

Channel and sediment transport dynamics near River Mile 193, Sacramento River

I GEOLOGIC AND GEOMORPHIC SETTING

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II MEANDER BEND AND GRAVEL BAR MIGRATION NEAR RIVER MILE 192.75 OF THE SACRAMENTO RIVER

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I GEOLOGIC AND GEOMORPHIC SETTING

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The Sacramento River drains the northern portion of the Great Valley of California, which is a structural trough between the Sierra Nevada and the Coast Ranges. The river enters the Great Valley near Red Bluff and flows within the alluvial valley fill, the surface of which is comprised primarily of recent alluvium and the Pleistocene-age, paleo-Sacramento River deposits of the Riverbank and Modesto Formations (Harwood and Helley 1987). Marked changes appear in the character of the river and its floodplain in the valley, with particularly dramatic changes at about Hamilton City (RM 199) (Olmstead and Davis 1961). Upstream of Hamilton City, the river is bounded by a well defined floodplain that is flanked by terraces. In contrast, downstream of Hamilton City, the river flows between natural levees. The recent alluvium is bounded on its margins by outcrops of both the Riverbank and Modesto Formations that define the width of the Holocene-age meanderbelt of the river. Changes in the character of the Sacramento River within the valley could be the result of ongoing structural deformation (Harwood and Helley 1987, Helley and Jaworowski 1985, Schumm and Harvey 1985, Fischer 1994).

Although the major bounding faults (Chico Monocline and Corning/Willows Faults) dominate the structural geology of the northern part of the valley, the smaller anticlines and synclines probably control the course, and perhaps the behavior of the river itself (WET 1990). The Sacramento River enters the Glenn syncline at about RM 205. The width of the active channel deposits widen as the river enters the syncline and then it narrows at RM 200 to RM 197, where the river crosses the axis of the syncline. The active channel deposits widen at RM 197 as the river turns abruptly east and then south to follow the axis of the syncline to RM 173, where the active channel deposits narrow again where the river flows up the structural dip and out of the Glenn syncline. Within the reach of interest (RM 198 to RM 190), the historical data (Larsen et al. 2002) indicate that this reach of the river has been very active within the last 100 years, probably because this reach flows down the structural dip towards the axis of the Glenn syncline (Schumm and Harvey 1985).

Larsen et al. (2002) summarized the historical locations of the river between 1870 and 1997. Between Hamilton City and the Stony Creek confluence, the current sinuosity (ratio of channel length to straight-line valley length, or ratio of valley slope to channel slope) of the river is a minimum for the period of record. Four large bends cut off upstream of Big Chico Creek confluence between about 1900 and 1952, a period before Shasta Dam was constructed. Since 1952 this reach of the river has remained essentially straight, and the river appears to be flowing along the line of the Modesto Formation outcrop, indicating that this is the farthest east the river has been in approximately the last 10,000 years. This suggests that future locations of the river are most likely to be to the west of the present location. An examination of the historical record of the region between Big Chico Creek confluence and Stony Creek also shows that the current alignment of the river represents a minimum sinuosity (Larsen et al. 2002). Minimum sinuosity for the entire reach from Hamilton City to Stony Creek implies that the river slope, and hence the potential sediment-transport capacity of the river, are at their historical maximums. Neill (1984) has argued that bank-erosion rates are equivalent to sediment-transport rates in a reach, and a balance exists between the volume of bed-material deposition in bars and bank erosion. This,

then, suggests that the bank-erosion potential within the reach should be high under current conditions, and that all other things being equal, the sinuosity of the river should increase with time. The rate of bank erosion is also related to the radius of curvature (R_c) –channel width (W) ratio (Nanson and Hickin 1983, 1986), and therefore, as the bend radius decreases with time, the rate of erosion increases until R_c / W is less than about 2.5, when the rate then decreases. The current radius of curvature of the bend at the M&T pumps is about 3,000 ft. For a bend with this radius of curvature, the expected annual migration in the Butte Basin reach of the Sacramento River is about 30 ft per year (Harvey 1989). If erosion continues so the radius of curvature of the bend decreases to about 2,000 ft, however, the annual migration rate can be expected to increase to about 80 ft per year.

Two other factors must be considered when assessing the potential for future bank erosion and planform adjustment in the M&T reach. Completed in the late 1940s, Shasta Dam has enabled the flows in the river to be manipulated to meet irrigation and other needs. A comparison of the pre- and post-Shasta Dam mean daily flow records at the Bend Bridge gage (WET 1990) revealed that the median flow has increased significantly (6,500 to 11,000 cfs) in the post-Shasta period. Although no records exist of the pre-Shasta flow at the Hamilton City gage, the median flow there during the post-Shasta period is about 9,000 cfs. Perhaps the increased summer flows are partially responsible for the increase in the bank-erosion rate in those areas where the toes of the banks are composed of noncohesive sands and gravels, such as the bank opposite the M&T pumps (Harvey 1989).

The potential for future bank erosion and hence lateral migration of the river at the M&T site is also related to the history of emplacement of riprap in the reach between Hamilton City and the M&T pumps. The right descending bank between about RM 198 and RM 197 was revetted by the U.S. Army Corps of Engineers under the Chico Landing to Red Bluff project in 1975 (Plate I-1). The downstream end of the revetment was flanked in the 1983 flood, and the river achieved its current configuration at the mouth of Pine Creek. The revetment, provided it is properly maintained, ensures that the river will remain along the line of the Modesto Formation outcrop between RM 196 and the M&T pumps. This location of the river further ensures that the revetment installed on the left descending bank at about RM 194 in 1973 to protect River Road (Plate I-2) will be required in the foreseeable future. Erosion of the right bank immediately upstream of the pumps is due to flow deflection off the upstream revetment (Plate I-3). With the revetment in place, bank erosion will continue to occur opposite the pumps unless the bank itself is revetted. The left descending bank from the mouth of Big Chico Creek is revetted for a distance of about 2,800 ft (Plate I-4), the revetment protecting the Phelan levee and the present location of the M&T pumps.

The bed, bars, and banks of the Sacramento River between the mouth of Stony Creek and Hamilton City were sampled by the U.S. Army Corps of Engineers (1981), DWR (1984), and WET (1990). The surface materials in the riffles were found to have a median (D_{50}) size of about 30 mm, with a D_{95} of about 80 mm (WET 1990, DWR 1984). The D_{50} of the reach bar surface material was about 22 mm, and the D_{50} of the subsurface material was about 4 mm (WET 1990) (Plate I-5). Approximately 30 percent of the subsurface material proved to be sand-size or finer (<2 mm) (WET 1990). The banks were composite, in generally having a noncohesive toe composed of sands and gravels, and the finer-grained upper bank was composed of more

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cohesive silts and clays (Thorne and Tovey 1981). In the RM 174 to RM 194 reach of the Sacramento River, the noncohesive toe sediments had a D_{50} of about 1.4 mm, while the upper-bank sediments had a D_{50} of about 0.4 mm (WET 1990).



Plate I-1 The Sacramento River, view downstream of revetment at about RM 198R.

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Plate I-2 View upstream of revetment at about RM 194L.



Plate I-3 View downstream of revetment at RM 193L.

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Plate I-4 View of eroding bank at RM 193.5R.



Plate I-5 Bar surface sediments at about RM 193.5L. The D_{50} of the surface sediments is about 22 mm.

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II MEANDER BEND AND GRAVEL BAR MIGRATION NEAR RIVER MILE 192.75 OF THE SACRAMENTO RIVER

By Eric W. Larsen

(Section 3.0 by Yantao Cui and Eric W. Larsen)

1.0 Introduction

Although the section of the Sacramento River near RM 192.75, which is the current location of the M&T pump (Figure II-1), has appeared to be stable with respect to lateral migration in the last century, this river reach has recently begun to migrate westward through natural processes of river migration. Because the pump is located on the east side of the river, the westward migration is a concern because it affects pump operations. Understanding the dynamics of the river that led to its current migration provides important information to inform decisions about effective long-term pump operation.

River meander migration is related to the channel planform shape, flow characteristics, bank erosion potential, and other factors. The history of river meander migration at this site suggests why the river is currently moving away from the current pump site, and helps anticipate future migration. After a brief introduction to the historic planform shape of this reach from 1870 to 1997, which shows the history of channel migration, this section of the report describes the current planform channel shape at the site, upstream from and near the site. Next, the findings suggest that, although the channel migration tendencies appear to have recently changed, now moving the channel away from the pump site, the current migration tendencies are in fact a continuation of the natural evolution of the channel at this site. Finally, the migration of the river 25 years into the future (from 1997 to 2022) is modeled with a physics-based mathematical model. The results, based on modeling the currently-existing conditions, and also modeling a condition with hypothetical additional bank restraint, suggest that the river will continue to move away from the current pump site.

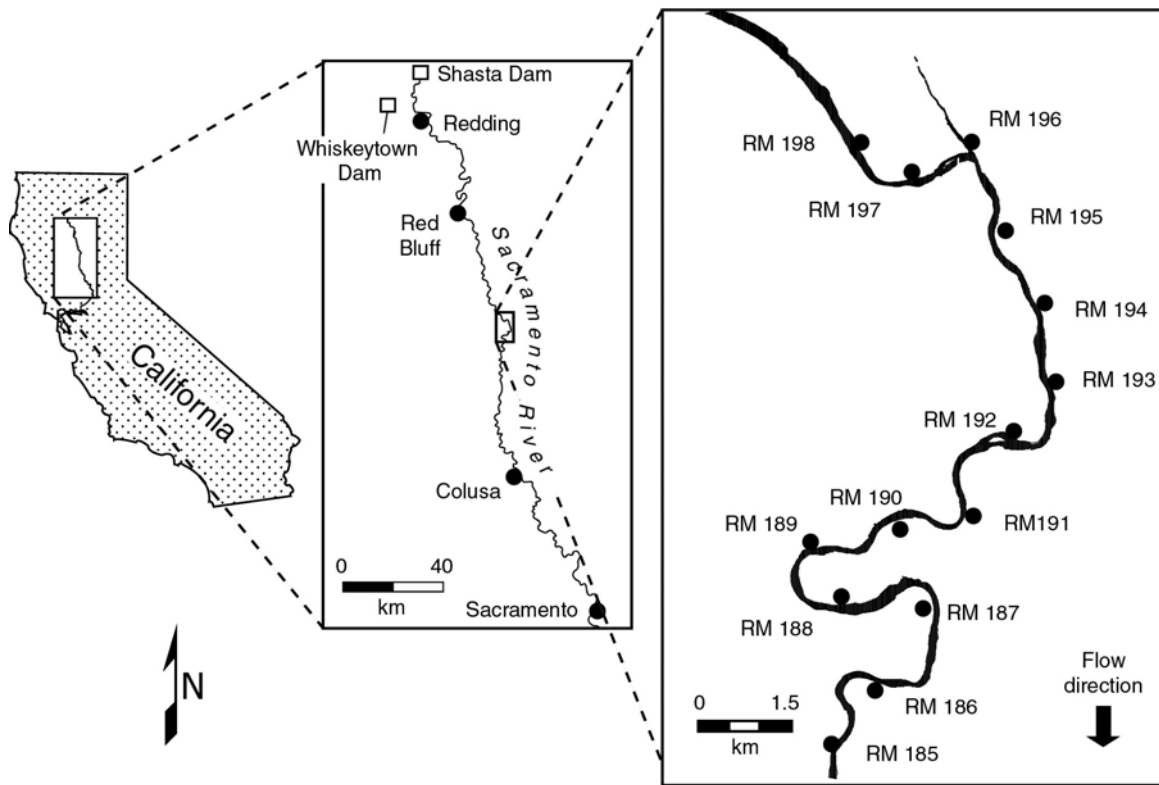


Figure II-1 Location of the Sacramento River and the study reach.

2.0 Existing Conditions

2.1 Site Description

The M&T pump site at about RM 192.75 is located on the upper Sacramento River, about 50 river miles south of Red Bluff, and about 150 river miles north of Sacramento. In most naturally migrating rivers, local meander migration is related to the shape of the local meander bend and to the shape of the river upstream (Johannesson and Parker 1987, Furbish 1988, Furbish 1991). To consider the local migration at the M&T site (RM 192.75), this report looks at a longer reach that includes a section of river upstream from the site, namely, the reach going up to the bend above Pine Creek (about RM 196) (Figure II-1). This reach, like much of the river between Colusa (RM 143) and Red Bluff (RM 243), contains some areas having a moderate amount of bank constraint where the river does not move, some areas that are migrating and evolving in relation to the bank constraint, and some areas that are evolving freely.

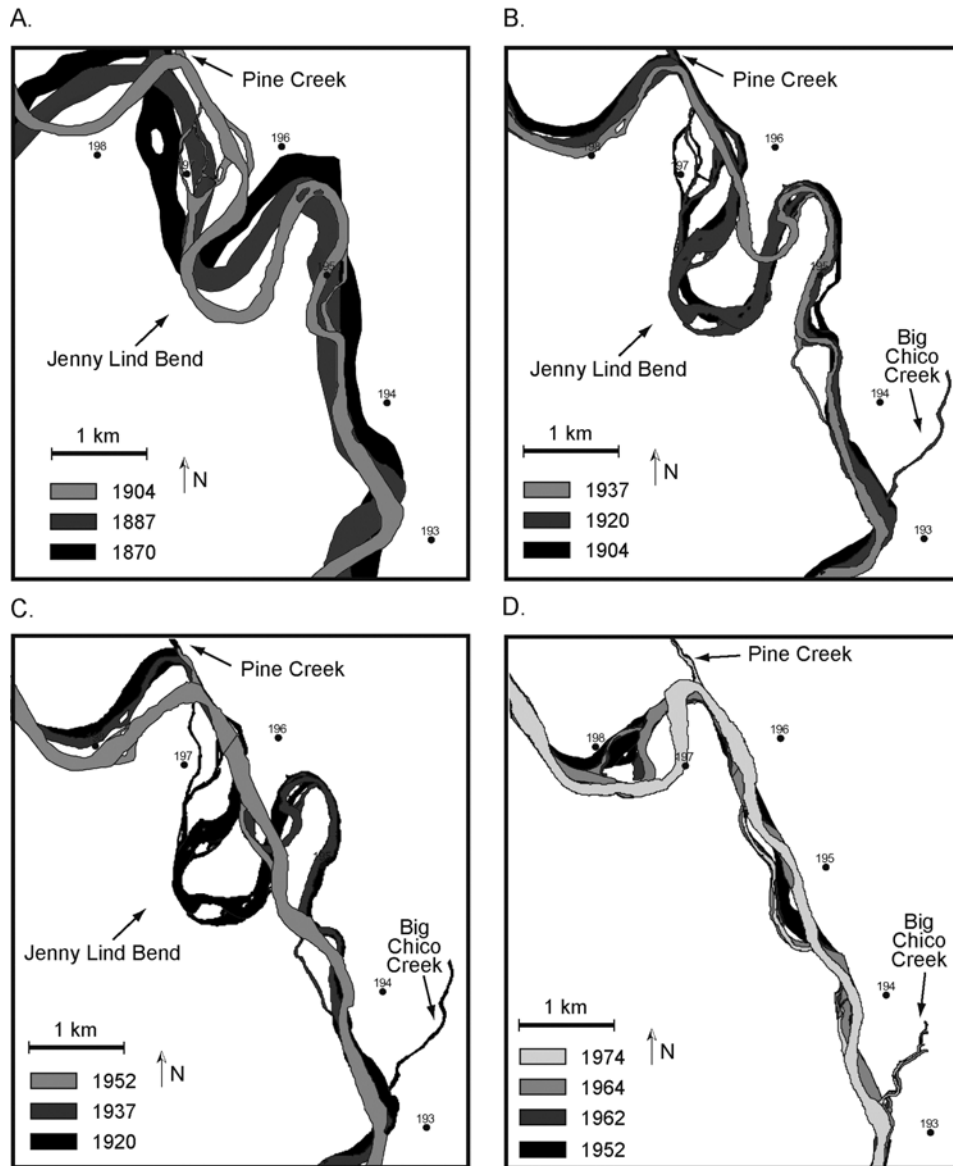


Figure II-2 Jenny Lind Bend, RM 198-193. Historical channel movement from (A) 1870-1904, (B) 1904-1937, (C) 1920-1952, (D) 1952-1974

Figures II-2 and II-3 show the history of the channel location from 1870 to 1997. Between river miles 198 and 192.5, a reach extending roughly from upstream of Pine Creek to south of Big Chico Creek, were two historically large bend complexes, Pine Creek and the Jenny Lind set of bends (Figure II-2 A, B, and C). By 1950, the Jenny Lind set of bends had cut off, and by 1974 Pine Creek Bend, restrained by riprap, had ceased to migrate (Figure II-2 D). After the cutoffs occurred, the channel started to develop some mildly sinuous bends, which were fairly well developed by 1974 and have evolved since then. In 1974, the site where the pumps are now located (RM 192.75) was located on the outside of the concave section of a bend (Figure II-5 1974-1982), although at that time, no pump was there.

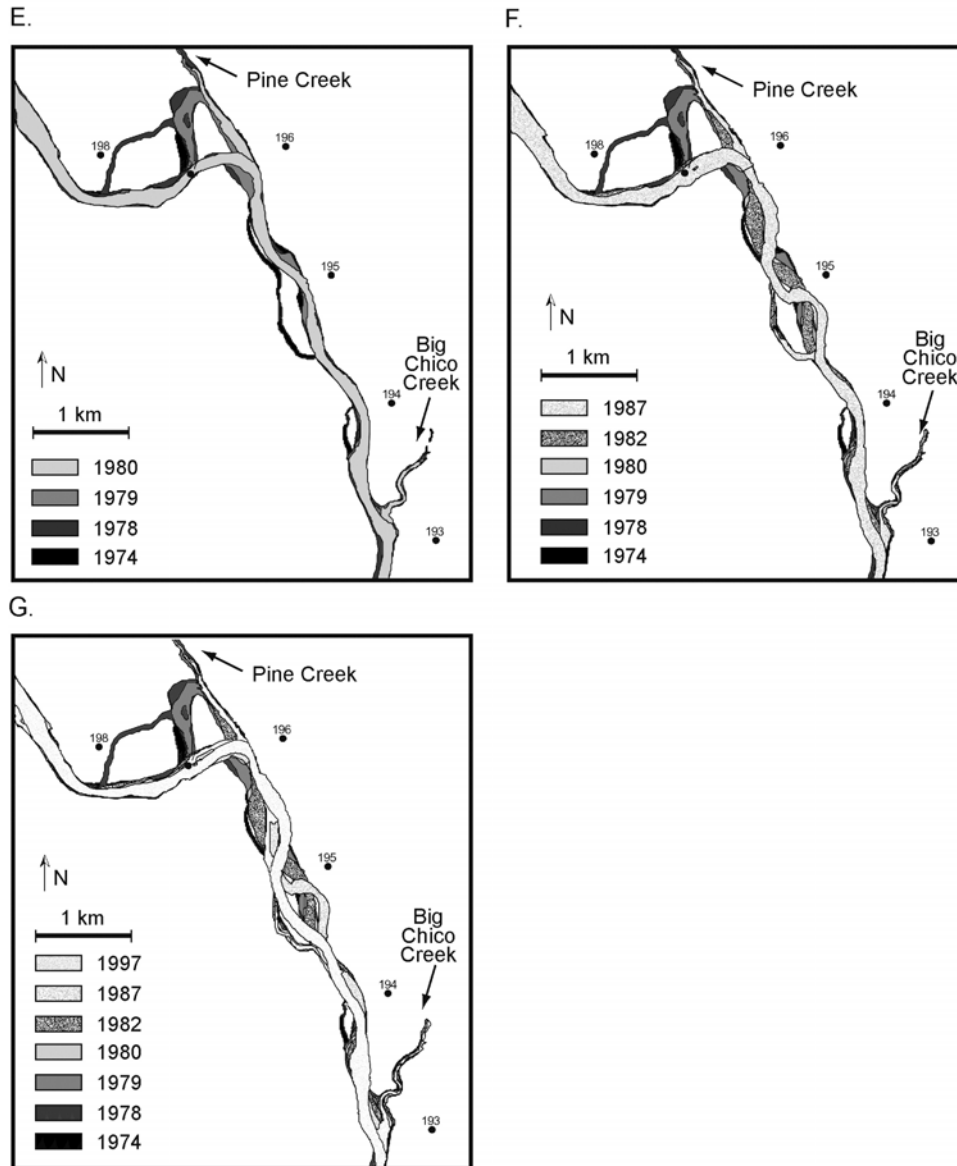


Figure II-3 Jenny Lind Bend, RM 198-193. Historical channel movement from (E) 1974-1980, (F) 1974-1987, and (G) 1974-1997.

Meander bends tend to migrate naturally across the landscape (Brice 1984, Hooke 1984). Bend migration tends to follow patterns that can be described by mechanical laws of fluid flow and by other methods (Brice 1974, Hooke 1984, Ikeda and Parker 1989). When such meander bend migration occurs, an individual bend tends to move both downstream and cross stream. In other words, because of the downstream component of migration, a bend will tend to continuously migrate downstream. At the same time, because of the cross-stream component of migration, a bend will tend to migrate cross stream. As the bend migrates, it also changes shape.

Natural river meander bends tend to be curved. When a bend impinges laterally on a bank that is erosion resistant, the curved shape tends to flatten against the resistant bank. As the bend moves downstream, the outward side of the bank will tend to maintain contact with the location of the

resistant bank. Once it has migrated sufficiently, the “end of the bend” will move downstream from the location, and the river channel will no longer maintain contact with the point in question. This is what appears to have happened near the pump site at RM 192.75.

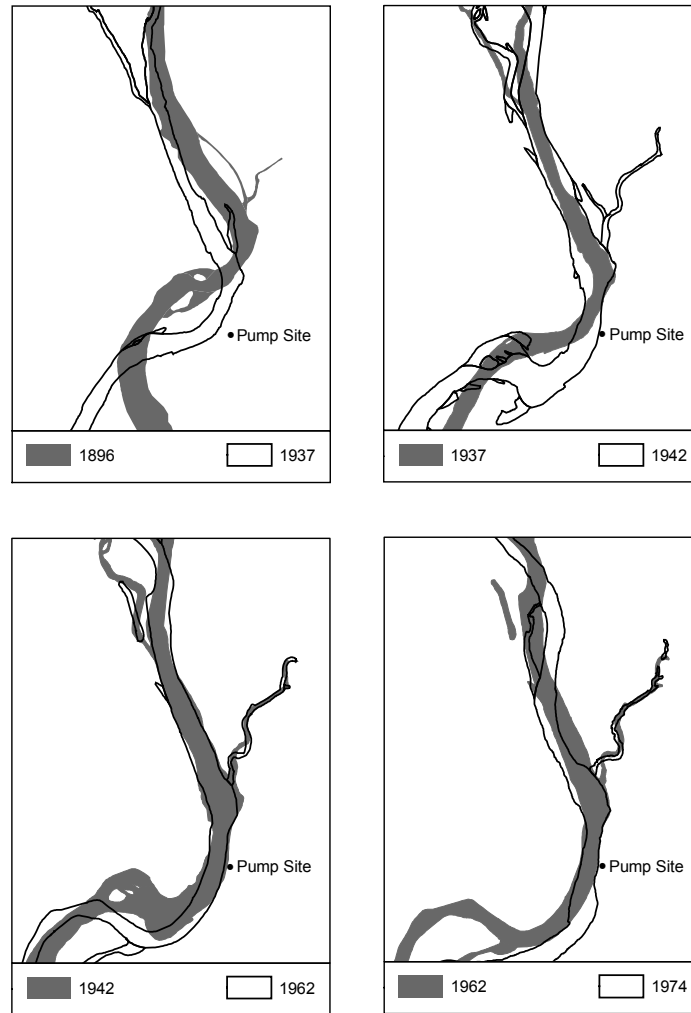


Figure II-4 Sacramento River, RM 194.5 to about RM 192, showing the river in the immediate vicinity of the pump. The figures show the edge of the water as taken from maps and aerial photos of the Sacramento River. The pump is shown as a dark black dot. Successive mapping periods are shown superimposed on each other.

In our database, 1896 is the year of the earliest channel planform for which we have a reliable map of Big Chico Creek where it joins the Sacramento River. The mouth of Big Chico Creek is directly upstream of current pump site. Between 1896 and 1937, the channel outside bend had not yet reached the site where the pump is now located (Figure II-4 1896-1937); the bend was moving progressively downstream (Figure II-4 1896-1937). By 1942, the channel’s outside bank had reached the pump site (Figure II-4 1937-1942). Over the next 20 years, the channel near the site did not move significantly (Figure II-4 1942-1962); the eastward movement probably ceased because the river had reached an erosion-resistant zone of bank (Buer, pers. comm.).

Meander Bend and Gravel Bar Migration Near River Mile 192.75 of the Sacramento River

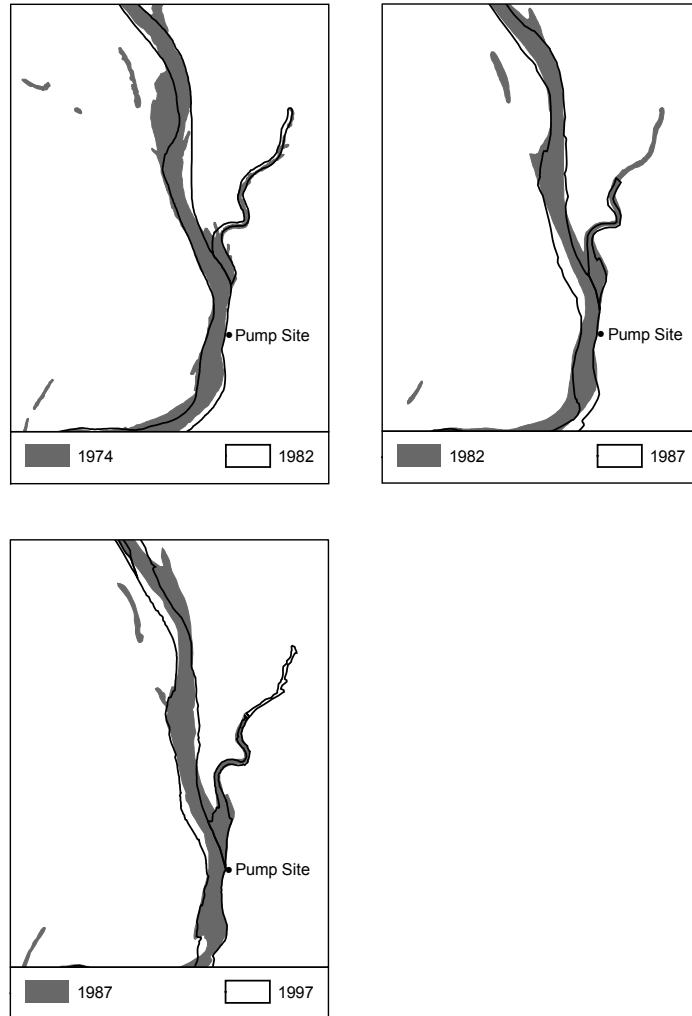


Figure II-5 Sacramento River, RM 194.5 to about RM 192, showing the river in the immediate vicinity of the pump. The figures show the edge of the water as taken from maps and aerial photos of the Sacramento River. The pump is shown as a dark black dot. Successive mapping periods are shown superimposed on each other.

By 1952 the major cutoffs upstream had occurred (Figure II-2 D), and by 1974 the channel had begun to develop some sinuosity with defined mildly sinuous bends (Figure II-3 F and Figure II-4 1962-1974). Between 1974 and 1982, these bends continued to migrate downstream, particularly upstream of the site. Between 1982 and 1987, the channel bend upstream had continued to migrate south, and the mouth of Big Chico Creek moved south and west (Figure II-5 1982-1987). Between 1987 and 1997, the apex of the bend continued to migrate south (downstream), reaching the pump site by 1997 (Figure II-5 1987-1997). Figure II-6 clearly shows the progression of bend migration near the pump between 1982 and 1997. The southern end of the mouth of Big Chico Creek, which corresponds to the apex of the bend in the Sacramento River, progressively moved south in this time period.

As the river continues to evolve, the apex of the bend will tend to move downstream and the river channel will appear to move away from the eastern bank at the current location of the

pump. At the end of this section of the report, we illustrate that even if the channel is constrained upstream on the opposite bank, it will attempt to follow a similar pattern of migration.

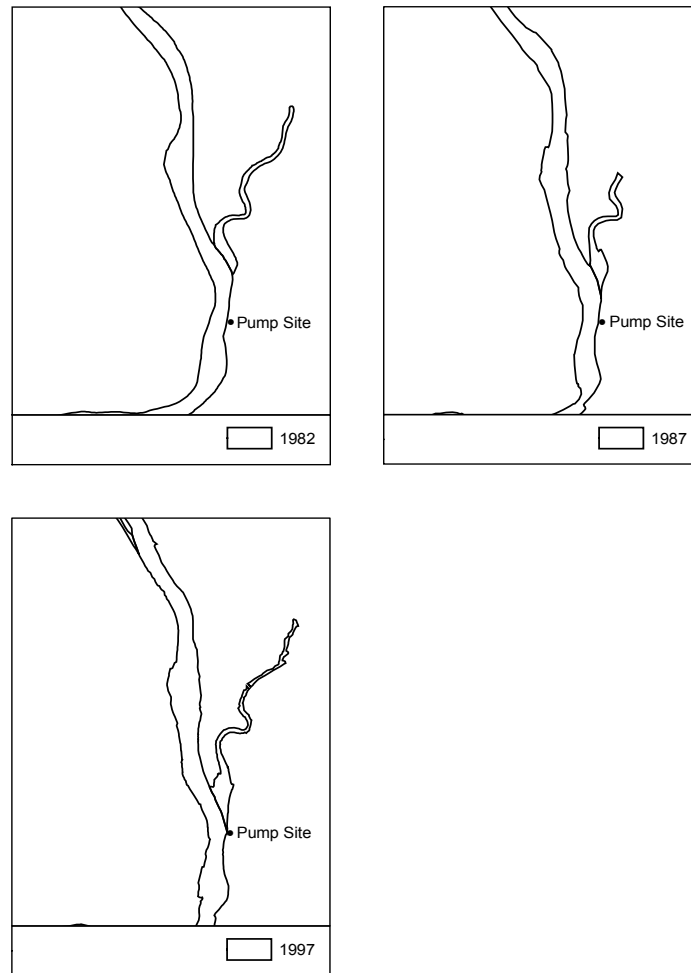


Figure II-6 Sacramento River near the mouth of Big Chico Creek. This series of channel locations shows the mouth of Big Chico Creek (and the apex of the bend) progressively approaching the pump site.

Another way to describe the river moving toward the location of the current pump but not going beyond it is to say the river reached the extreme eastern edge of the historic meander belt, and that it is now moving away from that edge. This location, being the eastern edge of the meander belt zone, has functioned as a geologic control. Essentially what has happened at this site is that a bend migrated mostly in the downstream direction. The outside of the bend reached the location of what would later become the pump site. Because this area was naturally resistant to erosion, the outside of the bend curvature effectively “flattened out” and then “slid down the site,” maintaining contact with the bank at the current pump location. During that time period the site was “stable” with respect to bank erosion.

Currently, the upstream end of the bend has reached the site and, as the channel continues to naturally migrate downstream, the concave portion of the upstream bend will migrate through the

site. From the point of view of standing at the site, the channel appears to be moving away from the eastern bank.

3.0 Alternate Bar Migration

by Yantao Cui and Eric W. Larsen

Another way to describe the channel migration tendencies near the M&T pumping station is that the observed channel movement is the result of migrating alternate bars. This can be observed from the historical channel planform and analyzed with the theory of Colombini et al. (1987).

3.1 Historical Channel Planform

A sketch of alternate bars, based on Figure 7 in Larsen et al. (2002) and historical aerial photographs is given in Figure II-7 below. It shows clearly that the alternate bars are moving downstream in time.

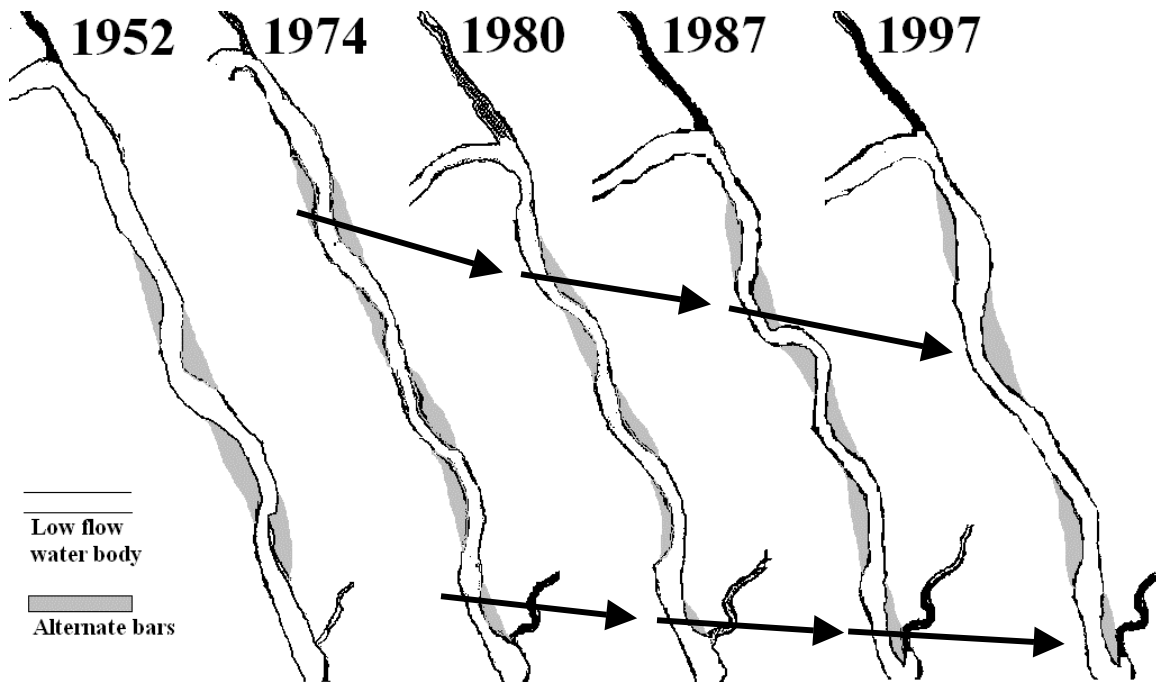


Figure II-7 Locations of alternate bars in the study reach in 1952, 1974, 1980, 1987, and 1997 (from aerial photos).

3.2 Theoretical Analysis

In attempting to understand why the bars are moving downstream, we considered a theory that describes natural bar formation and migration. Based on a mathematical analysis of the growth rate of finite amplitude alternate bars (“finite amplitude” means that they reach a certain shape and maintain that shape, rather than grow and disappear), Colombini et al. (1987) discovered that alternate bars form in rivers under certain conditions. Although they confirmed that alternate bars can migrate either upstream or downstream, depending on the hydraulic conditions in the river, most natural bars in meandering rivers migrate downstream.

According to Colombini et al. (1987), alternate bars form when the width-to-depth ratio of a river exceeds a critical value of approximately 13. The bankfull discharge of the Sacramento River at the study reach can be estimated as a peak flow of between 1.5 and 2 years, (Leopold et al. 1964), or between 70,900 and 86,300 cfs. Assuming that the bankfull discharge is 80,000 cfs for simplicity, the estimated bankfull width reach is approximately 1,300 ft based on a U.S. Army Corp of Engineers HEC-RAS study (USACOE 2003). Assuming a Manning’s n value of 0.035, the estimated bankfull depth of the river in the study reach is approximately 15 ft. Based on that analysis, the bankfull width-to-depth ratio in the study reach is approximately 87, much higher than the critical width-to-depth ratio of approximately 13, and thus alternate bars should naturally form in the river.

Although the theory of Colombini et al. (1987) provides general guidance on the direction of migration of alternate bars, its prediction is not practical because of the difficulty in determining various parameters involved. The drawings of the historical aerial photographs shown in this section, however, undoubtedly indicate the rapid downstream migration of the alternate bars.

4.0 Future Predictions

4.1 Introduction

The historical analysis of meander bend and alternate bar migration and the theoretical analysis of bar migration all suggest that the current pattern of channel movement will continue into the future. If such movement continues, the river channel near the current pump will continue to move to the west as the bends and bars migrate downstream.

Another approach to understanding the future channel movement near the pump site is to model its future migration. As Larsen et al. (2002) recently did for a longer reach of the river that includes the pump site, “we simulate migration using a channel migration model that is based on mathematical-physical algorithms for flow and sediment transport” (the main physical processes responsible for channel migration) (Larsen and Greco 2002). Because the model is based on physical processes, it can accommodate changes in input variables and can predict the consequences of conditions, such as bank stabilization measures that have not existed in the past.

Unlike empirically-based models, which tend to focus on local conditions, the physically-based numerical model integrates the effects of local morphology and upstream conditions.”¹

4.2 Methods

Because of time constraints, modeling relied on a previous calibration of a similar model (Larsen et al. 2002), although the current calibration results show good agreement (Figure II-8). Based on the current calibration, we expect the overall direction and pattern of the current prediction to be valid, although the timing and distances of movement could be more thoroughly estimated with a more extensive model calibration and validation.

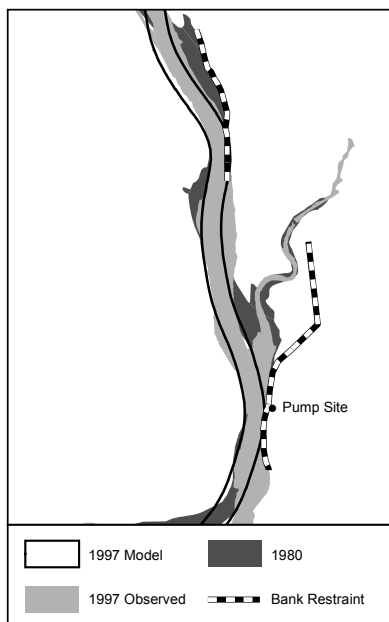


Figure II-8 The unfilled black lines indicate the calibrated modeled channel bank lines. Upstream of the site, the calibrated and observed channels show good agreement.

According to Larsen et al. (2002), “A steady flow of 80,000 cfs is used in the analysis, which approximates the calculated two-year return interval.” Slope, channel top width and area of flow within the designated channel come from HEC-RAS output. Average depth is calculated using channel area and channel top width ($\frac{\text{area}}{\text{top width}}$). The overall slope for the study reach is calculated based on HEC-RAS model information. The study reach has a drop of 10 m (34 ft) in water surface elevation over 16 river miles, or 0.00040 m/m. The following input parameters for the meander migration model for predictive modeling were calculated using the output of HEC-RAS:

- Slope: 0.00040 m/m
- Top width: 235 m (770 ft)
- Average depth: 5 m (18 ft)²

4.3 Model Calibration

“Calibration of our meander migration model is required because we do not know the erodibility of the sediments within the study reach. Calibration allows us to calculate an erodibility field by running the model on historic channel data. Calibration also allows us to fine-tune the model to local conditions by adjusting the coefficient of friction.”³

Figure II-6 shows the calibration of modeling. To calibrate, we started with the observed locations of the channel in 1980 and in 1997. We adjusted bank erodibility near the channel until the 1997 modeled channel matched the observed 1997 channel location, as shown in Figure II-8. These conditions were then used for model predictions.

¹ Larsen et al. 2002.

² Larsen et al. 2002.

³ Larsen et al. 2002.

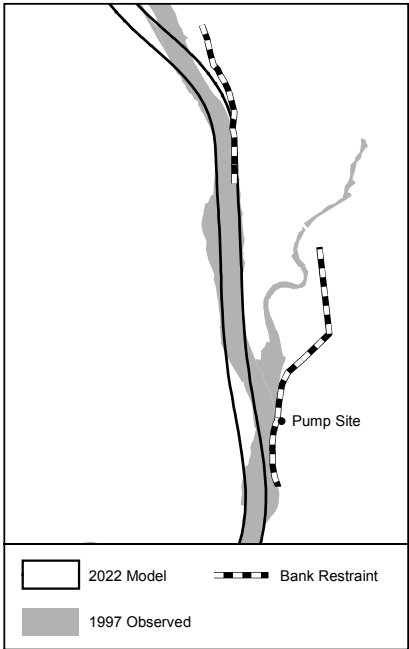


Figure II-9 Sacramento River near M&T pump site, 25-year predicted channel migration with current conditions.

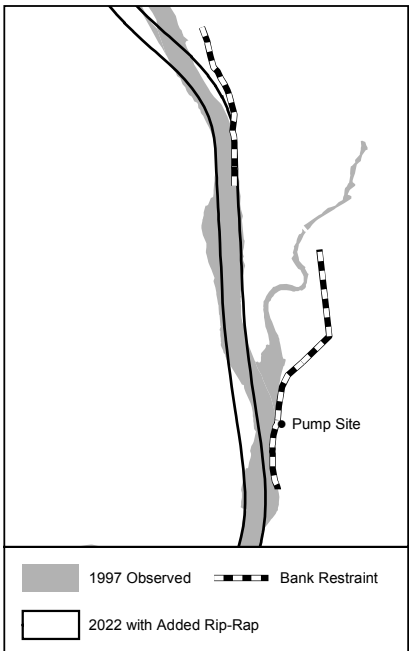


Figure II-10 Sacramento River near M&T pump site, 25-year predicted channel migration with additional bank restraint installed upstream on the west bank.

the migration away from the pump might be stopped, but it would probably not solve the current problem of sand deposition near the site. Because the river is currently away from the pump site,

4.4 Results

Based on the input values for hydraulic variables given above and the calibrated bank erosion values, two predictions 25 years into the future were made. The first predicted future channel movement if no additional bank constraints were added, while the second predicted movement if bank restraints were applied to the eroding west bank upstream of the pump site.

4.4.1 With No Additional Bank Restraint

Figure II-9 shows the channel location with 25 years of predicted migration using the input parameters that were used for the calibration, and the existing bank restraints as shown in Figure II-9. The 25-year prediction suggests that the channel will continue to move away from the pump site.

4.4.2 With Additional Bank Restraint

Figure II-10 shows the channel location after 25 years of predicted migration with additional bank restraint installed upstream on the west bank. With the addition of bank restraint, the 25-year prediction suggests that the channel will continue to move away from the pump site in a manner almost identical to that of the previous case. In essence, with bank restraint installed upstream to limit westward migration, the channel continues to “slide along the riprap” and migrate downstream.

5.0 Discussion and Conclusions

Because the channel is moving downstream and because the channel would continue to move away from the pump even with the modeled riprap, the results suggest that constraining the channel in this way will not keep the channel from moving away from the pump, and that bank restraint would not be an effective strategy to protect the pump as it currently exists.

If more extensive bank restraint than we modeled were installed on the west bank directly across from the pump, the migration away from the pump might be stopped, but it would probably not solve the current problem of sand deposition near the site. Because the river is currently away from the pump site,

and a problem already exists, the river – or at least the flow – would need to be redirected toward the east bank if the current problem were to be solved. One possibility for remedying this problem is the use of groins, with which upstream and perhaps downstream areas would also have to be constrained so that the river forces producing downstream migration of bars and bends could be anticipated and counteracted where possible. Detailed studies of the feasibility of success, initial expense, and continued maintenance would be key issues to explore with any of these channel restraint and training options. With the sediment transport dynamics, the meander migration dynamics, and some natural variability, finding ways to pump water so that the main pump intake structure is out of the river seems critical.

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III RIVER REALIGNMENT AND BANK PROTECTION

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Potential solutions to the sedimentation problems at the M&T pumps are to prevent further erosion of the right bank of the river opposite and upstream of the pumps, and to force the currently eroding riverbank farther to the east. Both solutions can be achieved by installing dikes along the right bank. As we have few hard and fast criteria for designing dikes (Biedenharn et al. 1997), and dikes have not been widely used on the Sacramento River, a review of the literature on dike design is in order.

1.0 Dikes

The dike is an indirect means of protecting banks from erosion (Biedenharn et al. 1997). Dikes are structures that extend out into the stream channel, are generally transverse to flow, and redirect the flow so the hydraulic forces at the channel boundary are reduced to a nonerosive level. Depending on their construction, dikes can be classified as either permeable or impermeable. Dikes are also referred to in the literature as groins, hard points, jetties, spurs, and wing dams.

The advantages of dikes over direct revetment techniques include:

- little or no bank preparation needed, which reduces cost and environmental impact
- ability to modify existing channel alignment and geometry
- ability to monitor an installation and extend if required
- generally increased geotechnical bank stability, from inducing deposition between the dikes that provides toe protection and reduces bank height

The major disadvantages of dikes include:

- inability to immediately remedy bank erosion caused by geotechnical instability (as when the bank height exceeds the critical bank height for stability)
- potential for creation of recreational and navigational hazards
- difficulty in construction
- necessity of monitoring and maintenance

Dikes can be applied to a wide range of conditions. The most common use, though, is on shallow, wide streams with moderate to high transport of suspended bed material. Such use is effective because shallow channel depths reduce the required height of the structures, a wide channel provides room for the channel alignment and geometry to adjust, and the suspended-sediment bed material accelerates the rate of induced deposition between the dikes. In larger rivers, dikes are generally used to increase depth for navigation, in addition to improving alignment and stabilizing banks. Where establishment of riparian vegetation is a high-priority,

dikes are preferred over direct bank-protection techniques, in part because they reduce the impact of the revetment on the availability of “soft” banks.

On the Sacramento River, dikes have not been used extensively to prevent bank erosion. Impermeable dikes (wing dams), though, have been used successfully below Verona (RM 80) to narrow the channel and provide for greater low-water navigation depths. Dumped concrete rubble dikes have also provided bank protection at a number of locations along the Sacramento River, including RM 186R. Rock dikes have been used successfully to prevent bank erosion on the American River downstream of Nimbus Dam, and on the Yuba River below Daguerre Point Dam. Permeable dikes (palisades) placed upstream of Woodson Bridge (RM 218) failed and had to be removed because of the relatively low suspended-sediment load of the river (WET 1989).

1.1 Design Considerations

No formal, standardized criteria exist for evaluating dike design. Brown (1985) provided an extensive review and analysis of dikes for the Federal Highway Administration (FHWA). The report was based on model tests, literature reviews, and a survey of several hundred field installations. WET (1989) conducted a field survey of permeable-dike installations on the Sacramento, Red, Arkansas, and Canadian Rivers. It evaluated the relationships between successful installations and the suspended-sediment load of the river, and applied the findings to a dike design at RM 192.4L on the Sacramento River.

The US Army Corps of Engineers (1981) also conducted studies of dikes and incorporated their findings into design parameters. Dikes are generally used in straight reaches and long radius-of-curvature bends ($R_c/W > 3$), because as bend radius decreases, spacing between the dikes must be reduced, and the number of dikes required increases to where a continuous parallel retard could be built for the same cost or, if channel realignment is not required, a direct armor technique could be employed.

Dike design involves the following factors:

- permeability
- length
- spacing
- angle with respect to flow
- height
- bankhead (root) design
- structural scour protection

1.1.1 Permeability

Permeability is defined as the ratio of the area of openings in the dike to the total projected area of the dike, and is expressed as a percentage. Brown (1985) suggests that where a large reduction in at-bank velocity is required, such as with sharp bends, permeability should not exceed 35 percent. Where a moderate reduction in velocity is sufficient, such as with moderate-curvature bends, the permeability can be as much as 50 percent. Permeabilities higher than 50 percent are

not recommended, except where rivers are transporting very high suspended bed-material loads (USACOE 1981).

1.1.2 Length

The length of a dike – measured from the existing bankline to the riverward end – is dictated by the desired alignment of the channel if the channel is to be realigned. Where stabilization of the existing bankline is the goal, the determining of the proper length of the dike is not so simple. Brown (1985) states that dike length affects the local scour depth at the tip of the dike, the angle of flow deflection induced by the dike, and the length of the streambank protected by the dike. Selection of an appropriate dike length for bank protection purposes is site-specific, but general guidelines can be given as follows:

Permeability (Percent)	Recommended Projected Length of Dike (Percent of Channel Width)
0-35	15 or less
80	25 or less
Table III-1 Dike permeability and length.	

For permeabilities between 35 and 80 percent, linear interpolation between 15 and 25 percent of channel width can be used to determine maximum allowable length. Channel width is defined as bankfull width, and projected dike length is measured perpendicular to the main flow direction. If the dikes are being used to change the channel alignment, then the dike lengths will often exceed the bank protection limits.

1.1.3 Spacing

Spacing and length are usually considered to be related, and thus much of the literature uses the ratio of the two rather than their individual values. In the absence of a need to construct dikes to a predetermined channel alignment, the optimum length-to-spacing ratio becomes a site-specific, economic determination involving a trade-off between shorter dikes at a closer spacing and longer dikes at a greater spacing. In practice, spacings from one to six times the dike length have been used, but model studies suggest that the optimum spacing of impermeable dikes is between two and three times the dike length (USACOE 1981, Copeland 1983). A conservative approach is to use a spacing equal to the dike length.

1.1.4 Angle With Respect to Flow

Few experts agree as to the optimum angle that dikes should have with respect to the direction of flow. But on this they agree: Holding other factors constant, the shortest dikes – those constructed on the shortest path from the bankline to the desired new channel alignment – will be the cheapest. Usually, this path will be approximately perpendicular to flow, or the bankline, or a compromise between the two. Brown (1985) suggests that the angle is not critical to permeable dikes, and that better performance can be obtained with impermeable dikes if the upstream dike in the system is constructed at an angle of about 150 degrees, with subsequent dikes having

successively smaller angles, reaching a minimum of 90 degrees for the downstream dike. Contrary to intuition, dikes angled downstream can form downstream scour holes nearer to the bank than if they were perpendicular to the bank or angled upstream to the flow, because overtopping flows will tend to form an erosive “roller” or plunging flow, immediately adjacent and parallel to the structure, to the detriment of bank stability.

1.1.5 Height

The height of dikes in a system is often related to the height of the bank, which in turn, is related to some recurrence frequency of river stage. Brown (1985) states that a dike needs only to be high enough to protect the bank zone of active erosion. He provides these general guidelines: (1) dike height should be no higher than the top bank, but no lower than three ft below the design flow, (2) impermeable dikes should be submerged at least three ft at the most severe expected flow condition to reduce local scour, and (3) permeable dikes should be lower than flow stages that carry significant debris loads.

In practice, the height of the dikes is often dictated by economics, as costs increase rapidly with height. For permeable dikes, this rapid increase in cost is due to structural considerations. For impermeable dikes, the increase in cost is due to an exponential increase in rock volume as height increases. Generally speaking, an acceptable range of dike heights is between one-third and two-thirds of the bank height. The height of the dike can be varied from the bankhead to the riverward end, this variation giving the dike a downward slope that confers both economic and environmental benefits. The sloped surface creates less flow constriction as flows increase, and often results in a more economical dike. Moreover, the dike profile can be notched to allow flows to enter the dike system for environmental purposes.

1.1.6 Bankhead (Root) Design

Dike bankheads must be designed so that erosion does not flank the structure. Two general approaches to preventing unacceptable amounts of erosion are: (1) excavate a trench into the bank and extend the dike back into the trench, and (2) armor the bank downstream of the bank with riprap over a graded bank. “Rules of thumb” derived from USACOE experience suggest that the dike root should be at least as high as the bank, and if local scour is likely to be a problem, the trench length should be increased by the local scour depth. If downstream bank paving is to be used, the downstream extent from the dike should be at least three times the bank height plus any local scour depth.

1.1.7 Structural Scour Protection

Structural scour protection counteracts scour induced by the dikes. Usually, this protection can be afforded by placing a rock blanket on the bed adjacent to the dike, or by placing extra material at the end and along the sides of the dike that will launch into any scour holes that form.

2.0 Preliminary Design for RM 193R

A preliminary design for a 2,500-foot-long dike field was developed for the RM 193R site. The objectives of the design were to both prevent further bank erosion and realign the channel bank somewhat to the east, thereby increasing the sediment-transport capacity of the flows while reducing the local supply of sediment from bank erosion.

The analysis included an evaluation of the channel geometry, and an assessment of the local hydraulic characteristics to assess potential scour. The downstream dike was located near the upstream end of the existing left bank riprap near the mouth of Big Chico Creek at RM192.9. The upstream dike was located just downstream of the split-flow channel mouth at RM193.4.

The geometry of the reach from RM129.9 to RM193.4 was obtained from a HEC-RAS model, which provided an average channel width through the reach of about 800 ft. A spur length of 120 ft was obtained using the rule of thumb that the impermeable spur length (the length perpendicular to the direction of flow) should be about 15 percent of the channel width. But because the right-bank angle of the upstream end of the reach was relatively flat, the length of the spur dikes was set to 170 ft to ensure the nose of the spur extended beyond the toe of the bank. The dike spacing was determined using a longitudinal distance of three times the spur length, or 360 ft. The existing channel geometry indicates that the bank height through the reach is about 30 ft. The portion of the dike keyed into the bank is referred to as the root, and the dimensions of the root are, again, based on the bank height. The height of the root was set to two-thirds of the bank height, while its width was set equal to the bank height. The top slope of the dikes was assumed to be 5 percent extending from the root to the top of the dike nose, and the slope of the spur nose was assumed to be 2H:1V. The top width of the dikes was set to 5 ft, and the spur sideslopes were 2H:1V. Figure III-1 shows an example of dike geometry.

The alignment of the dikes is shown in Figure III-2. A total of eight dikes is required to protect the reach at a spacing of 360 ft. Typical dike designs include a downstream alignment of the upstream dikes to deflect flows from the bank, with downstream dikes being positioned perpendicular to the direction of flow. The upstream three dikes are 170 ft long, ensuring their noses extend beyond the toe of the bank slope, while the downstream five dikes are the standard 120 ft.

To evaluate the increase in rock volume necessary to protect the dikes from scour, a preliminary scour analysis was performed. The analysis was based on hydraulic information from the model at the 50-year event ($Q = 160,000$ cfs). Results found bend scour ranging from about 2.4 ft to about 4.2 ft, with contraction scour around the noses ranging from 3.6 to 4.9 ft, and a maximum total scour depth of about 9.1 ft. To account for potential scour around the noses of the dike, the volume of rock in the noses was multiplied by a factor of 2 to provide sufficient stone for launching into the scour hole.

A summary of the length, height, volume, and cost of each of the dikes is presented in Table III-2. The weight of the riprap is based on a bulked specific weight of 100 pcf, and an in-place cost of \$30 per ton was used for the cost analysis. The total cost for the design of the dike, which covers about 2,520 ft, is about \$1.34 million, or about \$530 per linear foot.

Dike No.	Length (ft)	Root Height (ft)	Riprap Volume (yd ³)	Riprap Weight (tons)	Cost (at \$30/ton)
1	200	20	4,420	5,967	\$ 179,009
2	200	20	4,420	5,967	\$ 179,009
3	200	20	4,420	5,967	\$ 179,009
4	150	20	3,959	5,345	\$ 160,347
5	150	20	3,959	5,345	\$ 160,347
6	150	20	3,959	5,345	\$ 160,347
7	150	20	3,959	5,345	\$ 160,347
8	150	20	3,959	5,345	\$ 160,347
				Total	\$1,338,762
				Subreach Length	2,520 ft
				Unit Cost	\$ 531
Table III-2 Summary of dike dimensions, volumes, and costs.					

3.0 References

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- Brown, S.A., 1985. Design of spur-type streambank stabilization structures. FHWA Report No. FHWA/RD-84/101.
- Copeland, R.R., 1983. Bank protection techniques using spur dikes. U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi, Miscellaneous Paper HL-83-1, 32 p.
- U.S. Army Corps of Engineers, 1981. The Streambank Erosion Control Evaluation and Demonstration Act of 1974. Section 32, Public Law 93-251, Final Report to Congress, Main Report and Appendices A through H, December.
- Water Engineering & Technology, Inc., 1989. Supplement No. 6 to Design Memorandum No. 2, Sacramento River Bank Protection, California. Prepared for the U.S. Army Corps of Engineers, Sacramento District, Contract No. DACW05-88-D-0044, Delivery Order No. 0002, Project No. 82-502-88, May.

River Realignment and Bank Protection

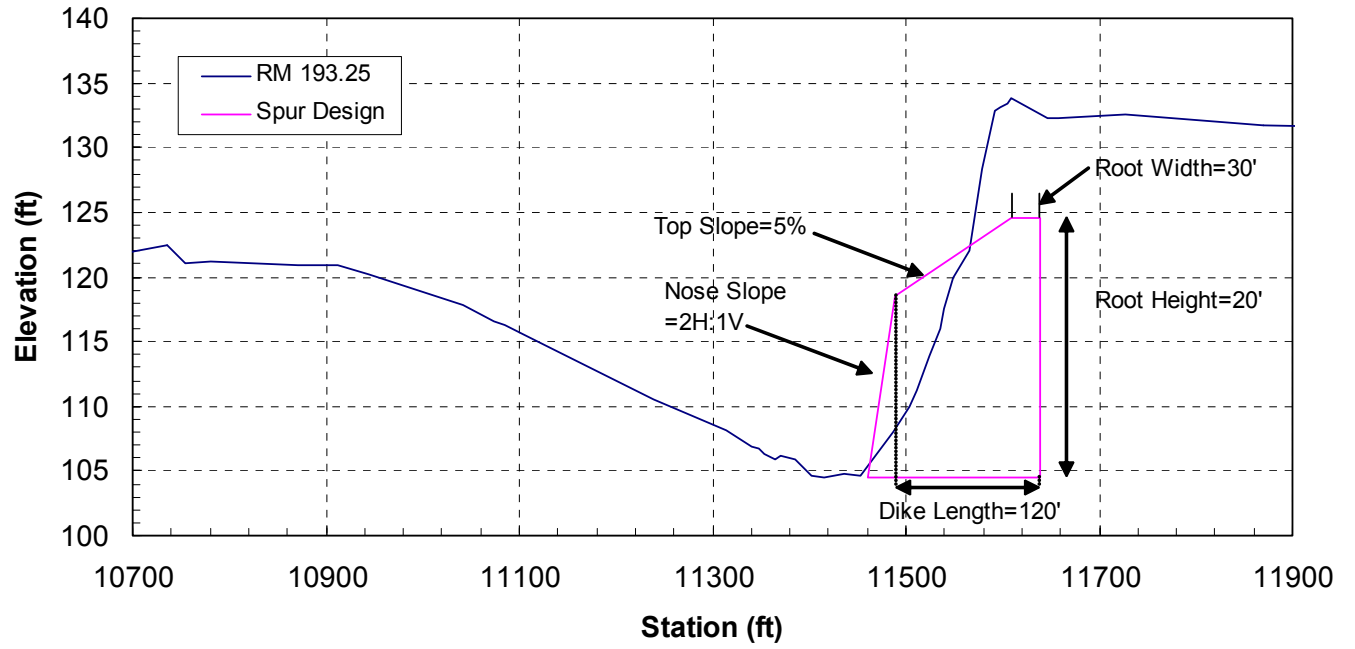


Figure III-1 RM193.25 showing an example of dike geometry.

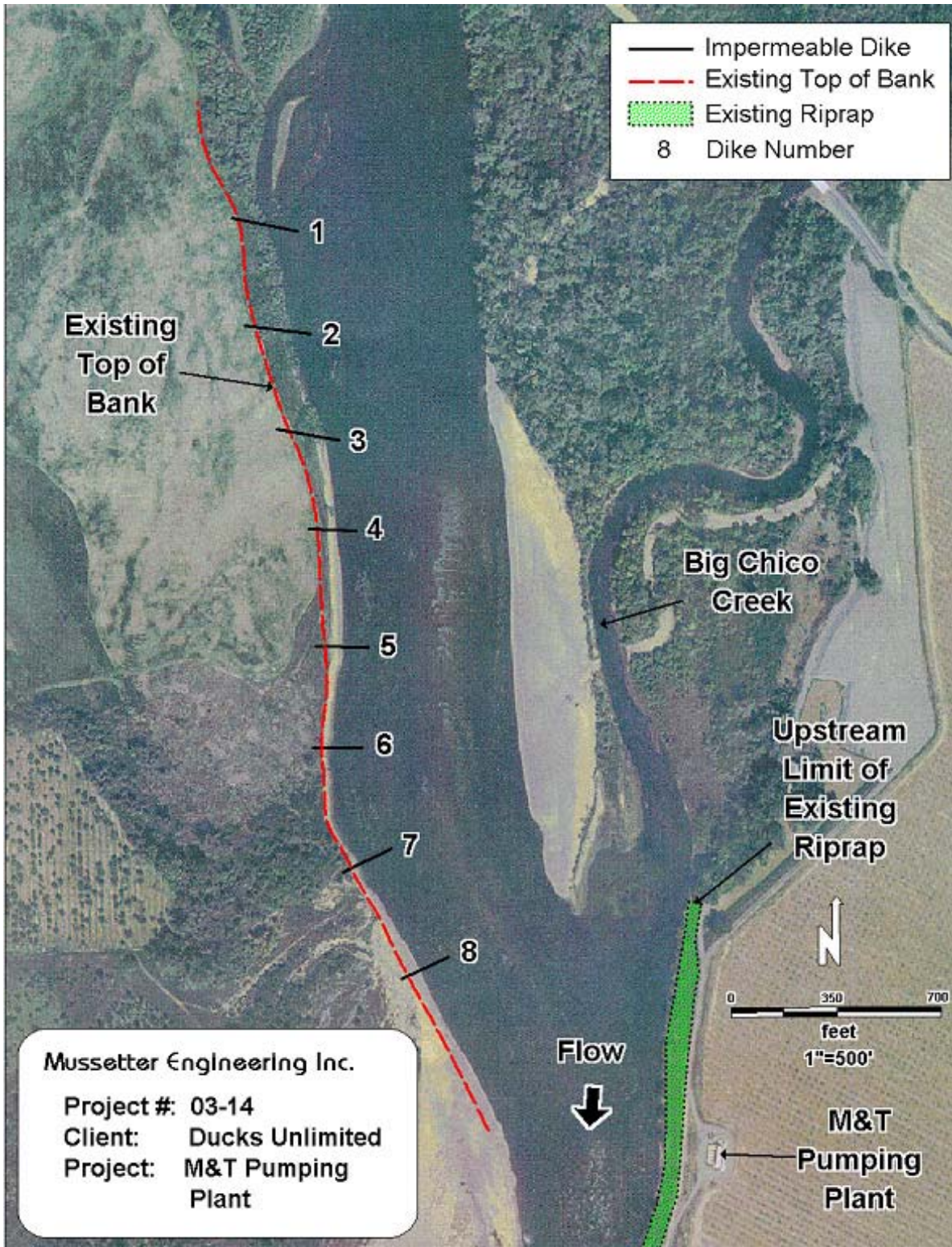


Figure III-2 Preliminary dike design and layout, RM 193R.

IV BED-LOAD TRANSPORT IN SACRAMENTO RIVER NEAR THE M&T PUMPING PLANT

**By Bob Mussetter, Mussetter Engineering, Inc., and
Yantao Cui, Hydraulic Engineer**

Sedimentation and resulting development of the bar at the mouth of Big Chico Creek causes problems with the current operation of the M&T Pumping Plant, and as a result, dredging of the bar has been used as a short-term solution. Because of the local hydraulic conditions caused by the current and likely future alignment of the river in this area, the bar will continue to reform and it may grow sufficiently during large storm events to completely bury the intake of the pumping plant. The magnitude of the problem was quantified by performing a hydraulic and sediment transport analyses using an existing one-dimensional, HEC-RAS model that was developed by the Sacramento District Corps of Engineers (Corps) and available hydrologic and sediment data.

1.0 Hydrology

The gaging station nearest to the project site is the Sacramento River near Hamilton City Station (USGS Gage No. 11383800), which is located at about RM 200, approximately seven miles upstream from the M&T pump station. Published data for this gage were used to develop a flood frequency curve for the study site based on Weibull plotting positions for the annual flood peaks for the post-Shasta Dam period (1946-2003). Peak discharges used in the analysis for the period prior to 1980 were obtained 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. The resulting flood frequency curve is presented in Figure IV-1, and the peak discharges associated with various return period events.

Peak Discharge (cfs)	Return Period (years)
59,300	1.20
72,000	1.50
90,000	2.00
129,700	5.00
148,600	10.00
179,500	50.00
374,100	100.00
<p>Table IV-1 Peak discharges and associated recurrence intervals derived from the flood frequency curve (Figure IV-1) at the Hamilton City gage.</p>	

The 1.5- and 2-year return period peak flows are 72,000 and 90,000 cfs, respectively, and as will be demonstrated in the next section, the average bankfull capacity of the channel in the M&T reach is between these values.

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 (1946 through 1980 from the USGS data, 1997-2000 and 2003 from the CDEC data) (Figure IV-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.

Annual runoff past the 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 IV-3) presents. The annual peak flows for the period of record are also shown in Figure IV-3 for comparison with the annual runoff volumes. The bar opposite Bidwell State Park probably 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 ft 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 tied to the occurrence of the high-magnitude flows.

2.0 Hydraulics

A one-dimensional hydraulic analysis of the project reach was conducted with an HEC-RAS model (USACOE 1999) developed by the Sacramento District Corps of Engineers for the Sacramento and San Joaquin River Basins Comprehensive Study. The portion of the river that was considered in this analysis (RM 187 to RM 195.25), and the locations of the cross sections are shown in Figure IV-4. The model was used to estimate the water-surface, velocity, and topwidth profiles for a range of flows between 20,000 and 200,000 cfs (Figures IV-5, IV-6, and IV-7), and these results were used to estimate the channel capacity in the M&T reach. The model results indicate that the channel capacity is about 90,000 cfs, and the bar at the mouth of Big Chico Creek is submerged between 30,000 and 35,000 cfs.

Figure IV-6 indicates that the velocities for the range of modeled flows are extremely low compared to those in the remainder of the reach at the nose of the bar at RM 193, just upstream from the M&T intact. At this location, the velocities do not exceed about 5 ft/sec over the range of modeled flows, whereas the velocities upstream of the nose of the bar at RM 193.25 and at RM193.75 exceed 10 fps and 14 fps, respectively at 200,000 cfs. As shown in Figure IV-7, the topwidth of the flow at the nose of the bar at RM 193 is significantly greater than the corresponding widths at the upstream cross sections (RM 193.25 and RM 193.75). The combination of low velocities and wide channel more or less ensure that sediment will be deposited and bars will be formed just upstream of the M&T pump inlets.

Bed-Load Transport in Sacramento River Near the M&T Pumping Plant

Return Period (years)

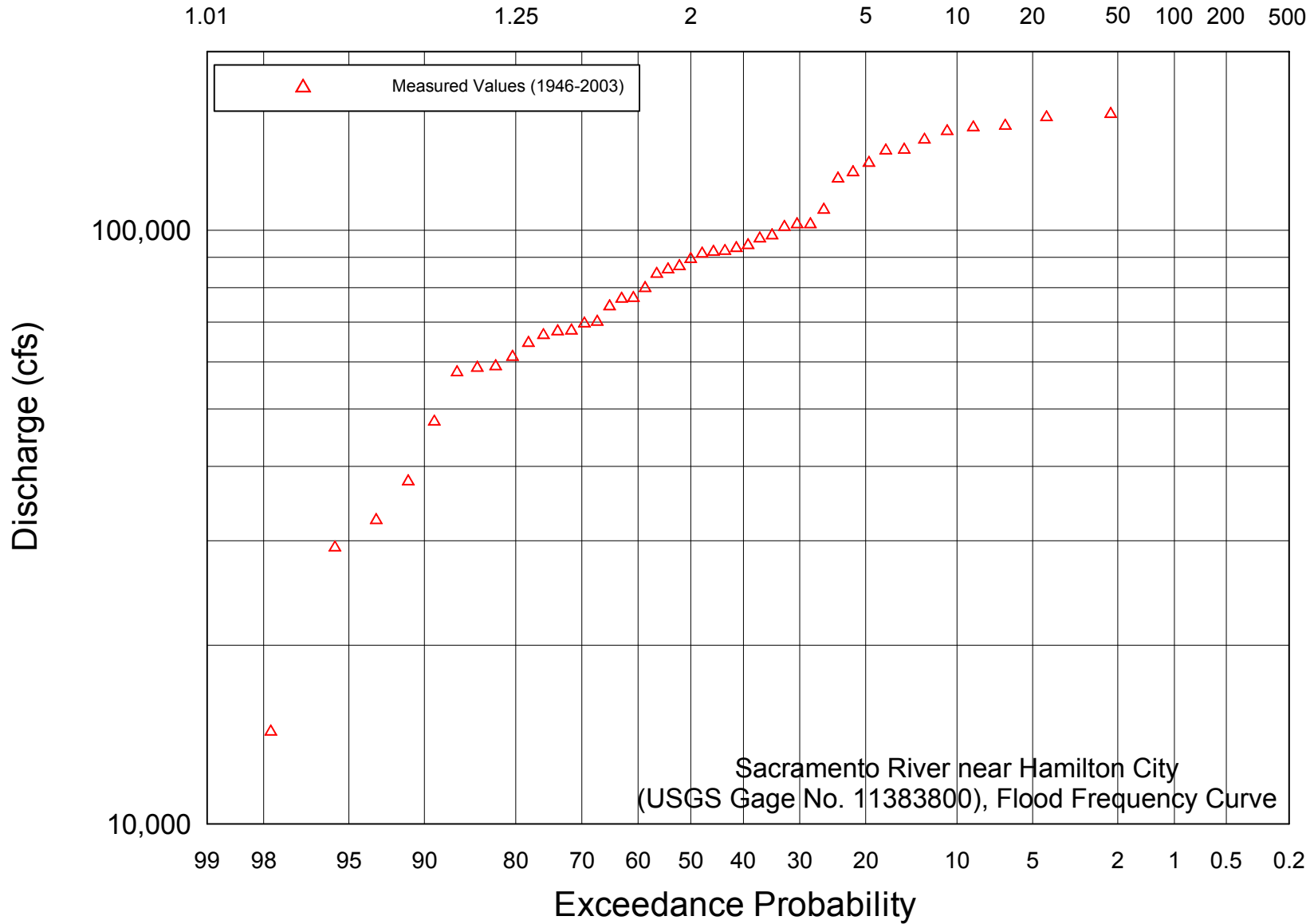


Figure IV-1 Post-Shasta Dam flood-frequency curve for Hamilton City gaging station.

Bed-Load Transport in Sacramento River Near the M&T Pumping Plant

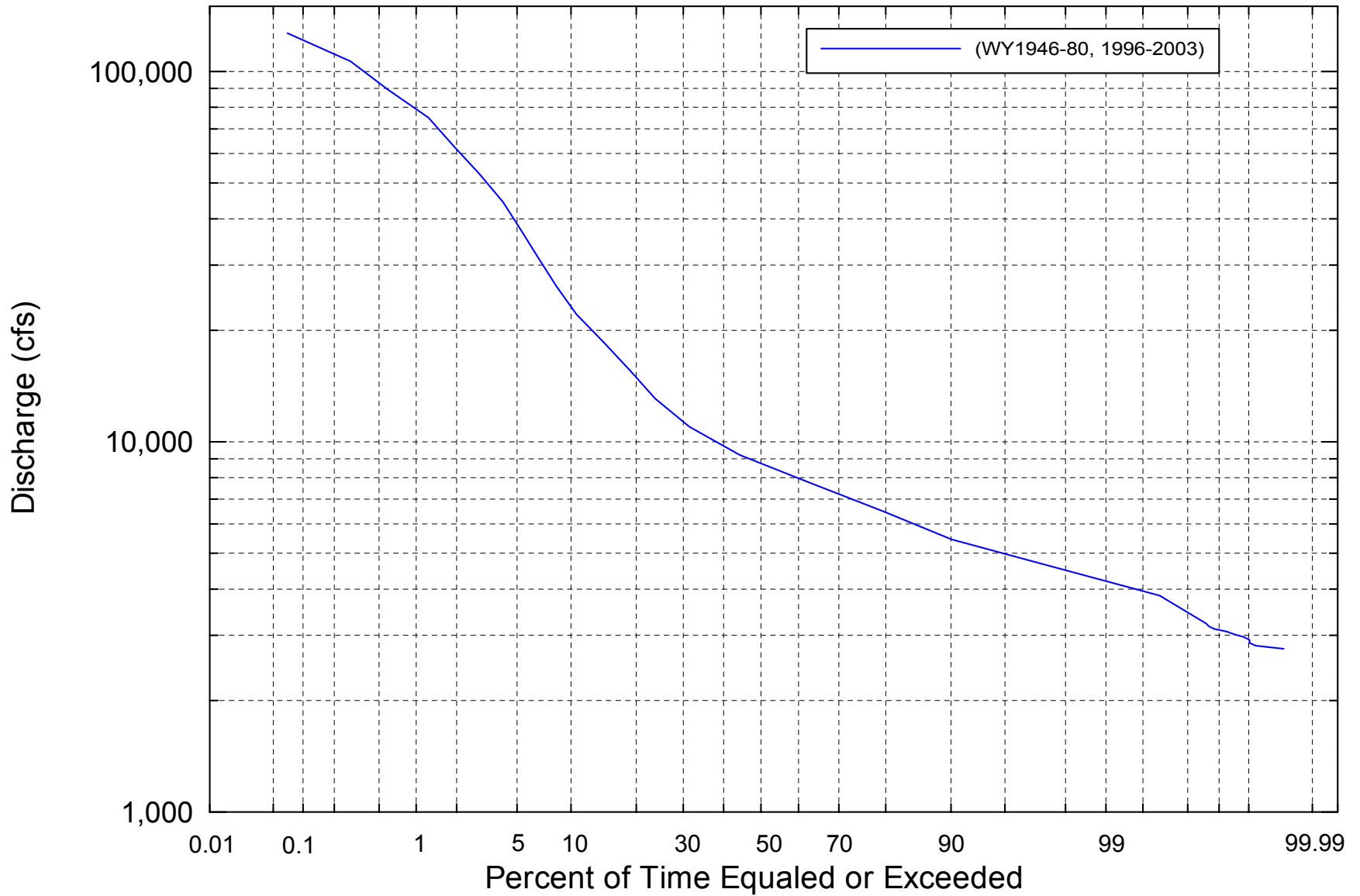


Figure IV-2 Post-Shasta flow-duration curve for Hamilton City gaging station.

Bed-Load Transport in Sacramento River Near the M&T Pumping Plant

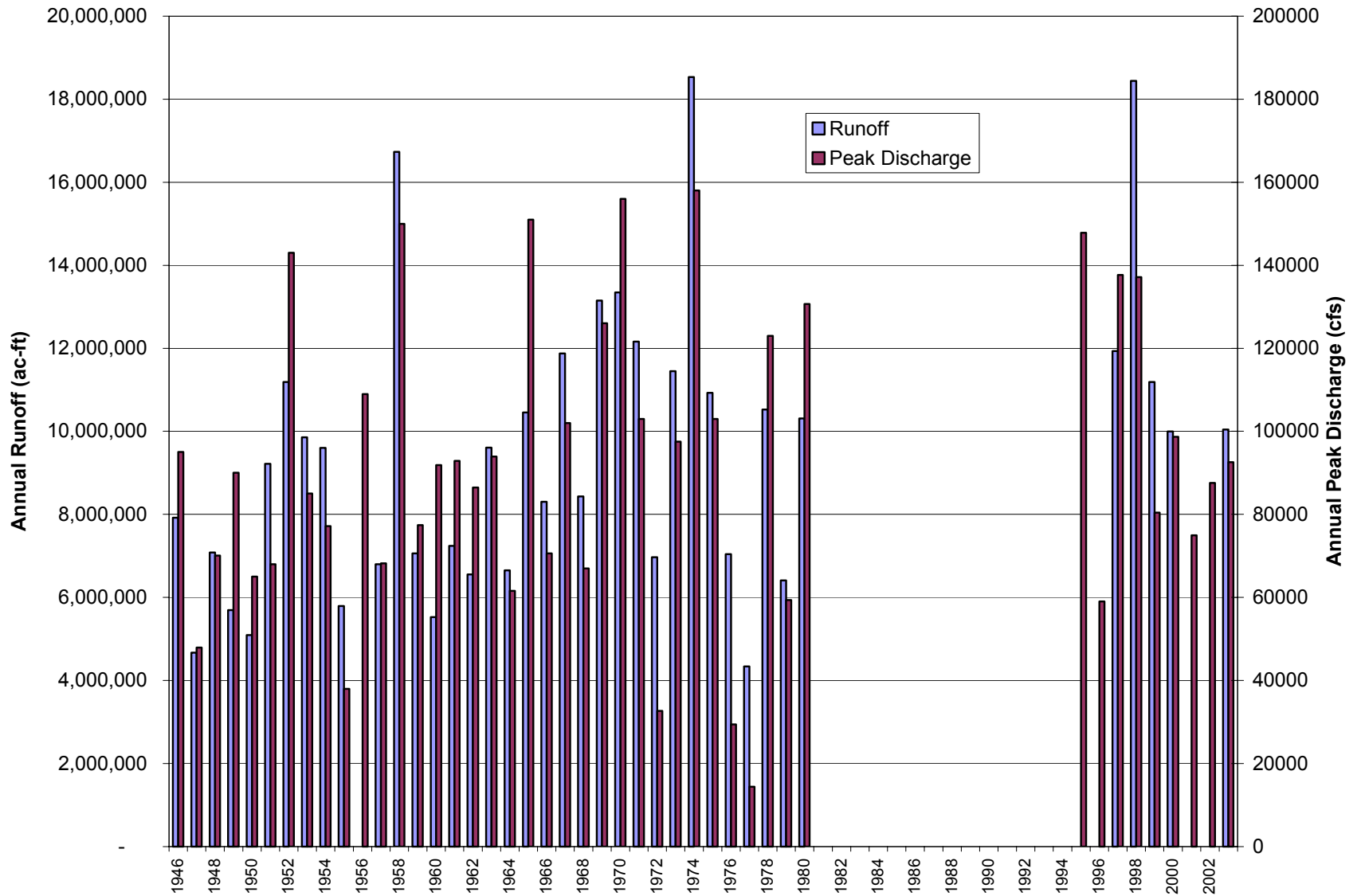


Figure IV-3 Annual runoff volumes and peak discharges at the Hamilton City gaging station from 1946 to 2003. No data are available for the 1981 to 1984 period.

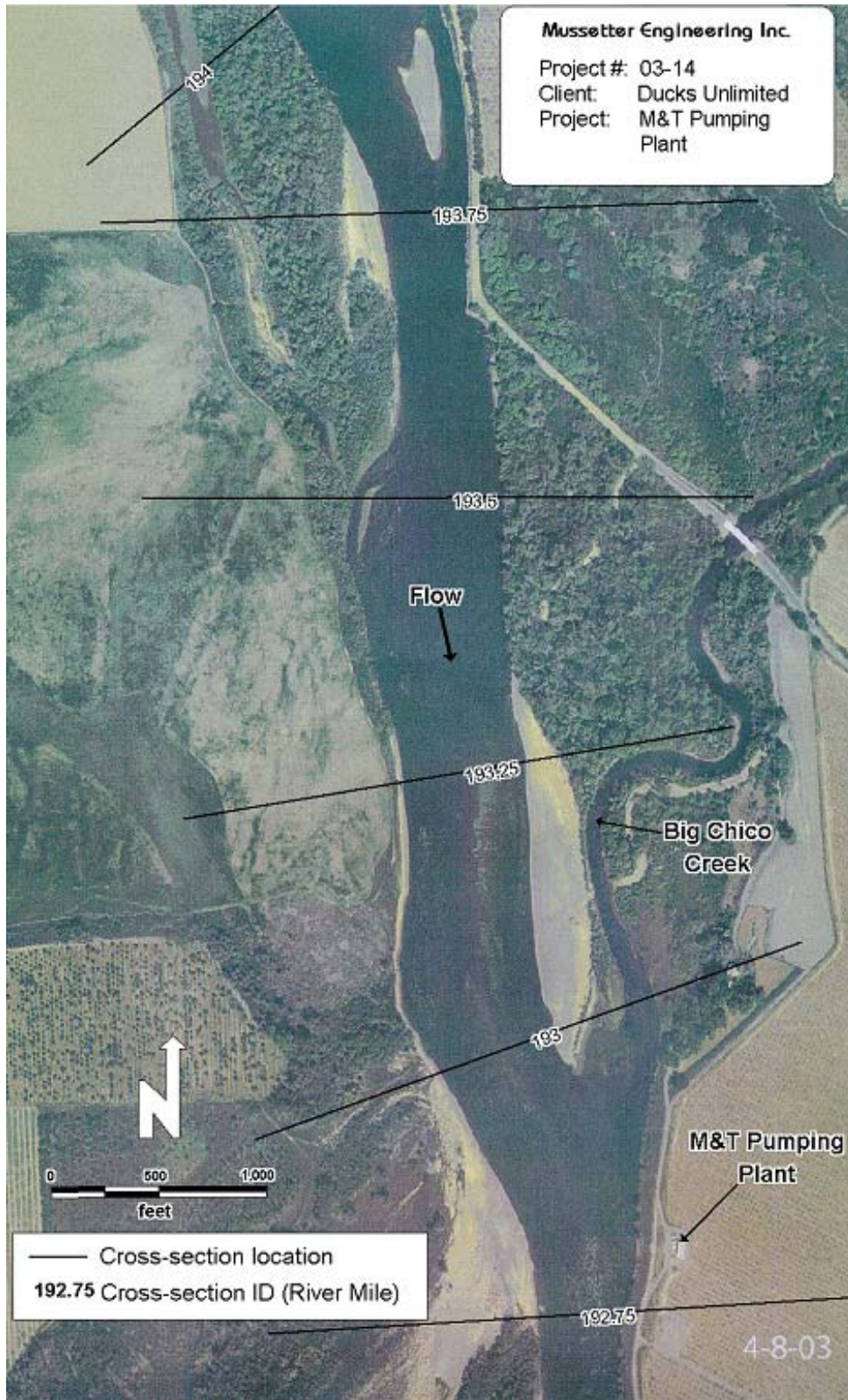


Figure IV-4 Aerial photograph showing the locations of the cross sections and other features within the HEC-RAS modeled reach.

Bed-Load Transport in Sacramento River Near the M&T Pumping Plant

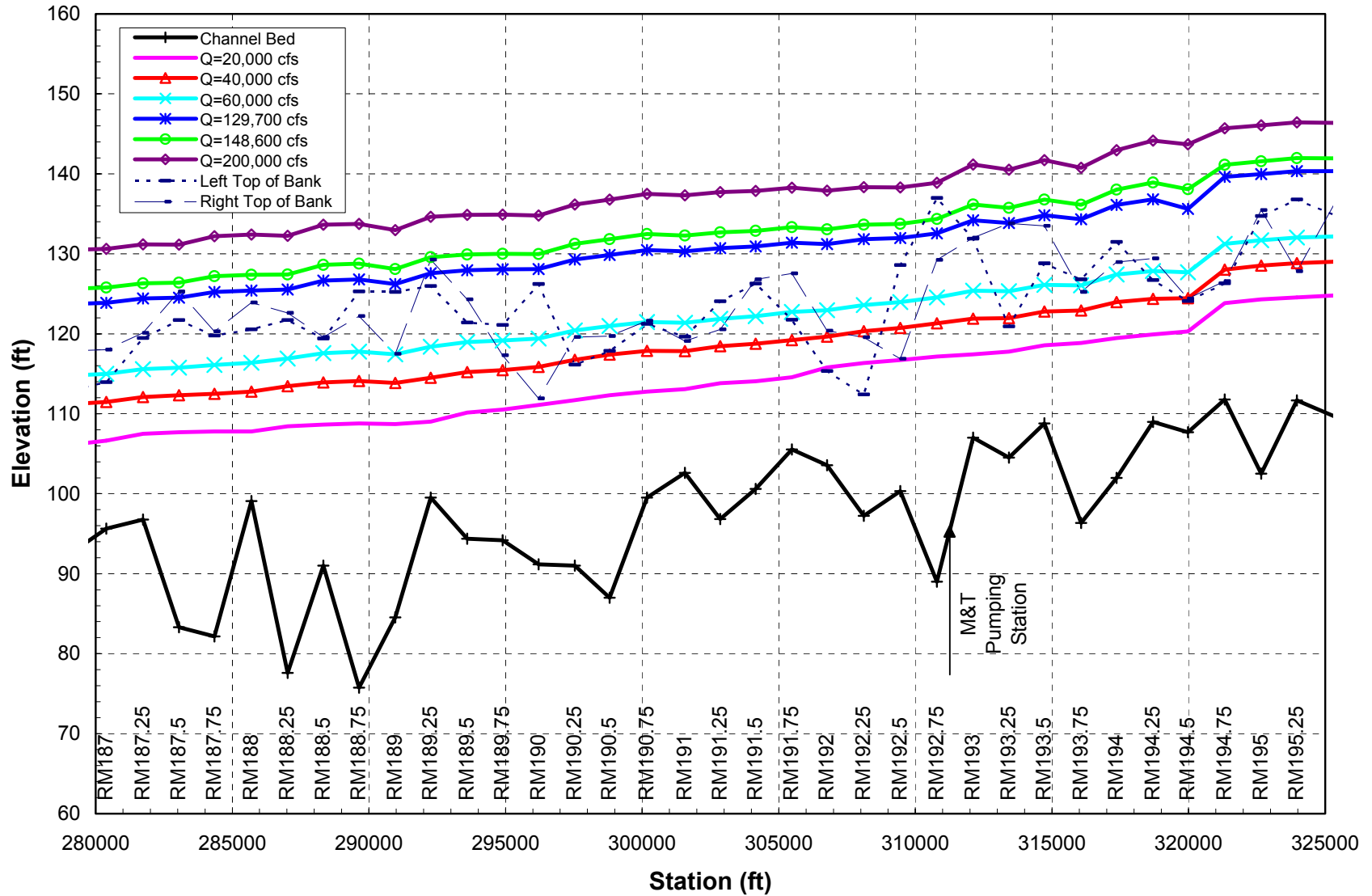


Figure IV-5 Water-surface profiles from RM 187 to RM 195.25 for a range of discharges between 20,000 and 200,000 cfs.

Bed-Load Transport in Sacramento River Near the M&T Pumping Plant

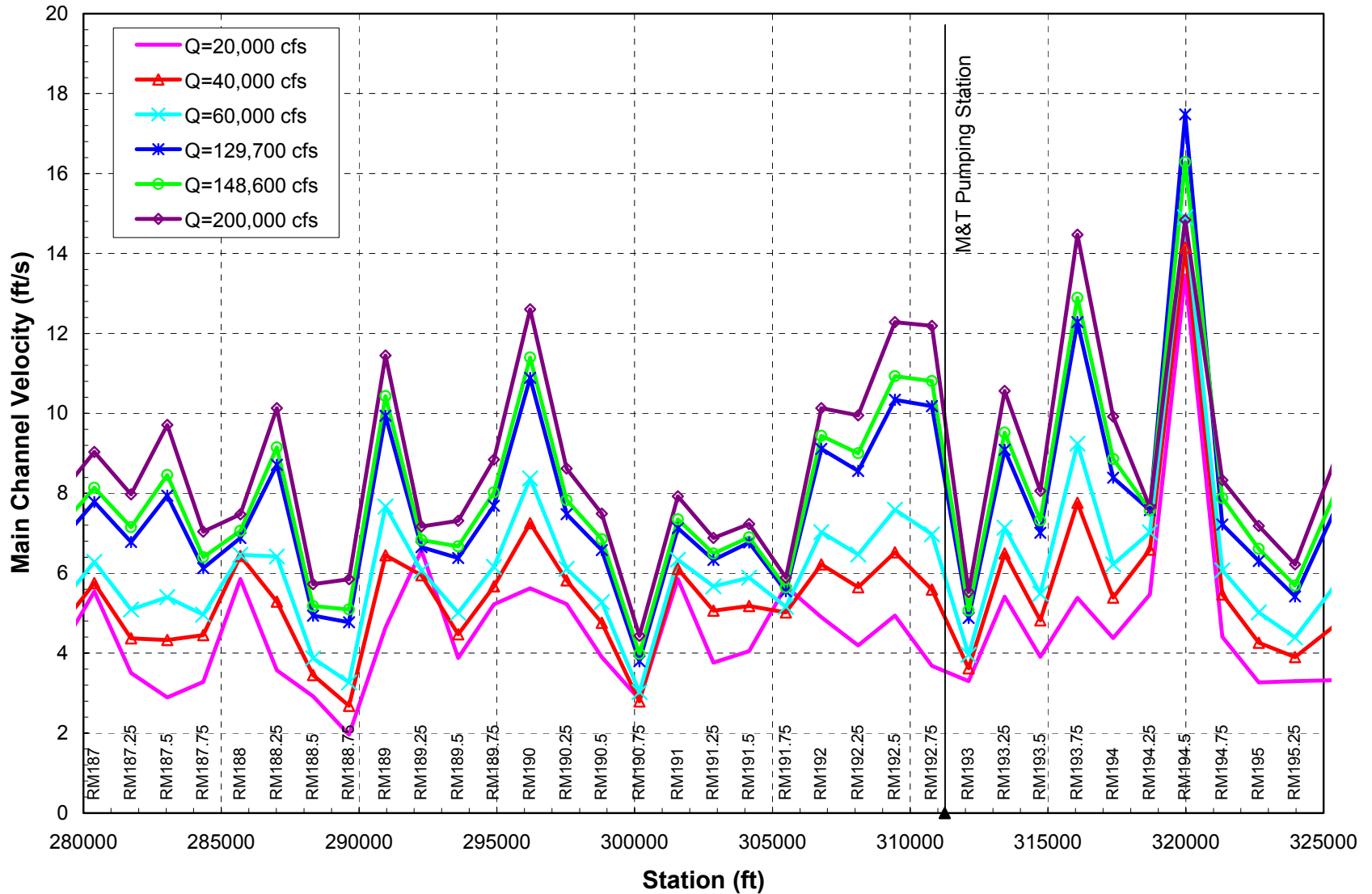


Figure IV-6 Velocity profiles from RM 187 to RM 195.25 for a range of discharges between 20,000 and 200,000 cfs.

Bed-Load Transport in Sacramento River Near the M&T Pumping Station

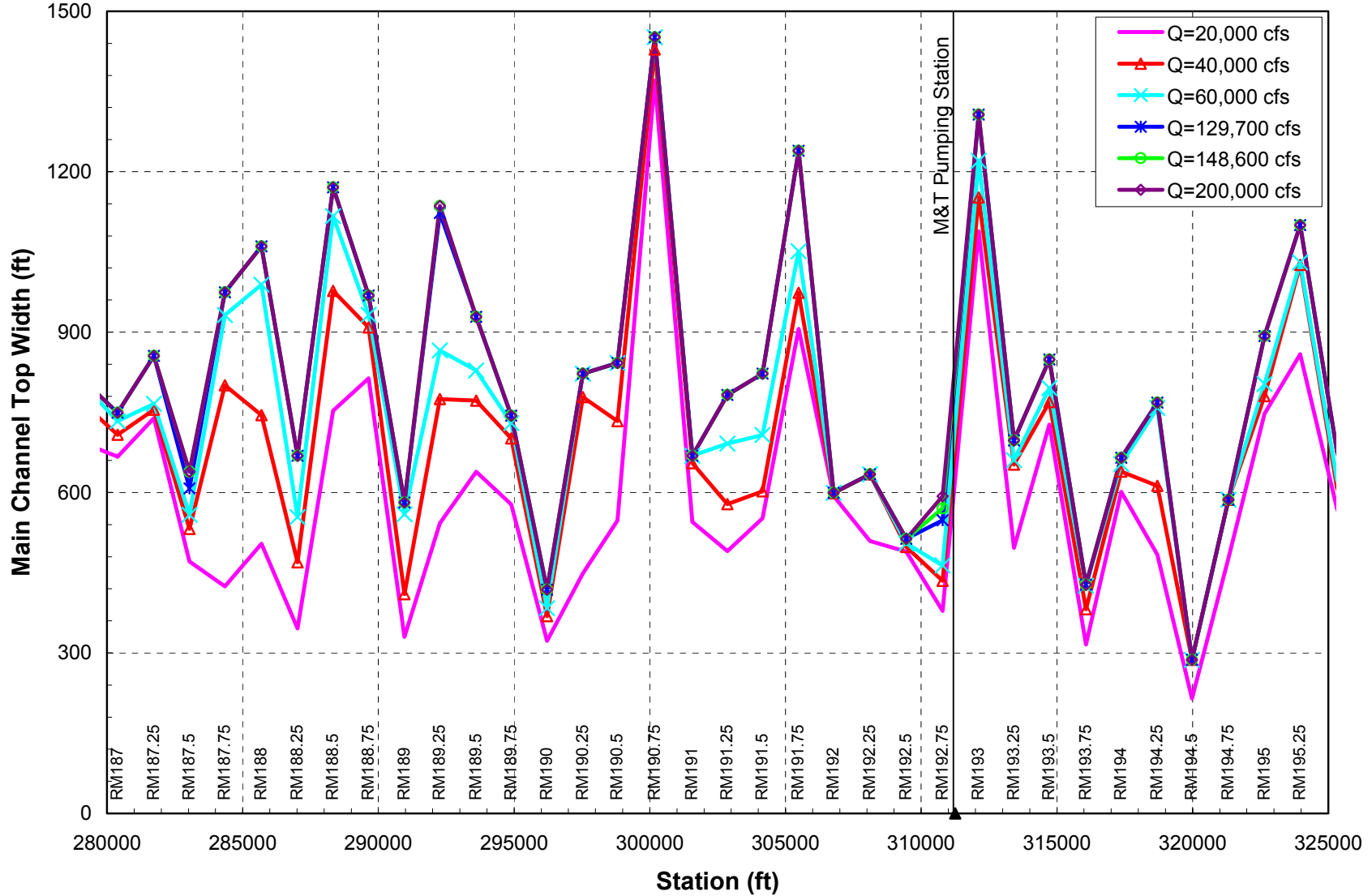


Figure IV-7 Main channel top width profiles from RM 187 to RM 195.25 for a range of discharges between 20,000 and 200,000 cfs.

3.0 Sediment Transport

The output from the HEC-RAS model was used to conduct both incipient-motion and sediment-transport calculations for the project reach. The incipient-motion analysis was conducted to estimate the range of discharges required to mobilize the bed material in the vicinity of the M&T site by evaluating the effective shear stress on the channel bed in relation to the amount of shear stress required to move the sediment that is present. The surface gradation for the bar materials was obtained from previous work that was conducted in the reach (WET 1990) that indicated that the D_{50} and the D_{84} of the surface sediments was 21.5 mm and 38 mm, respectively. The incipient condition was identified by computing the normalized grain shear stress (ϕ'), which is the ratio of the grain shear stress (τ') to the critical shear stress for particle mobilization (τ_c):

$$\phi' = \frac{\tau'}{\tau_c} \quad (1)$$

The grain shear stress was computed using the following formula:

$$\tau' = \gamma Y' S \quad (2)$$

where Y' is the portion of the total hydraulic depth associated with grain resistance (Einstein 1950) and S is the energy slope at the cross section.

The value of Y' is computed by iteratively solving the semilogarithmic velocity profile equation:

$$\frac{V}{V_*'} = 6.25 + 5.75 \log\left(\frac{Y'}{k_s}\right) \quad (3)$$

where V is the mean velocity at the cross section, k_s is the characteristic roughness of the bed, and V_*' is the shear velocity due to grain resistance, given by:

$$V_*' = \sqrt{g Y' S} \quad (4)$$

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

The critical shear stress (τ_c) is estimated using the Shields (1936) relation, given by:

$$\tau_c = \tau_{*c} (\gamma_s - \gamma) D_{50} \quad (6)$$

where τ_{*c} is the dimensionless critical shear stress [assumed to be 0.03 from the work of Neill (1968), Andrews (1984) and others], γ_s is the unit weight of sediment ($\sim 165 \text{ lb/ft}^3$), γ is the unit weight of water (62.4 lb/ft^3), and D_{50} is the median particle size of the bed material (21.5 mm). When the normalized shear stress (ϕ') is approximately 1, the bed begins to mobilize, and substantial transport of the bed material occurs when ϕ' exceeds about 1.5 (Mussetter et al. 2001).

Figure IV-8 shows the relationship between the normalized grain shear and the discharge for five cross sections that encompass the M&T site. At the downstream-most cross section (192.25), the bed begins to mobilize ($\phi' > 1$) at a discharge of about 115,000 cfs. Significant sediment transport ($\phi' > 1.5$) occurs only at discharges exceeding about 160,000 cfs (the 50-year flood peak). In contrast, immediately downstream from the M&T pumps (Cross Sections 192.5 and 192.75), significant transport begins at discharges between the 1.2- and 1.5-year recurrence interval peak flows (59,300 to 72,000 cfs). At the nose of the bar (Cross Section 193), neither incipient conditions nor significant sediment transport occurs within the range of modeled flows, indicating that deposition occurs immediately upstream of the pump inlets over the entire range of discharges that was analyzed. At Cross Section 193.25, which is located near the head of the bar, initial mobilization occurs at a discharge of about 20,000 cfs, and significant sediment transport occurs at about 60,000 cfs (1.2-year recurrence interval). The higher energy conditions at this cross section likely explain why the bar has moved downstream over time.

The volume of bed material-sized sediment that is transported by the bed mobilizing flows was estimated using the bar surface gradation ($D_{50} = 21.5$ mm) and the Parker (1990) surface-gradation-based bed-load equation. Because sediment sizes in the sand and finer size-range are excluded from the calculation and the bar material contains about 30 percent sand (WET, 1990), the computed sediment volumes were bulked by 30 percent to provide an estimate of the total bed material load.

The above procedures were used to estimate annual bed material transport volumes for 7 years from the available mean daily flow records in which the annual flood peak that ranged from the 1.2-year (1979) to 46-year (1974) recurrence interval flood peaks and had annual runoff volumes ranging from 6.4M ac-ft to 18.5 ac-ft (Table IV-2). The results that were obtained for five individual cross sections in the vicinity of the M&T intake, and using reach-averaged hydraulics for the reach are presented in Table IV-3.

Year	Peak Discharge (cfs)	Recurrence Intervals (years)	Annual Runoff Volume (M ac-ft)
1952	143,000	7.7	11.22
1962	86,500	1.8	6.55
1974	158,000	46.1	18.53
1978	123,000	4.2	10.53
1979	59,400	1.2	6.41
2000	98,000	2.5	10.10
2003	92,572	2.2	10.00
Table IV-2 Characteristics of typical water years.			

Bed-Load Transport in Sacramento River Near the M&T Pumping Plant

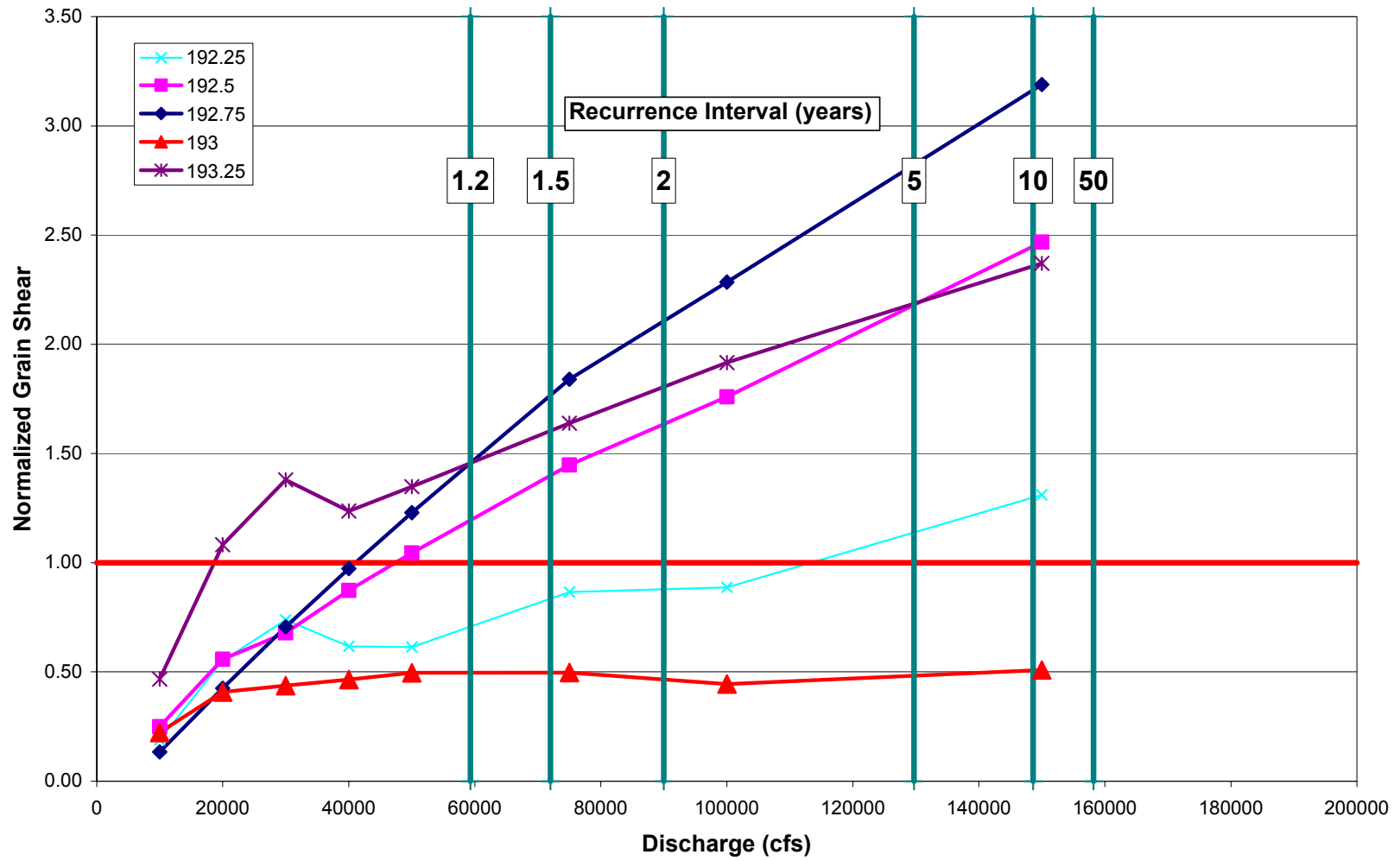


Figure IV-8 Normalized grain shear for five cross sections in the M&T reach for a range of flows between 20,000 and 160,000 cfs. The recurrence intervals for the 1.2- to 50-year peak discharges are also shown.

Cross Section	Sediment Transport (tons)						
	1952	1962	1974	1978	1979	2000	2003
192.25	1,428	451	42,664	10,324	41	2,601	454
192.50	22,796	12,677	324,906	111,866	1,814	65,828	17,361
192.75	8,370	2,506	228,045	61,639	43	15,115	1,278
193.00	0	0	0	2	0	0	0
193.25	22,534	9,324	188,219	64,102	2,342	42,496	16,234
Reach Average	1,203	471	30,736	7,849	50	2,627	530
Table IV-3 Computed total bed-material loads for identified water years.							

The long-term average annual sediment load was also estimated for the individual cross sections and for the reach as a whole by integrating the annual flow-duration curve (Table IV-4).

Cross Section	Sediment Transport (tons)
192.25	7,534
192.5	73,366
192.75	42,492
193.00	0
193.25	43,034
Reach Average	5,713
Table IV-4 Sediment-transport estimates based on integration of the annual flow-duration curve.	

On a reach-averaged basis, the annual transport varied from only about 50 tons in 1979 to about 30,800 tons in 1974, the year with the highest peak discharge and runoff volume. Consistent with the incipient motion results, the reach-averaged sediment-transport results in Table IV-3 illustrate that significant volumes of sediment are transported only during years having a high peak flow and total runoff volume, which answers the question as why there has not been significant sediment deposition observed over the past two years since the completion of the dredging operation. When the bar was dredged in 2000, approximately 189,000 tons of sediment were removed, which formed a sediment sink in the river that effectively intercepted most of the incoming sediment. Limitations on the current state-of-the-science knowledge in sediment transport theory prevent us from giving an accurate estimate as how long it will take to refill the dredged area. One way to look at it is based on the reach average sediment transport data in Table IV-3, according to which it will take about six years similar to 1974 (a 46-year event), or many more years for the relatively dry water years to replace the dredged volume. This simple analysis, however, is not accurate because the problem was created by the tendency for the gravel bar to migrate downstream and the tendency for the river to migrate, as suggested in Section II. Under this interpretation, the gravel bar migrates downstream even though the reach average sediment transport rate is not enough to replace the dredged volume. The other way to

look at the problem is to compare the sediment transport rates at the upstream and downstream most cross sections of the gravel bar, i.e. RM193.25 and RM193.0, respectively, and take the difference in transport between the two cross sections as the potential sediment deposit. This approach has its limitations as well, but the results in Tables IV-3 and IV-4 indicate that the 189,000 tons of sediment have the potential to be refilled within a single year similar to 1974 (a 46-year event), or about four years, on average.

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.

4.0 References

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