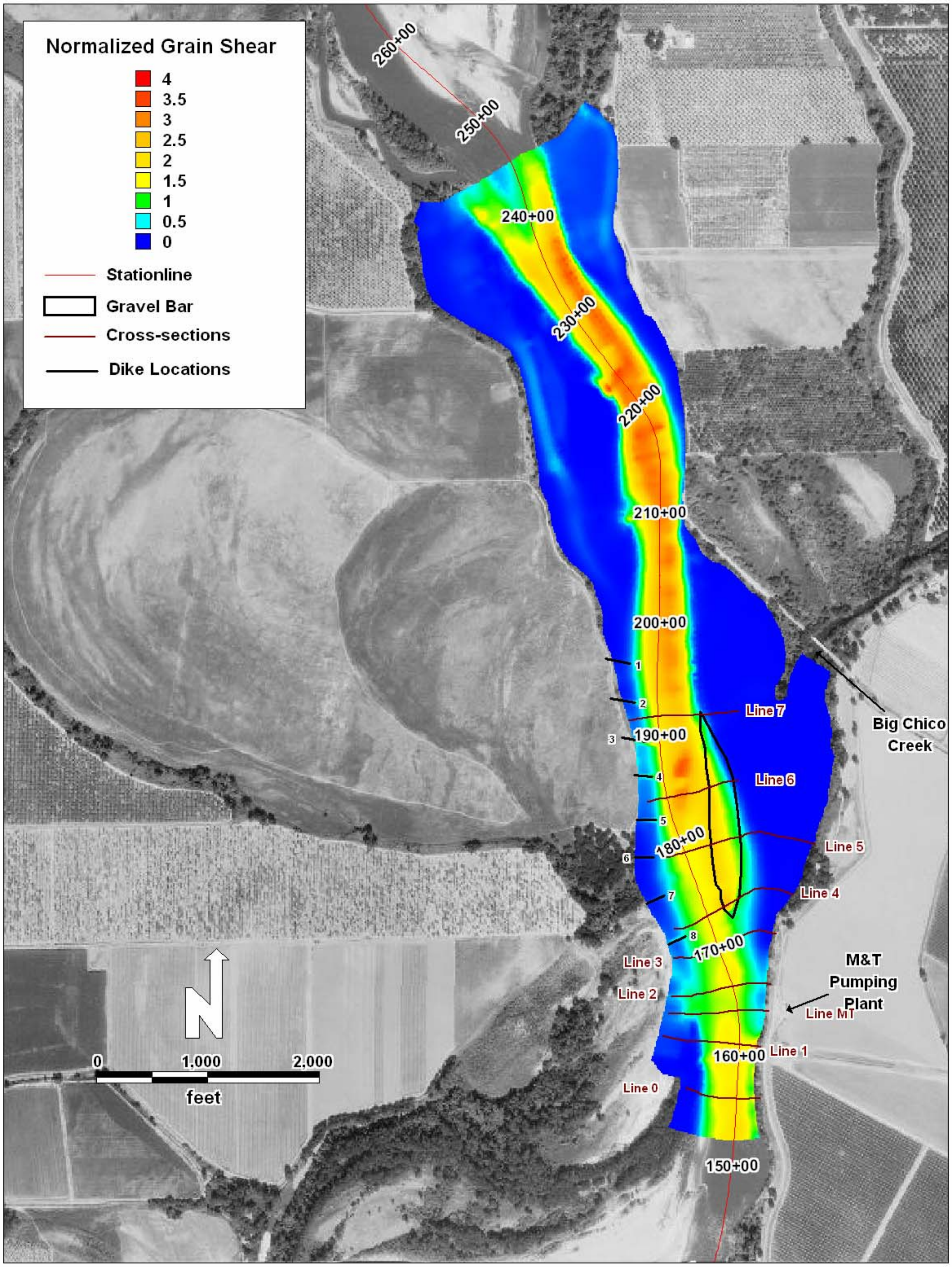


Figure 3.16. Normalized grain shear distribution at 90,000 cfs predicted by the 2003 model with the proposed dike field.

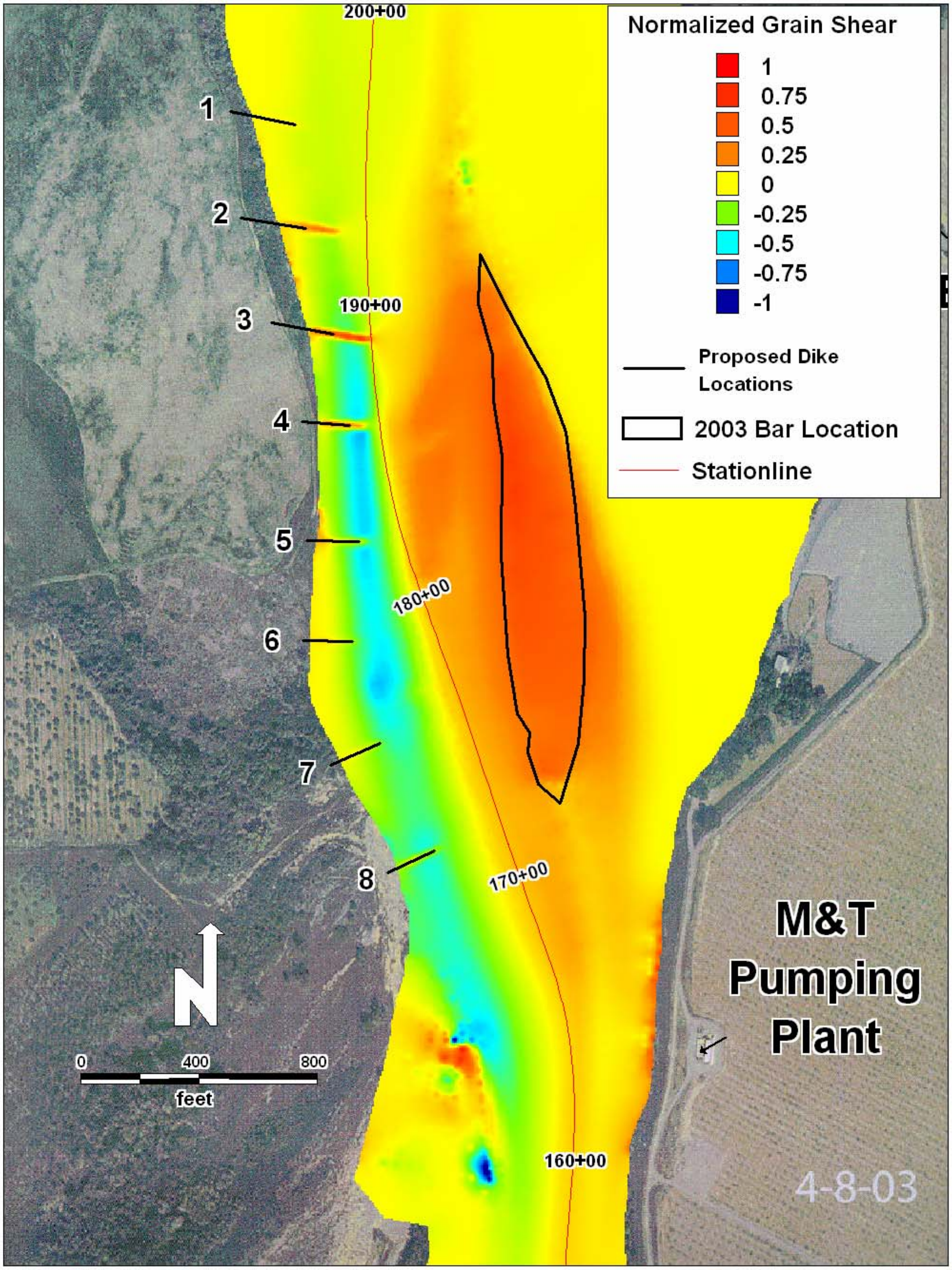


Figure 3.17. Difference in normalized grain shear between the 2003 models with- and without the proposed spur dikes.

## 3.4. Additional Model Scenarios

Because the results from above analyses indicate that the channel will remain depositional in the vicinity of the M&T intake, even with the proposed dike field, two additional scenarios were investigated to estimate sediment-transport characteristics prior to the erosion of the right bank and associated development of the gravel bar, and to assess the dike configuration that would necessary to restore the transport characteristics to these historical conditions.

### 3.4.1. Development of 2-D Model for 1979 Conditions (Scenario 4)

A model was developed to represent, to the extent possible given the available data, conditions that existed at the time of the 1979 aerial photography, prior to significant erosion of the right bank and development of the gravel bar (**Figures 3.18 and 3.19**). The 1979 photograph indicates that the channel through the reach that includes the M&T intake was considerably narrower than it is today, but a small point bar was present along the inside of the bend near the upstream end of the present gravel bar. The left (east) bank in the vicinity of the intake was, however, on the outside of a bend; thus, the main flow path (and probably the channel thalweg) were located along the left side of the channel in this area, directly over the present location of the intake. Since 1979, the bend opposite the point bar has migrated downstream, effectively changing the inflection of the bend in the vicinity of the intake to the opposite side of the channel and allowing the gravel bar to grow larger and migrate in the downstream direction across the mouth of Big Chico Creek.

Since mapping that represents 1979 conditions is not available, the topography for the 1979 conditions model was approximated by making the following changes to the 1996 conditions model, based on features that are visible in the 1979 aerial photograph:

1. At the upstream end of the reach, the left bank was re-contoured to match the 1979 bankline.

2. The point bar on the inside of the bend (left side of the channel) near the top of the photograph in Figure 3.18 was created.

3. The confluence of Big Chico Creek was modified to its location in the 1979 aerial photograph.

4. The alignment of the right bank from Sta 160+00 to Sta 190+00 was altered to match the photography.

5. The channel bed elevations were adjusted to reflect a long, relatively uniform reach beginning at Sta 200+00, and transitioned into a subaqueous riffle near Sta 175+00. The thalweg of the channel was located to the right of the centerline as would be expected on the outside of a bend and the bed elevations were created with similar thalweg elevations to those observed in the 1996 model.

While the resulting model is only a rough approximation of the actual topography that existed in 1979, it is believed to be sufficiently close for purposes of comparing hydraulic and bed material transport conditions through the reach with the current conditions.

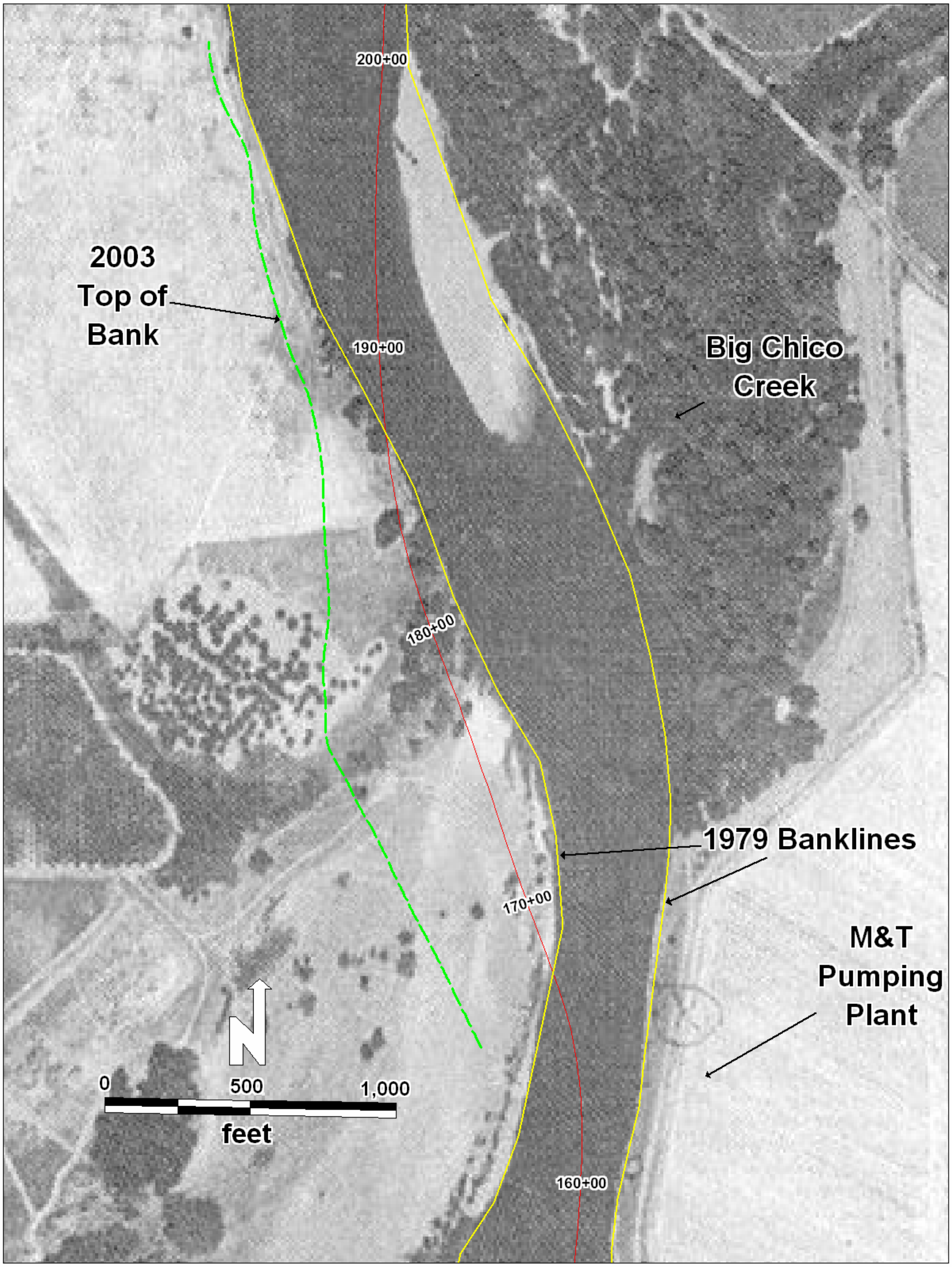


Figure 3.18. Aerial photograph taken in 1979 showing the approximate bank lines along with the banklines at the time of the 2003 photograph.

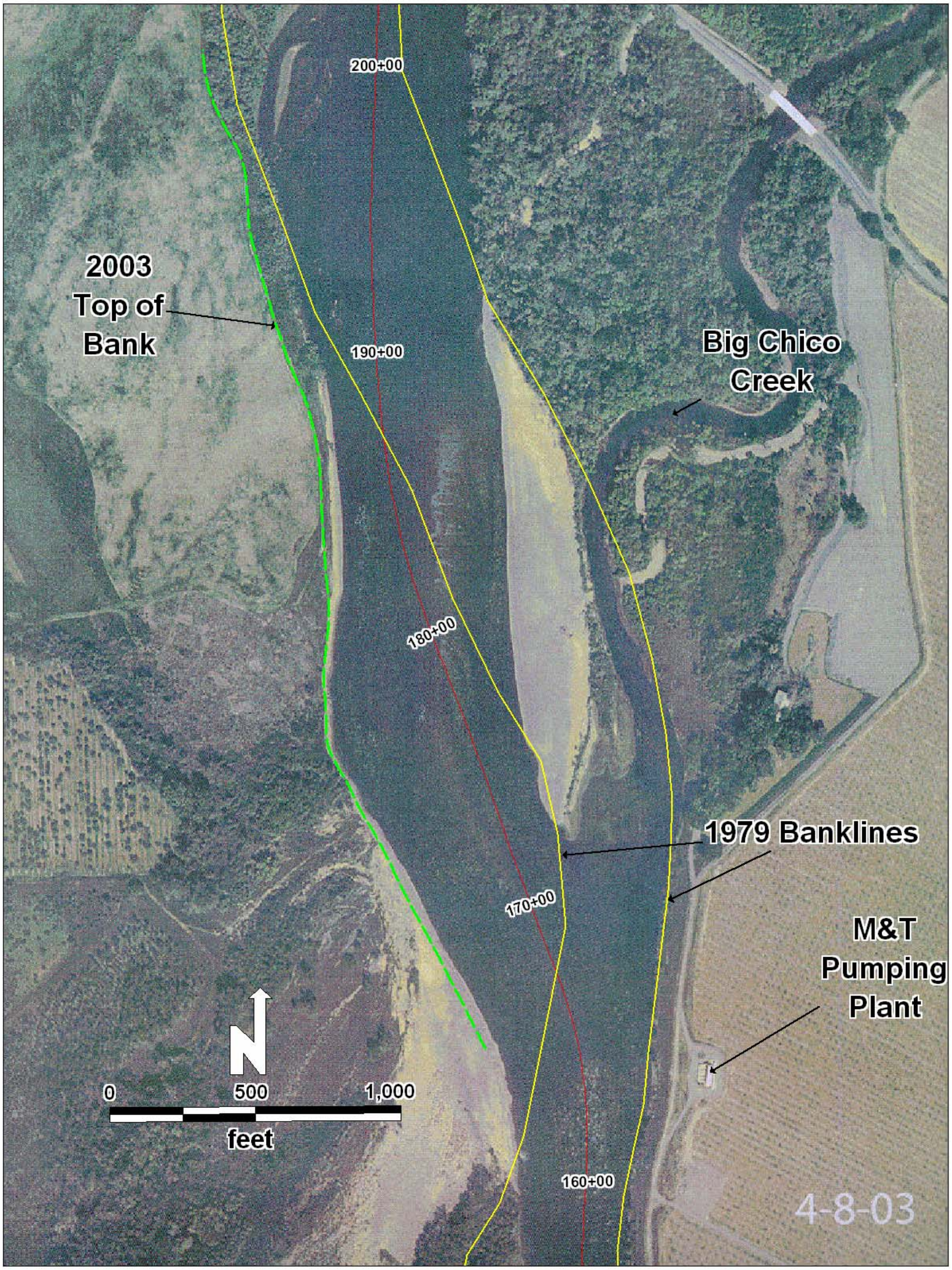


Figure 3.19. Aerial photograph taken in 2003 showing the 1979 banklines.

### 3.4.2. 1979 Model Sediment-transport Calculations

Results obtained from the 1979 conditions model indicate that the velocities and shear stresses are generally higher in the narrower channel that was present at the current location of the gravel bar and downstream through the M&T intake, with normalized grain shear values well above 1.5, and approaching 3.0 or higher, in some locations including the main channel in the vicinity of the intake (**Figure 3.20**). This is in contrast to results from the 2003 with- and without dike models where the normalized shear stresses are less than 1.0 over much of this reach (Figures 3.12 and 3.16). The estimated bed material transport capacities for the 1979 model are considerably higher and more uniform along the entire reach that includes the present location of the gravel bar and the intake than those in either the 1996 or 2003 conditions models (**Figure 3.21**). It is interesting to note, however, that the area in the vicinity of the point bar that is visible in the 1979 photograph has relatively low transport capacity compared to the remainder of the reach. The low transport capacity and location on the inside of the bend indicate conditions for continued growth and downstream migration of the bar.

## 3.5. Effect of Realigning of River Road Upstream from the Project Site

The curvature of the revetment on the left (east) bank along River Road between Sta 207+00 and Sta 220+00, upstream from the project site, appears to deflect the flow toward the right bank opposite the gravel bar. A preliminary analysis was performed to investigate the potential effect of realigning the revetment to eliminate this curvature by making appropriate changes to the topography in the 2003 baseline conditions (**Figure 3.22**).

Results of the 2-D model show that the realignment of the left bank changes the hydraulics in the vicinity of the realigned revetment, but the effects dissipate downstream of the road bend. The 2003 baseline model indicates that curvature of the revetment does not deflect flow towards the right bank and realignment of the bank does not change the orientation of the flow. In both models, the flow orientation is aligned parallel with the left bank upstream of the revetment and remains on a straight path until the upstream end of the bar.

Under 2003 baseline conditions, the contraction of the channel (Sta 210+00) created by the curvature, creates slight backwater conditions and a narrow flow conveyance width that extends downstream at this location. Downstream of the contraction, the main flow path is narrow with high velocities in the center and slower velocities on the margins. Realignment of the bank increases the overall channel width, increases the width of the main flow conveyance area, and decreases the backwater effect.

The channel widening decreases the locally high velocities at the contraction by up to 2 fps (**Figure 3.23).** Downstream of the contraction, the velocities toward the center of the channel decrease by approximately 0.3 fps at Sta 200+00 due to channel widening, while the velocities located toward the west of the station line increase by approximately 0.5 fps. This increase in velocity toward the right bank is due to dispersion of flows over a greater width of the channel. The dispersion of flow increases the velocities in the previously slower area adjacent to the main conveyance area.

At the center of the channel near the head of the bar (Sta 191+80), the velocities have decreased from approximately 9.7 to 9.5 fps (approximately a 2-percent decrease). There are no changes in velocities over the gravel bar or at the toe.

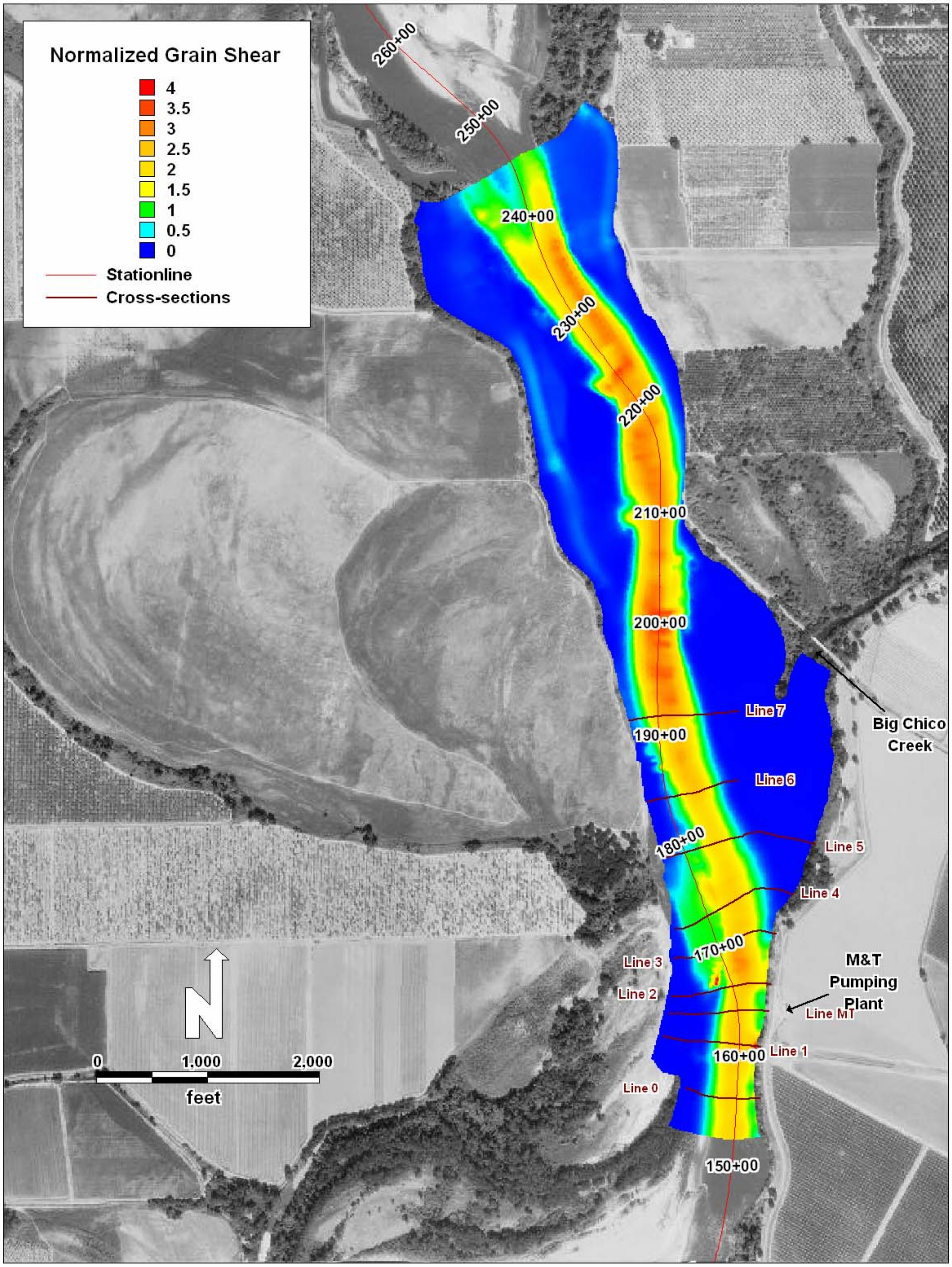


Figure 3.20. Distribution of normalized grain shear stress predicted by the 1979 conditions model.

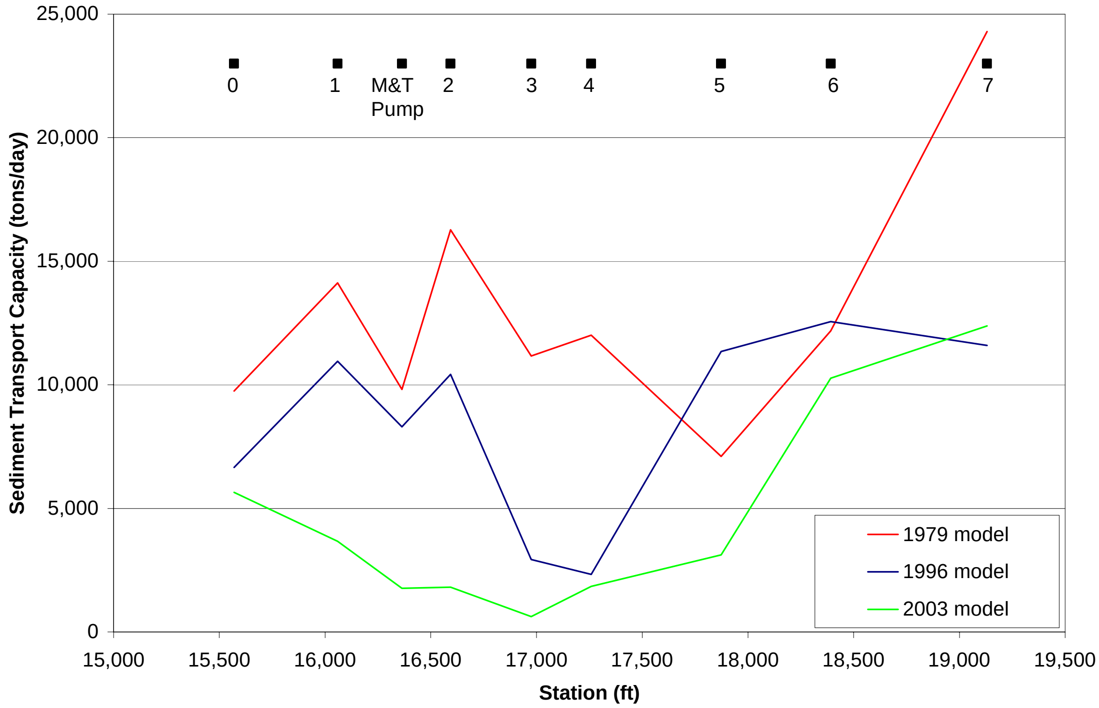


Figure 3.21. Bed material transport capacities estimated at the nine cross sections from the 1979, 1996 and 2003 baseline model results.


Figure 3.22. Aerial photograph taken in 2003 showing the current and modified alignment of River Road at Sta 210+00 (RM 194L).

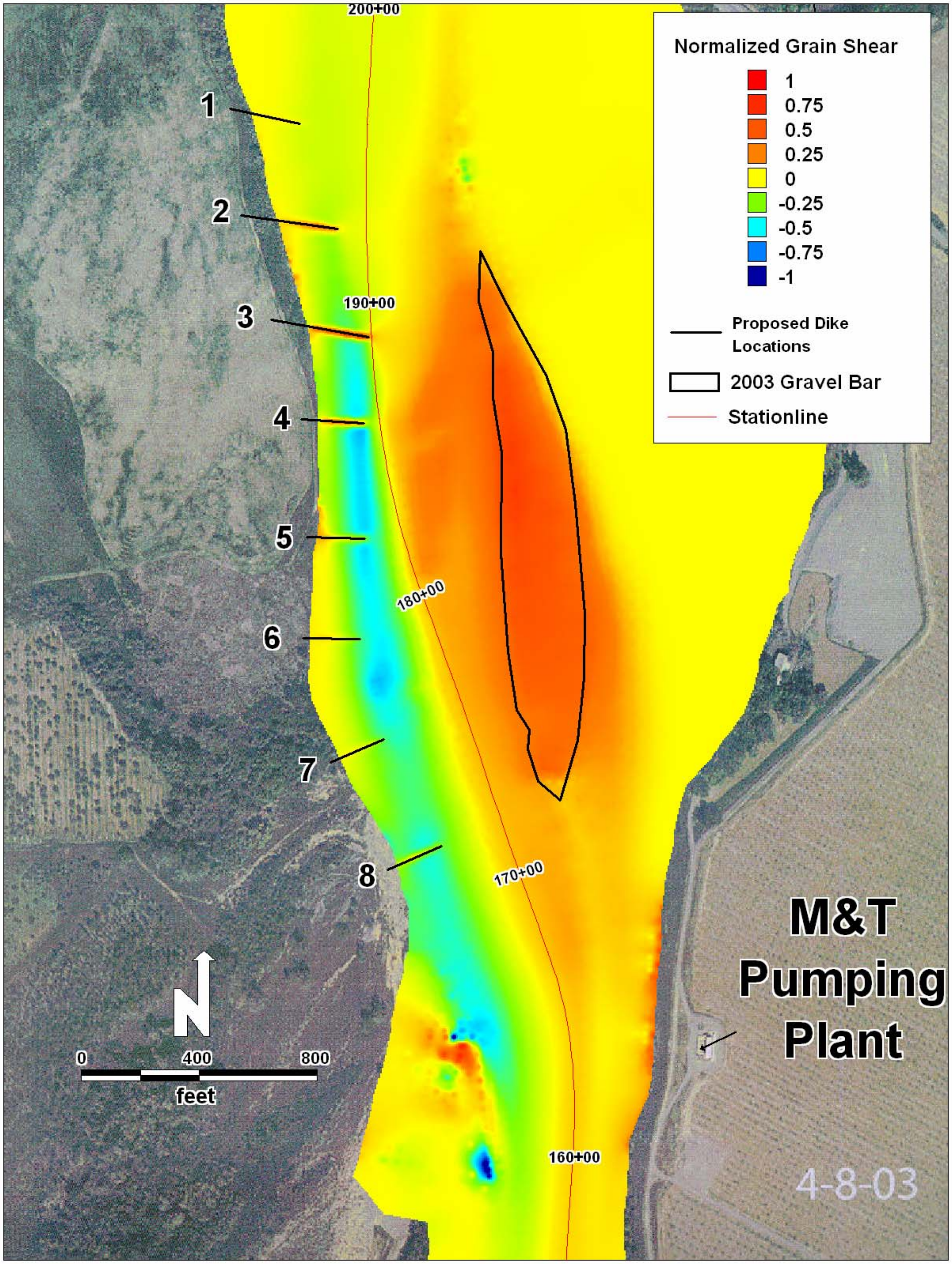


Figure 3.23. Difference in velocity between the realigned River Road model and 2003 baseline model.

Realignment of the bank decreases the normalized grain shear values at the center of the channel from approximately 2.9 to 2.4 at the realigned section (**Figure 3.24**). Downstream of the contraction (Sta 200+00), the normalized grain shear values toward the center of the channel decreased from approximately 2.9 to 2.7, and the area located toward the right of the station line increased from approximately 0.7 to 0.8. At the center of the channel near the head of the bar (Sta 191+80), the normalized grain shear values have decreased from approximately 2.4 to 2.3 (approximately a 2-percent decrease). There are no changes in normalized grain shear values over the gravel bar or at the toe. Similarly, the sediment-transport calculations showed no change as a result of realignment of the revetment compared to 2003 baseline conditions.

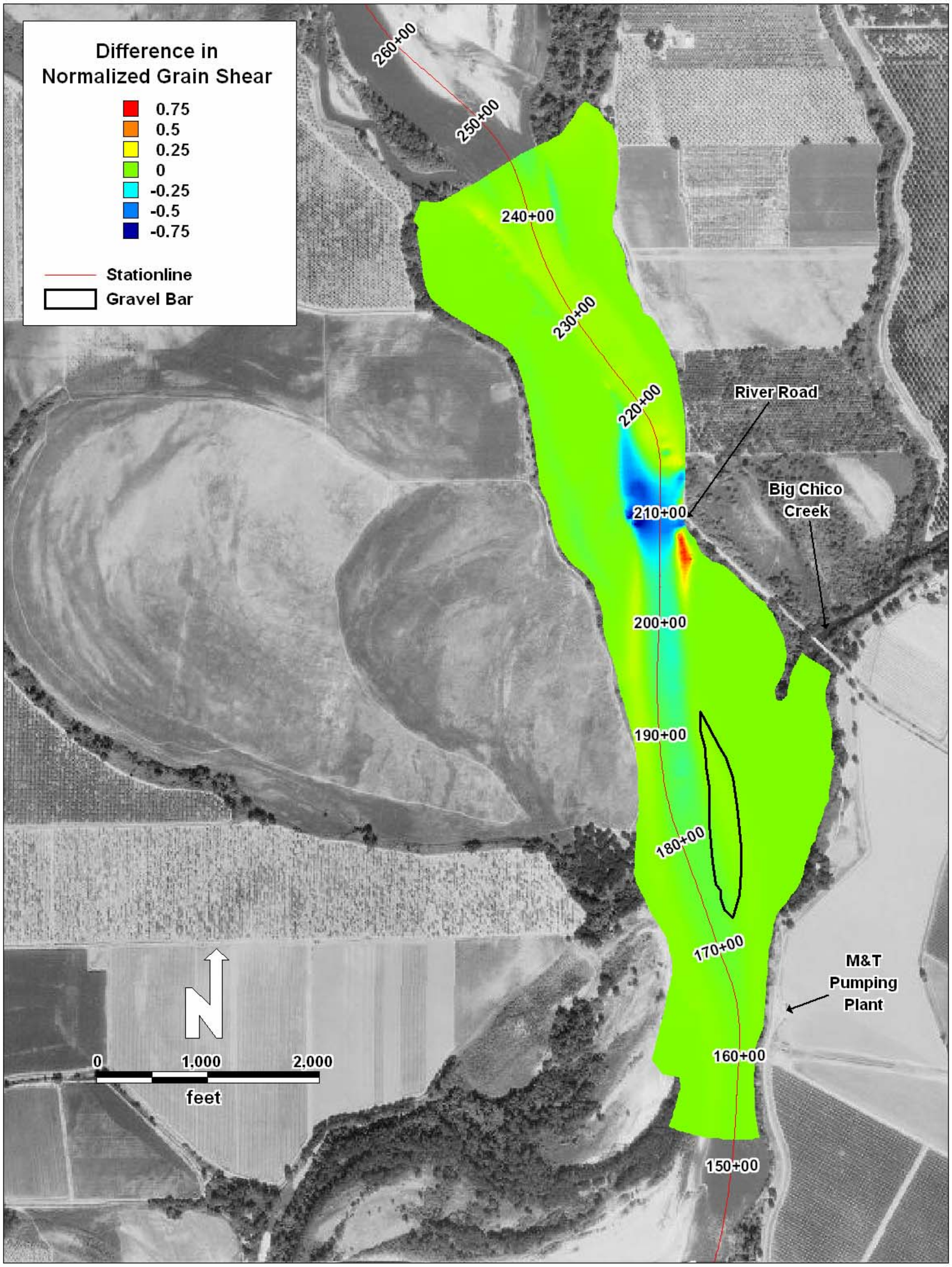


Figure 3.24. Difference in normalized grain shear values between the realigned River Road model and 2003 baseline model.

# 4. SUMMARY AND CONCLUSIONS

Based on recommendations in the original steering committee report (Harvey et al., 2004), a series of 2-dimensional (2-D) models were developed and applied to further evaluate the historic and present dynamics of the Sacramento River in the vicinity of the M&T pumping plant, and to evaluate the potential effects of a proposed spur dike field along the left (west) bank. The purpose of the dike field is to force the currently eroding riverbank to the east, preventing further lateral erosion and potentially increasing flow velocities sufficiently to prevent further deposition in the vicinity of the M&T intake. The proposed dike field included a series of eight, approximately 150- to 200-foot long spur dikes spaced at about 380-foot intervals along about 2,500 feet of the west bank opposite and upstream from the pump intake. Four different scenarios were considered in the modeling, as follows:

1. 1996 conditions, based on the available topographic and bathymetric mapping,

2. 2003 baseline conditions, developed by modifying the 1996 topography to reflect changes in channel alignment and other features that are visible in the 2003 aerial photograph and survey data collected by NorthStar in March, 2003 subsequent to dredging of about 189,000 $yd^3$ of material from the gravel bar,

3. 2003 conditions with the proposed spur dike field, and

4. 1979 conditions, based on approximate topography that reflects the channel alignment and other features that are visible in the 1979 aerial photograph.

The 1996 conditions model was used to establish and validate the model input parameters because it is based on the most complete topographic data, and because result from the model could be directly compared with the validated 1-dimensional model of the reach that was prepared by the USACE for the Sacramento/San Joaquin Comprehensive Study. The 2003 baseline conditions model was used to estimate changes in hydraulic and bed material transport conditions in the study reach associated with continued erosion of the west bank since the 1996 topography was prepared. The proposed dike field was then evaluated by superimposing the dikes on the estimated 2003 topography. The 1979 conditions model was used to evaluate differences in hydraulic and sediment-transport conditions when the channel was considerably narrower and the channel thalweg was most likely along the east bank at the M&T intake. A final model was developed and applied to evaluate the effects of realigning the revetment on the east bank along River Road upstream from the mouth of Big Chico Creek and the M&T intake. In all cases the models were run for a discharge of 90,000 cfs, which is the approximate bankfull capacity of the reach, to facilitate the analysis.

Results from the various models led to the following conclusions:

1. Maximum flow velocities in the main channel along the reach generally range from 8 to 12 fps, and maximum channel depths range from 17 feet to 40 feet at the modeled discharge of 90,000 cfs.

2. 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.

3. The maximum flow depth in the project reach occurs near the M&T pump intake. The velocities in this area are in the range of 7 fps.

4. At 90,000 cfs, the velocity over the gravel bar is approximately 6 to 7 fps and the flow depth is approximately 10 feet.

5. The orientation of the main flow channel upstream of the bar is directed slightly towards the right bank (west bank), but most of the flow is concentrated toward the center of the channel.

6. 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.

7. Normalized grain shear stresses in the main channel from the analysis range from 2.2 to 4.0 at 90,000 cfs, indicating that significant sediment-transport is occurring over the entire reach. The highest NGS values occur in the reach upstream from the head of the bar (Sta 190+00) and the lowest values occur at the downstream end of the bar in the vicinity of the M&T intake. (Note that NGS values of 1.0 indicate that the surface bed material is at a condition of incipient motion and NGS values of 1.5 or greater indicate significant bed material transport).

8. With the 1996 topography, bed material transport capacities through the reach were relatively uniform, except for the approximately 500 foot reach at and just downstream from the downstream end of the gravel bar (i.e., just upstream from the M&T intake).

9. Although the overall range of shear stresses predicted by the 2003 conditions model are similar to those from the 1996 model, the distribution is significantly less uniform, with significantly lower shear stresses and bed material transport capacities between the downstream end of the gravel bar and several hundred feet downstream from the intake.

10. Model results for the proposed dike configuration indicate that the velocities and shear stresses along the gravel bar will increase significantly compared to 2003 baseline conditions due primarily to narrowing of the channel. In fact, the right (west) side of the gravel bar appears to mobile, and the upper surface of the bar is at or slightly above incipient conditions at the modeled discharge; whereas, only the edge of the bar appears to be mobile under baseline conditions.

11. In the vicinity of the intake, however, the shear stresses and bed material transport capacities are relatively low (1.0 or slightly above) under both scenarios.

12. Based on the above results, the proposed dike field would prevent the river from migrating further to the west, and would likely prevent the gravel bar from continuing to enlarge. The low energy in the immediate vicinity of the M&T intake, however, indicates that this area will likely continue to be depositional, even in the presence of the dike field.

13. The 1979 conditions model confirms that the shear stresses and bed material transport capacities were considerably more uniform before the river began to migrate to the west in the project reach; however, a low energy area is evident at the location of the point bar that is visible in the 1979 aerial photograph near the upstream end of the present gravel bar. This indicates that conditions for formation of the gravel bar had begun by at least 1979. This process is consistent with typical river meander patterns, in which the bend

tend to develop laterally and migrate in the downstream direction through erosion on the outsides of the bends and point bar deposition on the insides of the bends.

14. The model indicate that realignment of the east bank revetment along River Road would probably not be effective in preventing continued growth of the gravel bar or depositional problems at the M&T intake.

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