FINAL REPORT

AMERICAN AND SACRAMENTO RIVER, CALIFORNIA PROJECT

Geomorphic, Sediment Engineering, and Channel Stability Analyses

Prepared for

U.S. Army Corps of Engineers
Sacramento District
Sacramento, California

Contract No. DACW05-93-C-0045

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF PLATES

Following Page Number

1. INTRODUCTION

This report has been prepared under Contract No. DACW05-93-C-0045 for the Sacramento District, Corps of Engineers (COE). The American and Sacramento River, California Project, that is the subject of the report, is located in Sacramento, Placer, and El Dorado Counties, California (**Figure 1.1**). The purpose of the project is to increase the level of protection afforded by the downstream elements of the Sacramento River Flood Control Project (SRFCP) by construction of a flood control only, storage dam (dry dam) that is referred to as the Main Dam Element (MDE). Congress through the FY-1992 Defense Appropriations Act requested additional information on the project including potential modifications to the existing flood control levees along the Lower American River (Downstream Levees Element: DLE).

The **Main Dam Element (MDE)** project reach for this study extends from about River Mile (RM) 47.6 (location of proposed dry dam) on the North Fork of the American River upstream to about RM 72, which is the approximate elevation of the top of the dry dam (Elevation 930 ft) (Figure 1.1). The North Fork reach includes the North Fork Dam (RM 52.5) and Lake Clementine that were constructed by the California Debris Commission (CDC) in 1940 to retain hydraulic mining debris. The project reach also extends up the Middle Fork of the American River from the confluence of the North and Middle Forks at RM 50.3 to RM 74, which is also the approximate elevation of the top of the dry dam (Figure 1.1). For the purposes of this study the channel stationing for the Middle Fork commences at RM 47.6 at the dry dam site and not at the confluence because the Middle Fork is the primary source of sediment to the dry dam site since Lake Clementine has a 100 percent trap efficiency for bed material derived from the North Fork upstream of the reservoir. The specific objectives for the MDE study were the following:

1. Review and circumstantiation of the results of previous U.S.B.R. studies of sediment yield at the dry dam site

2. Determination of average annual and design event (200-year) sediment inflow to the dry dam site

3. Determination of sediment accumulation and its distribution along the project reach and at the dry dam site during the project life

4. Determination of the potential for coarse bed material to enter the flood control sluices and/or diversion tunnel

5. Development of conceptual design for structural measures to prevent coarse bed material from entering flood control sluices and the diversion tunnel.

The **Downstream Levees Element (DLE)** project reach for this study extends from the confluence of the Lower American River with the Sacramento River (RM 0) upstream to Nimbus Dam at RM 23 (Figure 1.1). Sacramento River Flood Control Project (SRFCP) levees extend on the north (right) bank from the confluence with the Sacramento River to about RM 6. American River Project levees extend from RM 6 to RM 14. On the south (left) bank the SRFCP extend from the confluence to RM 12. Local levees are located from about RM 13 to RM 15 on the south bank. The specific objectives for the DLE study were:

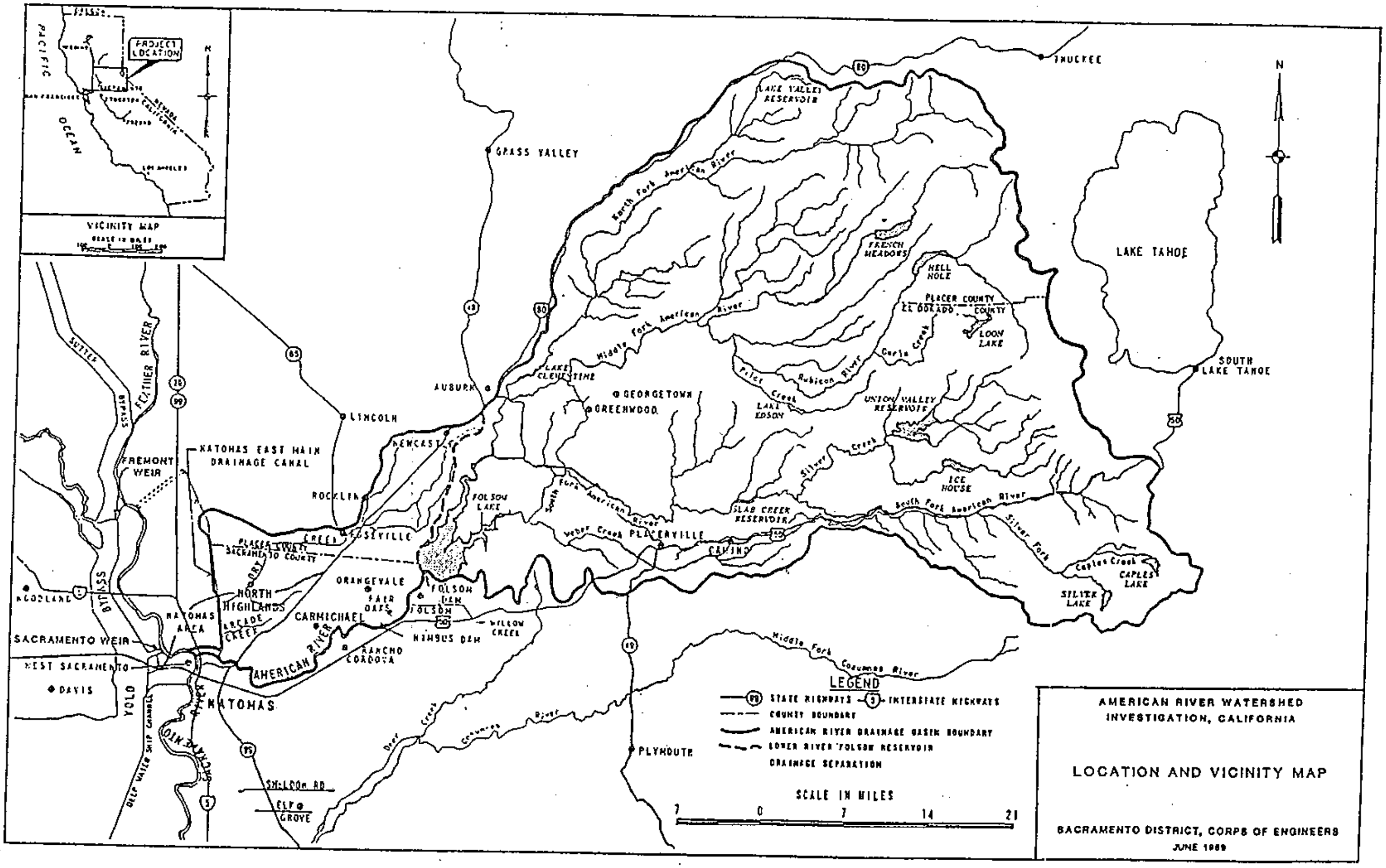


Figure 1.1. Map of American River watershed (COE, 1989).

1. To perform reconnaissance/feasibility-level geomorphic and sediment engineering investigations with respect to concerns regarding channel stability, bank protection requirements, and other sediment engineering concerns for objective releases from Folsom Dam of 115,000, 145,000, and 180,000 cfs

2. To identify potential problem areas with respect to lateral and vertical channel stability at each of the objective releases and identify potential mitigation measures for any such potential problems

3. To perform a comprehensive technical review of the American River Watershed Investigation California (ARWI) Feasibility Report (COE,1991) and evaluate the validity of comments in a review report of the AWRI Feasibility Report prepared for the Sacramento Area Flood Control Agency (SAFCA) by WRC-Environmental (WRCE) and Mitchell Swanson and Associates (MSA) (WRCE/MSA 1992)

4. To determine project features required to safely convey the three objective releases in the study reach based on the geomorphic, sediment engineering, channel stability analyses and geotechnical analyses of levee stability

5. To develop recommendations for future technical studies, (including geomorphic, hydraulic and geotechnical) to be performed during Preconstruction Engineering and Design (PED) to address concerns raised by this study.

1.1. Authorization

This investigation by RCE/Ayres Associates was authorized under the Flood Control Act of 1962 (Sacramento River Flood Control Project). The study was performed for the Metropolitan Sacramento (California) Area element of the SRFCP that includes the American and Sacramento River, California Project. The Technical Managers (TM) for the American and Sacramento River, California Project were Rick Johnson and Rich Nishio. The Senior Project Manager for this project was Bob Childs. The Sacramento District Project Manager for this contract was Ed Sing of the Hydraulic Design Section. At RCE/Ayres, Michael Harvey was Principal Geomorphologist and Project Manager. Robert Mussetter served as Principal Hydraulic Engineer. Gary Wolff and Lyle Zevenbergen served as Senior Hydraulic Engineers. Technical review of the work products was performed by Mark Peterson.

1.2. Field Investigation and Data Collection

Field investigations and data collection efforts in both the Lower American River (DLE) and North and Middle Forks American River (MDE) reaches were conducted by staff of Resource Consultants & Engineers and Meridian Consulting Engineers (topographic surveying) in May of 1993. The Middle Fork of the American River was rafted between Indian Bar at RM 74 and the proposed dry dam site at RM 47.6. Sediment samples were collected during the 2-day raft trip at a discharge of about 1,500 cfs, and observations were recorded regarding sediment sources slope stability and other factors that would affect downstream sediment delivery. Based on the field observations a detailed data collection effort was mounted at Mammoth Bar. This included detailed topographic mapping of the reach and intensive sediment sampling that would permit subsequent detailed hydraulic and sedimentologic analyses of the effects of local bedrock contractions on sediment storage and downstream delivery. A vehicle-based reconnaissance of the watershed of the North Fork of the American River was also conducted. Based on this effort a sedimentation survey that included 10 cross sections and sediment sampling of Lake Clementine was conducted to provide data on watershed sedimentation rates in a watershed that had been severely impacted by hydraulic mining.

The Lower American River study reach was traversed by jetboat, and sediment samples were collected at sites where the field evidence indicated that bed material had been deposited during early 1993 flows of up to 16,200 cfs. Field data on sites of bank erosion and bank protection collected previously (WET 1991) were checked for accuracy, and modifications and additions were made if required. Ten cross sections were surveyed across the channel and floodplain at locations where sections had been previously surveyed in 1987. Within the leveed reach, cross sections extended from levee to levee. Because of lack of adequate monumentation and other problems the cross sections within the leveed reach could not be recovered exactly.

1.3. Data Sources

A major source of data and information for this study was the American River Watershed Investigation California (ARWI) Feasibility Report (1991), Appendices I, J, K, L, and M. A previous geomorphic investigation of the Lower American River (Water Engineering & Technology, Inc. 1991) provided considerable data and background information for that portion of the study. Sacramento District map files provided historic surveys (1935/36) of the North and Middle Forks of the American River. Murray, Burns, and Kienlen provided historic thalweg profiles of the Lower American River. Mitchell Swanson and Associates provided bank erosion rate data for the Lower American River.

The Sacramento District provided hydrologic data for both the Main Dam Element and the Downstream Levees Element. Hydrographs for the five design scenarios for the DLE included the Existing Condition (400,000 ac-ft flood control storage [FCS] in Folsom Dam and an Objective Release [OR] of 115,000 cfs), FEMA 100-year (400,000 ac-ft FCS, 145,000 CFS OR), FEMA 100-year (590,000 ac-ft FCS, 115,000 cfs OR), 125-year (650,000 ac-ft FCS, 180,000 cfs OR), Recommended Project (dry dam 400,000 ac-ft FCS, 115,000 cfs OR). The scenarios were developed to bracket the range of potential impacts of the project on the river's geomorphic, sediment transport, and channel stability characteristics. An HEC-2 deck for the Lower American River was provided by the District as were downstream stage rating curves. For the MDE, total inflow (combined Middle and North Forks) and outflow hydrographs and a reservoir pool stage hydrograph for the design storm were provided by the COE. In addition, a single-valued, stage-discharge rating curve for the proposed sluice configuration was provided for use in establishing pool elevation for flows other than the design storm. An HEC-2 deck was generated for the North Fork and Middle Fork using available topographic data. A detailed HEC-2 deck was constructed for the Mammoth Bar site from topographic surveys conducted during this project.

1.4. Data Analyses

Geomorphic, sedimentologic, and geologic data were evaluated and analyzed for the MDE. Reviews of pertinent engineering and geomorphic literature were carried out for steepland channel dynamics and sediment impacts on flood control sluice and/or diversions. Hydraulic analysis was conducted with an HEC-2 model and sediment transport analyses were conducted with an RCE/Ayres-modified (Sacramento District) version of HEC-6 (RCE/Ayres 1993). Analyses were conducted for a baseline condition with no dam and for the dry dam condition. The geomorphology of the DLE was updated from that compiled by WET (1991). Hydraulic analyses were conducted with an HEC-2 model and sediment transport analyses were conducted with the RCE/Ayres-modified version of HEC-6. A total work analysis was conducted at 13 eroding bank sites identified in the leveed section of the Lower American River, and at nine sites that had been previously protected with the objective of developing a quantitative methodology for ranking bank protection sites.

1.5. Report Organization

The report is comprised of five chapters. Chapter 1 is an **Introduction**. Chapter 2 discusses the **American River Basin Geology and Geomorphology**. Chapter 3 presents the analyses and results of the **Main Dam Element** investigation. The **Downstream Levees Element** analyses and results are presented in Chapter 4. Chapter 5 presents the **Summary and Recommendations**. The review of the WRCE/MSA report and peak discharge water surface profiles for the five investigate scenarios for the DLE are contained in an **Appendix**. The report is accompanied by a set of plan sheets that are based on the 1986 aerial photography of the DLE reach and include the locations of sites of bank erosion, existing bank protection, sediment sampling locations, locations of surveyed cross sections and geologic features that have effects on the channel dynamics.

2. AMERICAN RIVER BASIN GEOLOGY AND GEOMORPHOLOGY

The American River drainage basin is approximately 1,860 sq-mi in extent. The drainage basin consists of three major subbasins: the North, Middle, and South Forks (**Figure 2.1**). The North and Middle Forks are the primary focus of this study. **Table 2.1** shows basin geomorphic characteristics for each of the major American River subbasins.

Upstream of Folsom, the three main stems of the American River flow through steep canyons of the Sierra Nevada. The tributaries join in the Sierran foothills in the vicinity of Folsom Lake. Downstream of Folsom dam, the river enters the Great Valley of California, flowing westward within a relatively broad floodplain to its confluence with the Sacramento River in the City of Sacramento. Within this reach, the American River is locally bounded by high bluffs, flood control levees to the north, and flood control levees to the south.

Because of the dramatic change in physical character between the Sierra Nevada and the Sacramento Valley geomorphic provinces, the geology and geomorphology of the two regions are discussed separately. As numerous references are made to geologic time in the discussions, a geologic time scale is included as **Figure 2.2**.

2.1. Geology and Geologic History of the American River Basin Upstream of Folsom Dam

2.1.1. Basement Rocks

The Sierra Nevada is a strongly asymmetric mountain range that has a gentle western slope and steep eastern escarpment. The range is 40 to 100 mi wide, extending from the Mojave Desert northward to the Klamath Mountains, over 400 mi (**Figure 2.3**). The southern half as well as the eastern portion of the northern half of the range are composed of plutonic rocks of Mesozoic age. These are the rocks of the Sierra Nevada batholith, which are primarily granites. In the northern portion of the range, the plutonic rocks are bounded to the west by deformed and metamorphosed sedimentary rocks of Paleozoic and Mesozoic age, which are commonly referred to as the western metamorphic belt (Bateman and Wahrhaftig 1966). The famous Mother Lode of California passes through this belt.

The entire western Sierra, including the American River drainage basin, is underlain primarily by steeply dipping metamorphic rocks of the western metamorphic belt. The metamorphism throughout most of the region is low grade (Clark 1960). Ultramafic bodies are present as elongate features along or within major fault zones and intrude the Paleozoic and Mesozoic age metamorphic rocks. The most abundant ultramafic rock type is serpentine. These units have been in turn intruded by numerous isolated granitic plutons. The small plutons are significantly more mafic (gabbro to quartz diorite) than the Sierran batholith (granodiorite to granite), which is located to the east of the project area. These mafic plutons lie to the west of and are older than the Sierra Nevada batholith (Clark 1960). The North Fork of the American River is underlain by quartz diorite plutons between Pilot Hill and Fair Oaks (Penryn Pluton and Rocklin Pluton; Olmstead 1971).

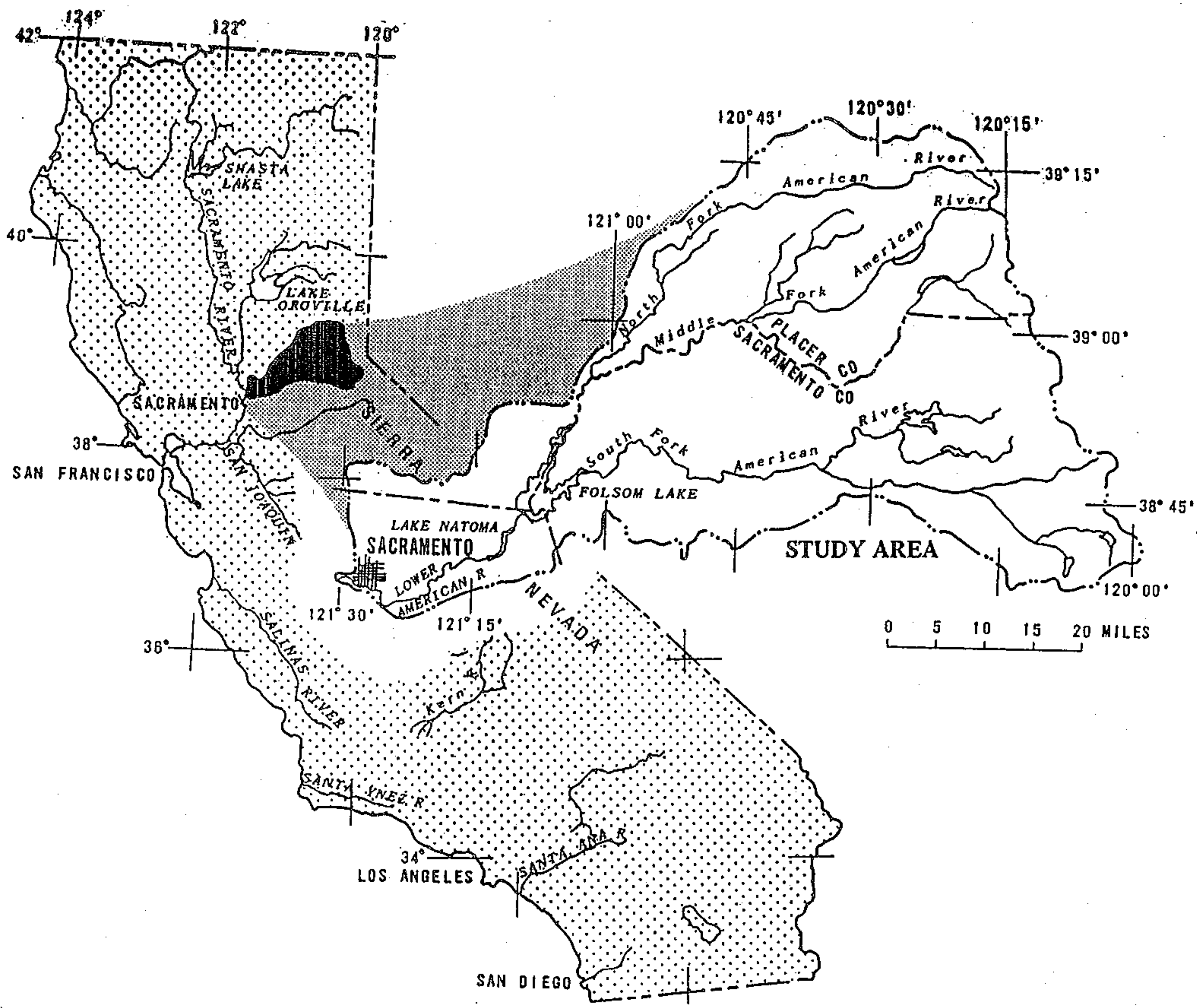


Figure 2.1. Index map of California showing the location of the American River basin and the major forks of the river (Shulters, 1982).

Table 2.1. Basin Characteristic Statistics for the American River Basin Upstream from the North Fork and South Fork Confluence Near Folsom Dam (after Shulters 1982).

Segment	Drainage Area (mi^2) (1)	Altitude (ft)		Basin Order (3)	Length of Streams (mi)		Drainage Density (mi/mi^2) (5)
		Low (2)	High (2)		Main Stem (4)	Total (4)	
North Fork to confluence with South Fork	1,012	240	9,000	7	94	1,130	1.1
North Fork to confluence with Middle Fork	396	520	9,000	7	70	500	1.3
Middle Fork to confluence with North Fork	616	520	9,000	6	62	570	.9
South Fork to confluence with North Fork	848	240	10,380	5	86	680	.8
TOTAL	1,860	240	10,380	7	242	1,810	1.0

(1) Drainage Area. The area of a river basin, measured in a horizontal plane, that is enclosed by a topographic divide, such that direct surface runoff from precipitation normally would drain by gravity into the river basin.

(2) Altitude. The low altitude is that elevation, in feet above National Geodetic Vertical Datum of 1929, where the stream exits the drainage basin. The high altitude represents that elevation that is the highest point on the drainage basin perimeter.

(3) Basin Order. Same as the highest stream order in the drainage basin. First-order streams have not tributaries; second-order streams have only first-order streams as tributaries and so forth.

(4) Stream Length. The main stem length is measured from the outlet of the basin to the basin divide following that fork with the largest drainage area. The total stream length is computed by measuring and summing with the main stem length all of the lower ordered tributaries in the basin.

(5) Drainage Density. Stream length per unit area. Determined by dividing the total stream length by the drainage area.

GEOLOGIC TIME SCALE

EON	ERA	SUB-ERA PERIOD SUB-PERIOD		EPOCH		AGE (MILLIONS OF YEARS) BP
PHANEROZOIC	CENOZOIC	Quaternary		Holocene		0.01
		Quaternary		Pleistocene		2.0
		Tertiary	Neogene	Pliocene	M E	5.1
				Miocene	L	11.3
					M	14.4
					E	24.6
			Paleogene	Oligocene	M E	38.0
				Eocene	L	42.0
					M	50.5
					E	54.9
				Paleocene	M E	65.0
	MESOZOIC	Cretaceous		Late K_2	Senonian	88.5
						97.5
				Early K_1		125
					Neocomina	144
		Jurassic		J_3	Malm	163
				J_2	Dogger	188
				J_1	Lias	213

Figure 2.2. Geologic time scale used in referencing ages of geologic units and events.

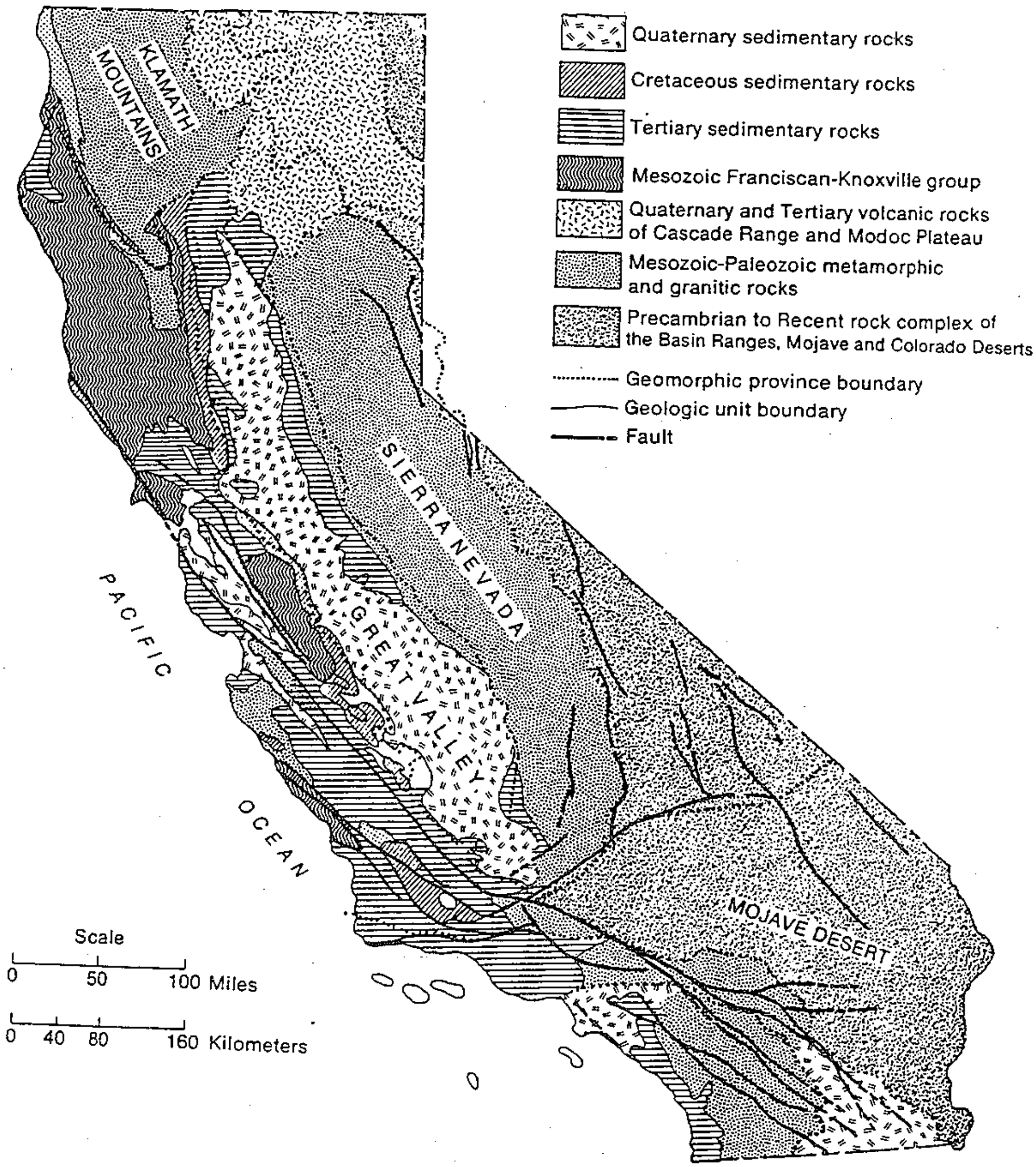


Figure 2.3. Geologic map of California, showing principal faults and generalized geologic units. (California Division of Mines and Geology, Norris and Webb 1990).

The metamorphic rocks that underlie the project area are separated from the Sierran batholic rocks by a major fault zone (Melones Fault Zone). Metamorphic rocks west of the fault zone are so unlike those east of the fault that they are thought to be exotic blocks that were attached to North America in early Triassic time (Norris and Webb 1990).

2.1.2. Cenozoic Rocks

Tertiary-age sediments in the western Sierra Nevada consist of the Eocene-age Ione formation as well as younger gold-bearing gravels that were hydraulically mined. The Ione Formation rests on a deeply weathered surface of crystalline basement rocks that displays over 1,000 ft of local relief (Bateman and Wahrhaftig 1966). The Ione Formation consists of sandstone, claystone, and lignitic coal, which were deposited under deltaic and lagoonal conditions. The maximum thickness of the Ione Formation is approximately 500 ft. The Ione Formation is exposed on the eastern margin of the Sacramento Valley, in the vicinity of Folsom (**Figure 2.4**).

Auriferous Gravels

The gold-bearing (auriferous) Tertiary-age gravels of the western Sierra Nevada were deposited prior to and during a major period of volcanic activity in the Sierra Nevada. The prevolcanic gravels are exposed as sinuous remnants on relatively flat interfluves among deep canyons in the Sierra Nevada. These gravel deposits comprise a pattern of five major drainages that have been named according to their modern analogs: the Tertiary Yuba, American, Mokelumne, Calaveras, and Tuolumne Rivers. The largest of these streams was the Tertiary Yuba River, which extended southward into the modern American River drainage basin (Figure 2.4). The Tertiary Yuba River deposits reach thicknesses of 600 ft. In contrast, prevolcanic gravels on the other drainages generally do not exceed 50 ft in thickness. Stratigraphic data indicate that the prevolcanic gravels were deposited in Eocene and early Oligocene time.

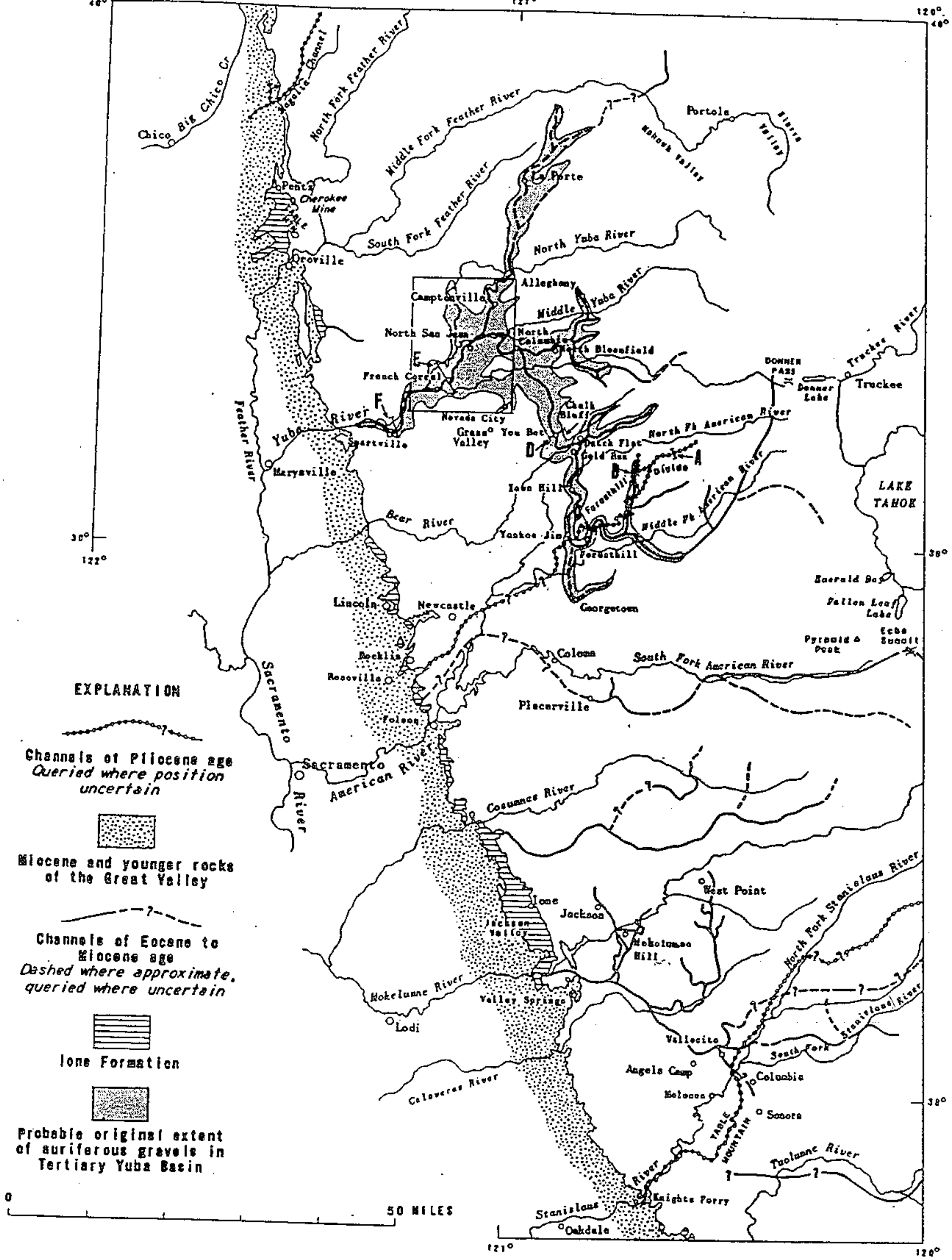


Figure 2.4. Map shows localities in northern Sierra Nevada mentioned in text, prevolcanic and intervolcanic channels, Ione Formation, and inferred original distribution of auriferous gravels in the basin of the Tertiary Yuba River. Channels are from Gold Belt Folios, Lindgren 1911, Plate 1, San Jose sheet (prelim. comp.) of Geologic Map of California, Olaf P. Jenkins edition (Bateman and Wahrhftig 1966).

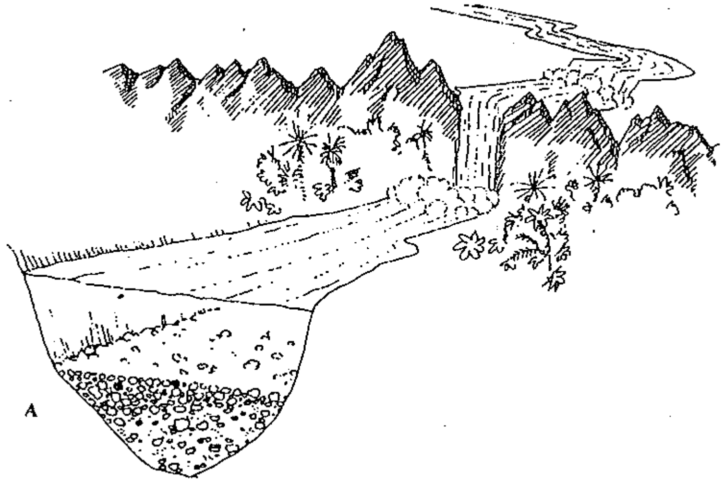
Vulcanism in the region began approximately 30 million yr ago, in mid-Oligocene time. Between 20 and 30 million yr ago, rhyolite tuffs, ash flows, and associated clastic rocks infilled narrow valleys. Creating new courses and stream canyons (**Figure 2.5**), streams reworked and cut through the volcanic deposits. From 5 to 19 million yr ago (Miocene and Pliocene time), extensive andesitic mudflows and conglomerates completely buried the pre-existing terrain. A new drainage network developed during Miocene and Pliocene time that was controlled by the distribution of the andesitic deposits (Figure 2.5). Nonmarine gravels deposited during this time period record the development of modern drainages and have been referred to as intervolcanic gravels. The Pliocene-age intervolcanic channel of the Forest Hill Divide records the diversion of the South Fork of the Tertiary Yuba River into the drainage of the American River (Figure 2.4; Bateman and Wahrhaftig 1966). The Forest Hill Divide channel, mined over much of its length was probably reworked from older prevolcanic channels. Many of the drainage divides in the American River basin are capped by volcanic rocks that are remnants of the once-continuous volcanic plain (Shulters 1982). **Figure 2.6** shows the Miocene-Pliocene erosional surface in the northern Sierra Nevada.

Uplift and tilting of the Sierra Nevada resulted in deepening of channels to levels well below the earlier channels. In certain areas, much of the volcanic rock was eroded and prevolcanic topography exhumed. Tilting probably began with the onset of vulcanism, as the intervolcanic gravels are found in steep narrow canyons and contain andesitic clasts.

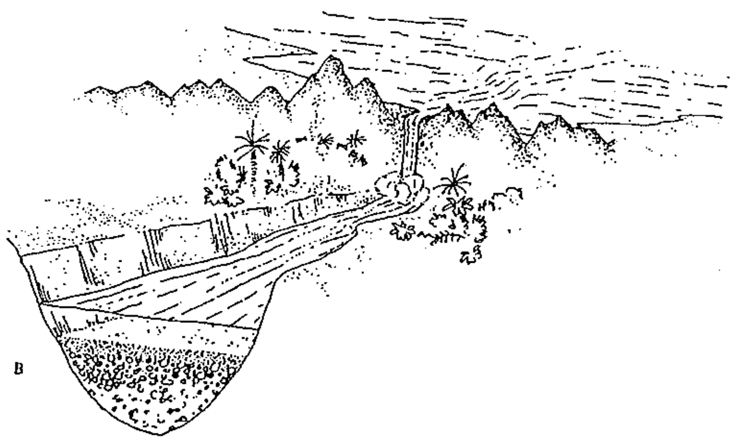
2.1.3. Faulting

The Foothills fault system is exposed in an area 200 mi long and 30 mi wide within the Sierran Foothills (**Figure 2.7**). The faults are consistently steeply dipping (Clark 1960). Several faults of the Foothills fault system cross the American River. The major fault zones that cross the project area are the Melones and Bear Mountain fault zones. Both fault zones are characterized by linear exposures of serpentinite and other sheared metamorphic rocks (Norris and Webb 1990). The faults are well-defined south of Placerville, but in the study area, both systems divide into a complex series of smaller faults. According to Clark (1960), most of the tectonic activity on the Foothills fault system occurred by Cretaceous time.

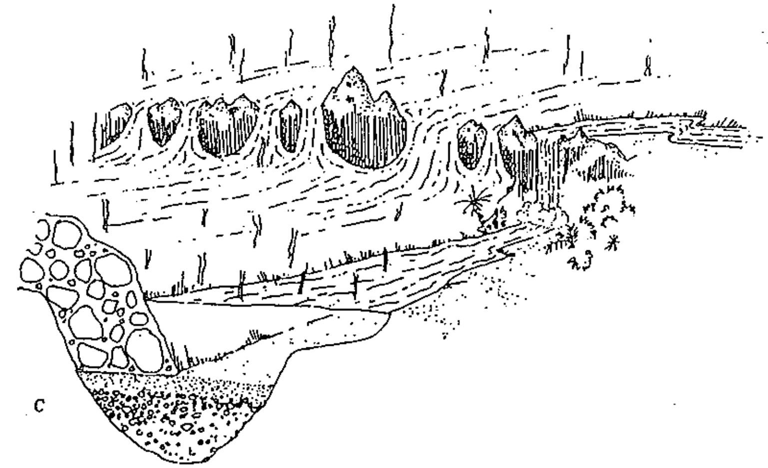
The Melones fault zone is the most prominent of the Sierran foothills faults (Norris and Webb 1990). This fault zone marks the structural boundary between Sierran rocks to the east and presumed exotic terrains of metamorphic rocks that were attached to the North American continent during Triassic time. The western branch of the Melones fault as described by Clark (1960) crosses the Middle Fork near Spanish Dry Diggings and the North Fork due east of Applegate (**Figure 2.8**). An exposure of this fault is located near the North Fork of the American River approximately 7 mi southeast of Colfax. The eastern branch of the Melones fault zone bifurcates as it crosses the American River drainage basin (Figure 2.8). The two faults cross the Middle Fork near Volcanoville, and the North Fork south of Monte Vista. South of Placerville, the Melones fault system has juxtaposed Jurassic age rocks on the west against Paleozoic age rocks on the east over a distance of over 100 mi (Clark 1960).



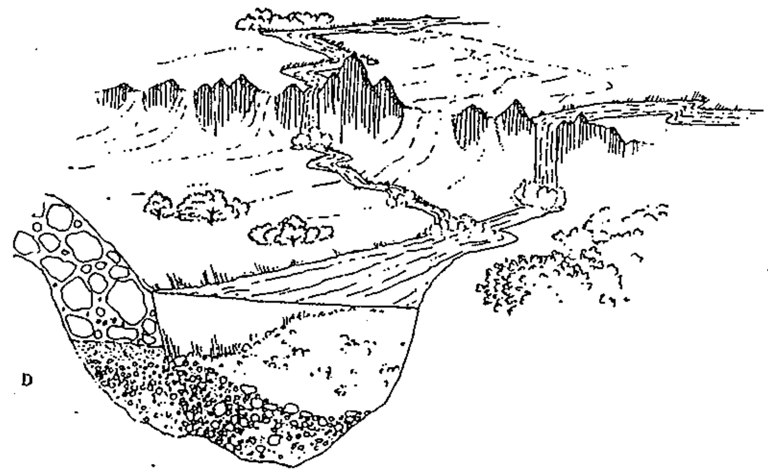
A, a stream of Early Tertiary time (about 50 million years ago) as it meanders through the gentle western slope of the Sierra Nevada. In the lower foothills, it encounters a ridge of resistant greenstone; it has cut through the ridge to plunge over a waterfall to the region underlain by soft rock below and to the west. A cut-out of the stream channel shows gravel in the bottom of the bed. Mixed with the gravel are nuggets and tiny fragments of gold, worn from the higher mountains and being carried by the stream toward the sea.



B, about 30 million years ago, shows how ash, falling from volcanoes erupting higher in the mountains, has clogged the stream. Where once the stream poured over the greenstone ridge in a rushing waterfall, a dam of ash has ponded the water behind the ridge. In the cut-out, ash may be seen covering the gravel of the river bottom.



C, steaming volcanic mud flows of 20 to 10 million years ago rolling downhill from near the Sierran Crest, covering much of the landscape. Here, the stream has been forced to seek a new route through the greenstone ridge; parts of the ridge and about half of the stream bed in the foreground have been covered by the mud. The cut-out shows a succession of deposits, including the original gravel in the bottom of the channel, followed in turn ash, then partly covered by a mixture of volcanic mud and rock.



D, the mud has cooled, and a new river has established its way through the greenstone ridge. The original route of the stream behind the ridge is now abandoned. The new stream course crosses the abandoned channel in the background; where it crosses, the stream may steal gold from the older channel. Where the channel lies buried and untouched by the new stream, gold may be locked beneath the ash and volcanic mud. The cut-out shows an older, buried gravel deposit, an ash bed, a volcanic mud layer, and a new gravel layer. Since the stream now has more water, and therefore more erosive vigor, the new deposit is actually lower than the older one. Part of the older gravel in the bottom of the bed and part of the covering ash have been reworked by the modern stream.

Figure 2.5. Damming and rerouting of a Tertiary stream by volcanic ash and mud (Hill, 1975).

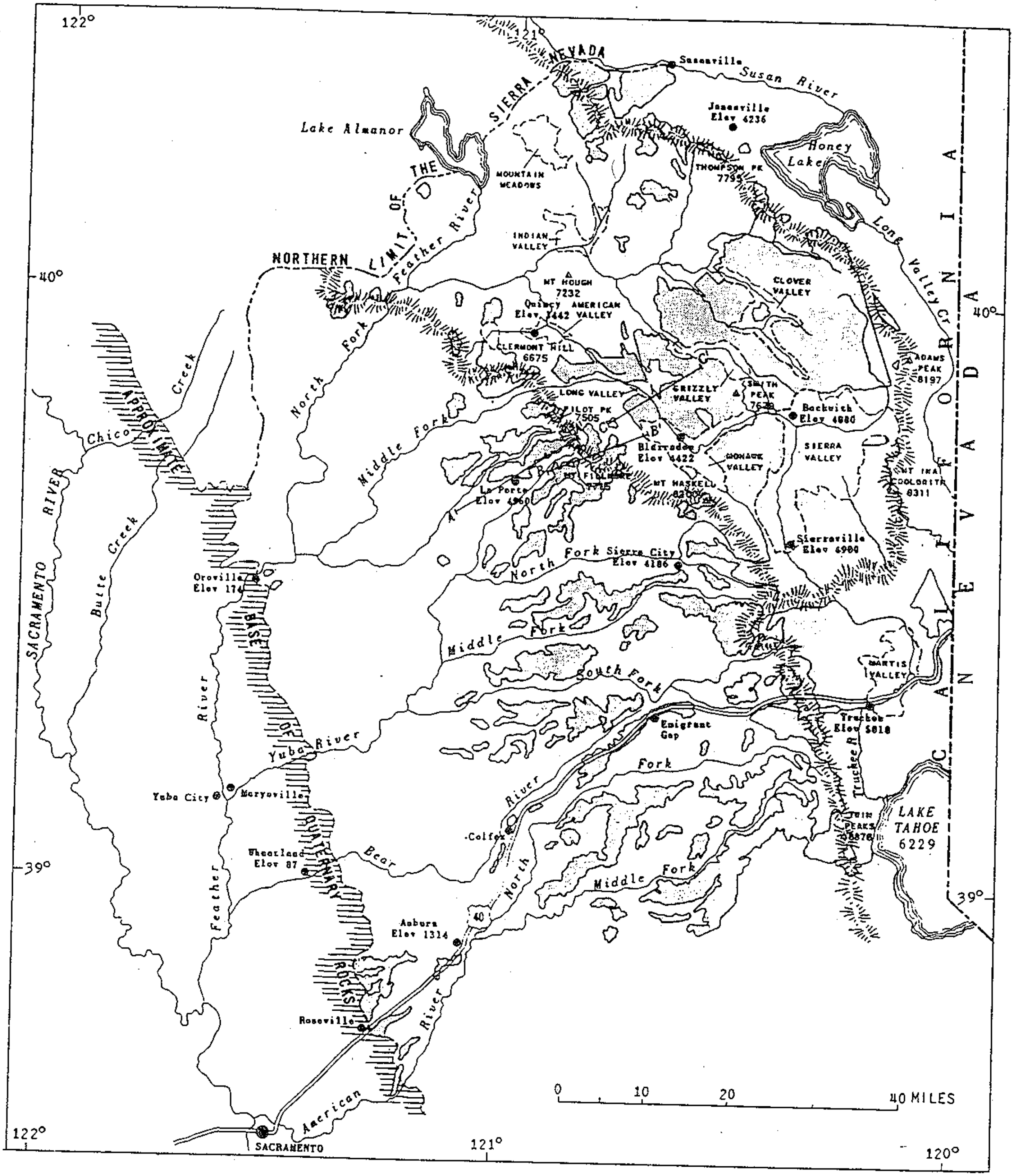


Figure 2.6. Map of the northern Sierra Nevada, showing the distribution of Miocene and Pliocene rocks (Durrell 1966).

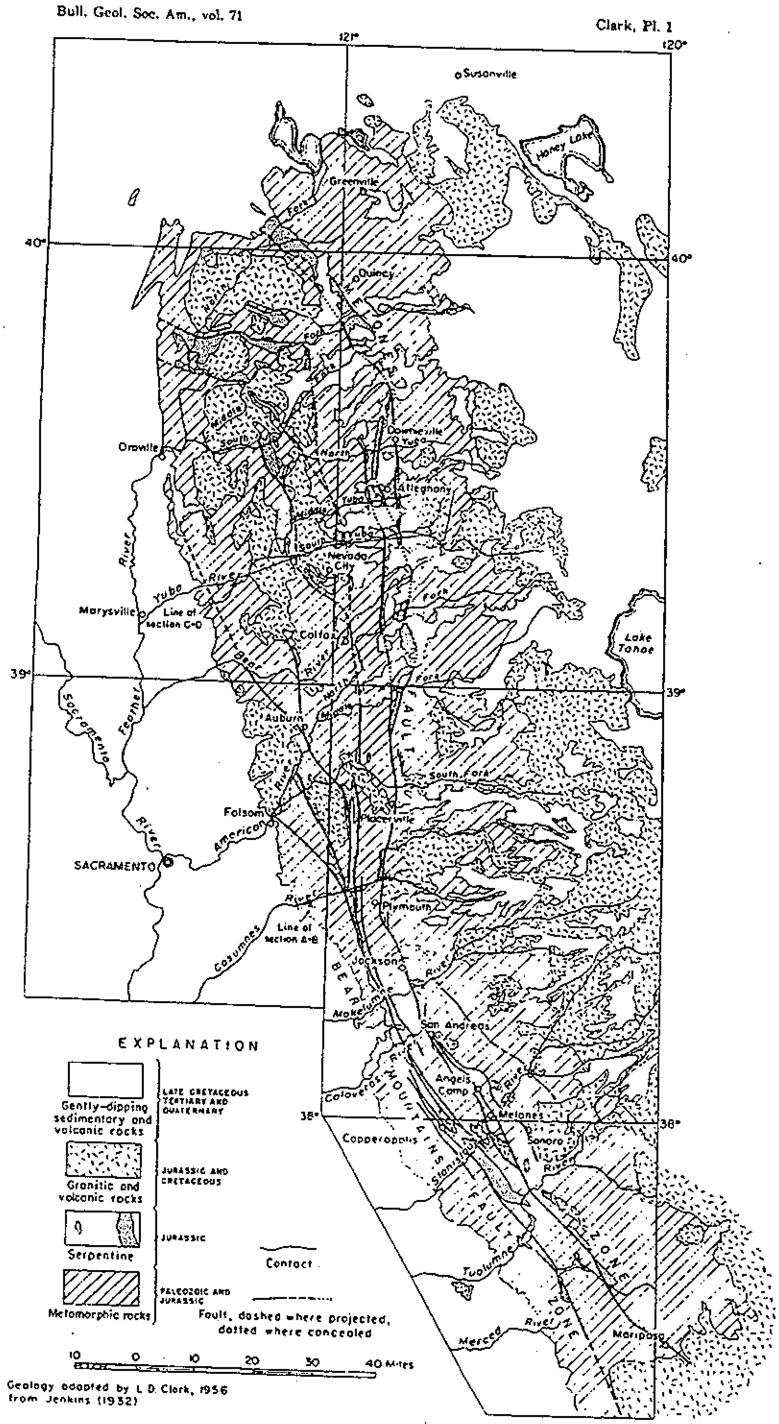


Figure 2.7. Outline geologic map of the western Sierra Nevada showing faults of the Foothills Fault System (Clark 1960).

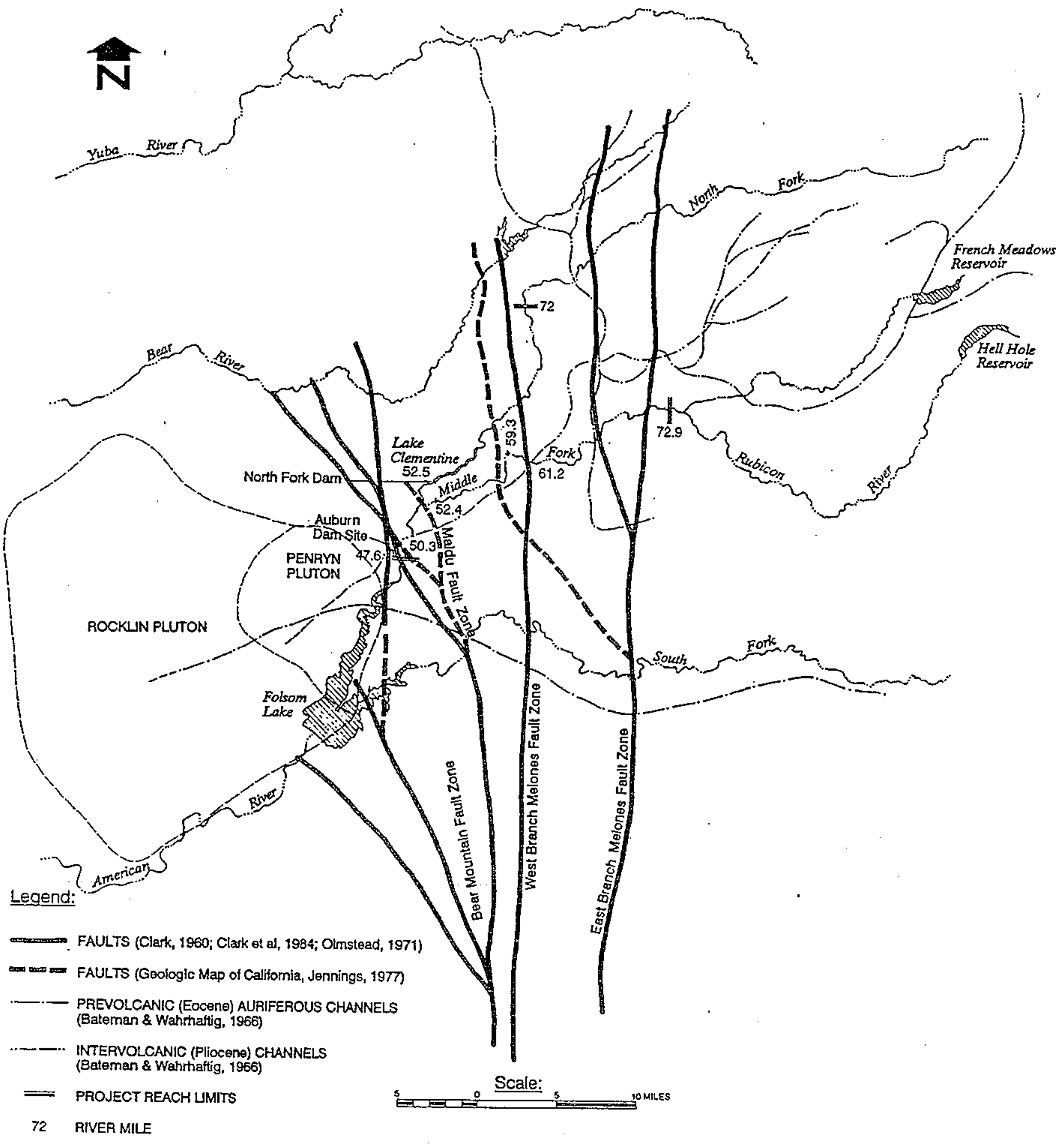


Figure 2.8. Map of the American River watershed showing the locations of faults and gold-bearing channels.

The Bear Mountain fault zone is located west of the Melones fault zone. This fault zone runs northward from Tuolomne County to Folsom Reservoir where it is truncated by the Penryn Pluton. The fault splits into several traces as it approaches the American River from the south. The Bear Mountain Fault zone separates subparallel belts of metamorphic rocks that may be parts of a single Triassic island arc or two separate exotic terrains (Norris and Webb 1990).

The Maidu fault zone is located within the area containing a series of faults related to the Bear Mountain fault zone, but the genetic relationship between the two is unclear. The Maidu fault zone crosses the North Fork of the American River just southwest of Auburn and crosses the Middle Fork northwest of Pilot Hill (Figure 2.8; Harwood and Helley 1987). The fault was identified in the Auburn area during trenching performed by Woodward Clyde Consultants (1977) as part of their earthquake evaluation of the Auburn dam site. The fault was originally identified as a surface lineament; trenching exposed a slicken-sided, clay-rich gouge that was bounded by fractured and bleached rock. Paleosols in the trenches were commonly offset, which led the investigators to conclude that some offset had occurred within the last 100,000 yr. Harwood and Helley (1987) investigated the features and concurred with the conclusions of Woodward Clyde Consultants (1977).

The Maidu fault zone is listed in the USGS map of Late-Quaternary faults of California (Clark, et al. 1984). The fault is described as vertical with a significant right-lateral component. Approximately 0.6 m (2 ft) of offset has been observed the B horizon on a buried soil on the fault plane. The age of offset is estimated to be 14,000 to 130,000 yr, and the estimated slip rate is from .005 to .07 mm/yr. The report also indicates that shrink-swell of expansive clays, downslope creep, and erosion are all possible causes of the apparent offset. The Maidu fault zone is the only fault within the area that has been demonstrated to be potentially active.

2.2. Geology of the Lower American River Basin

Downstream of Folsom, the American River flows into the Sacramento Valley. The geology of this lower reach is very different from that upstream of Folsom in the Sierra Nevada. The axis of the Sacramento Valley is underlain by marine sedimentary rocks that range in age from Late Jurassic to Early Miocene. These units are unconformably overlain by alluvial deposits and volcanic rocks of Early Miocene to Holocene age. The alluvial units that are of importance to this study include the Laguna, Riverbank, Modesto, and Turlock Lake Formations.

2.2.1. Laguna Formation

The Laguna Formation is Pliocene in age, which consist of interbedded alluvial gravel, sand, and silt. The fluvial deposits are Sierran-derived and are exposed only on the southeast side of the Sacramento Valley between Oroville and Sacramento. Helley and Harwood (1985) describe the Laguna Formation as deposited by the ancestral west-flowing Feather, Yuba, Bear, and American Rivers. The Laguna Formation is estimated to be approximately 180 ft thick near Oroville (thinning to about 60 ft south of the city of Sacramento).

2.2.2. Turlock Lake Formation

The Turlock Lake Formation is the oldest alluvial fan deposit in the American River study area. The unit forms an intermittent high bluff on the right bank of the American River from RM 15 to RM 23. One of the most distinguishing features of the Turlock Lake Formation is that its upper surface is generally extensively dissected, displaying vertical relief of up to 100 ft. The Turlock Lake Formation consists of arkosic silt, sand, and gravel. The gravel and sand beds are typically massive, lenticular, crossbedded, and difficult to trace laterally (Marchand and Allwardt 1981). In the study area, the Turlock Lake Formation is exposed topographically above the younger fans and terraces along the American River. The unit is well-exposed in the vicinity of the Sunrise Boulevard Bridge where it contains at least three units separated by paleosols. The upper unit of the Turlock Lake Formation is approximately 600,000 yr old (Marchand and Allwardt 1981).

2.2.3. Riverbank Formation

The Riverbank Formation consists of Sierran-derived arkosic sediments that form coalescing alluvial fans and terraces on the western margin of the Sierra Nevada foothills. In the American River study reach, Riverbank terraces and fans are cut into Turlock Lake alluvium or fill post-Turlock drainages; the Riverbank is in turn cut and filled by terraces of the Modesto Formation (Marchand and Allwardt 1981). The alluvial fan deposits of the Riverbank Formation dip gently westward beneath the younger Modesto Formation. The unit is dominated by sand and contains some gravel lenses and interbedded fine sand and silt. Locally, fine-grained horizons of the unit are discretely laminated, which suggests deposition in small ponds. The age of the Riverbank Formation ranges from approximately 130,000 to 450,000 yr (Marchand and Allwardt 1981).

The Riverbank Formation has been divided into two members. The upper member consists of unconsolidated but compact, dark-brown to red gravel, sand, silt, and minor clay. The lower member contains red, semi-consolidated gravel, sand, and silt. Both units represent severely dissected alluvial fan deposits. Helley and Harwood (1985) suggest that the asymmetrical distribution and spatial extent of the Riverbank Formation is the result of broad, slow, and relatively aseismic tectonic movement of the Sacramento Valley. The present character of the lower American River has been significantly influenced by the presence of Riverbank Formation in the channel bed and banks.

2.2.4. Modesto Formation

Similar to the Turlock Lake and Riverbank Formations, the Modesto Formation consists of arkosic Sierran-derived sediment that form alluvial terraces and some alluvial fans on the western margin of the Great Valley. Alluvial deposits of the Modesto Formation occur in a wide semi-continuous band, which extends from the Kern River drainage north beyond Oroville. The Modesto Formation consists of tan and light-gray gravely sand, silt, and clay. The unit borders modern rivers, which suggest that the stream systems that deposited the Modesto alluvium exist today (Helley and Harwood 1985). Strath terraces underlain by Modesto Formation are present along the study reach of the Lower American River. Reflecting the relatively young age of the Modesto Formation, terraces of the Modesto Formation are inset into Riverbank Formation terraces. The age of the Modesto Formation ranges from 12,000 to 42,000 yr (Marchand and Allwardt 1981).

2.2.5. Faulting

The only fault mapped within the Lower American River study area is an "uncertain extension" of the Willows Fault (Harwood and Helley 1987). The southeast extension of the Willows Fault is based upon three wells that show possible displacement of the basement across the fault. The mapped extension of the Willows Fault crosses the American River at approximately RM 8.0. No surficial evidence of the fault has been documented at this location. Although a potential for modern activity on the main segment of the Willows Fault to the northwest has been suggested, there has been no evidence documented to suggest that the southeast extension of the Willows Fault constitutes an active structural element in the Sacramento Valley.

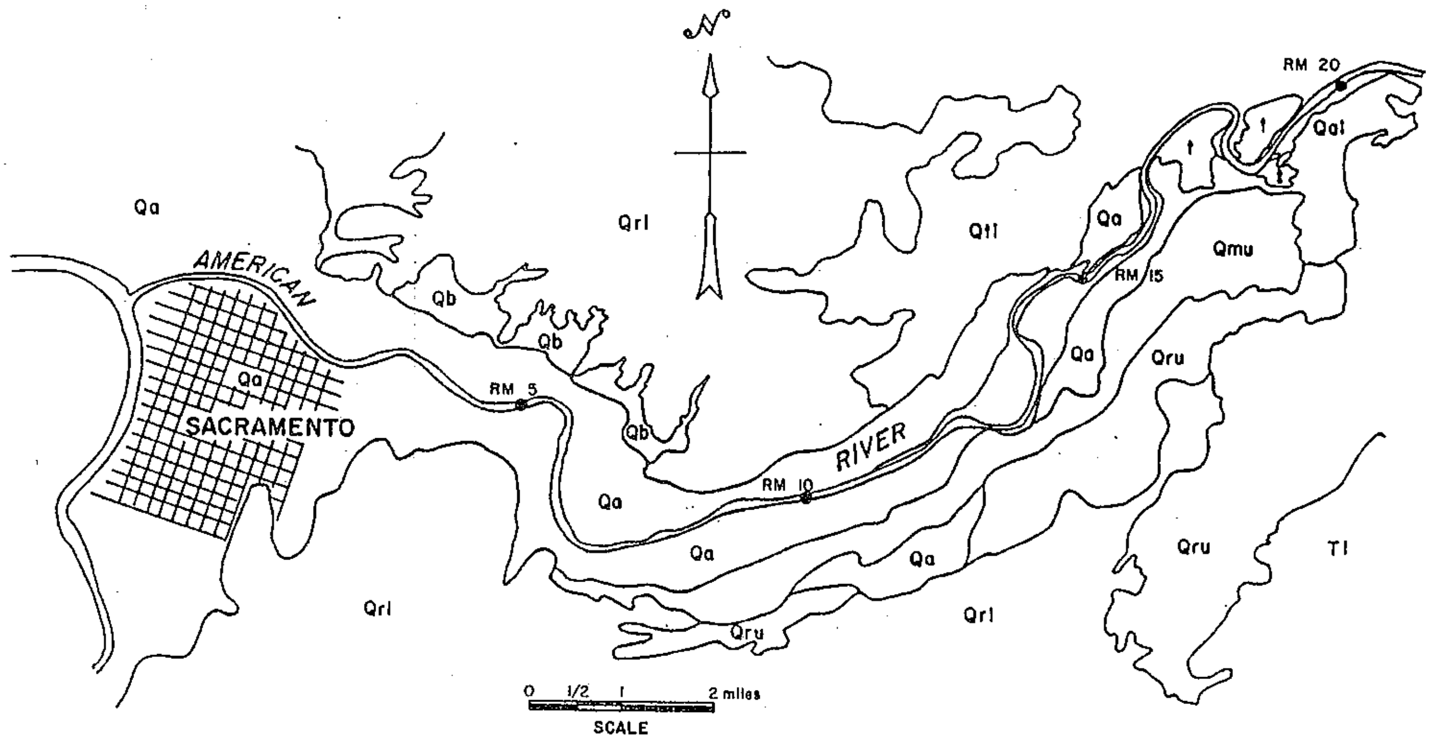
2.2.6. Pleistocene Geologic History

Downstream of Folsom dam, the American River flows through a valley that is bordered by nested terraces that are underlain by Pleistocene-age alluvial deposits (**Figure 2.9**). The nesting of these terraces records cut-and-fill cycles along the western edge of the Sierra Nevada foothills. These alluvial sediments were deposited during uplift of the Sierra Nevada, and although lithologically similar, have been subdivided on the basis of soil profile development, topographic position and expression, local lithologic differences, and unconformities associated with buried soils (Marchand and Allwardt 1981). These units include the Turlock Lake, Riverbank, and Modesto Formations. Early studies considered all three to be part of the Victor Formation (Olmstead and Davis 1961).

The Turlock Lake, Riverbank, and Modesto Formations all record periods of Pleistocene activity of the American River. Shlemon (1972) mapped out five individual Pleistocene-age American River channels that depict a distinct northward migration of the American River (**Figure 2.10**). Each of these channels is located in a different stratigraphic horizon, which records several Pleistocene cut-and- fill cycles (**Figure 2.11**). The modern American River channel flows through a deep entrenchment into the older alluvial deposits. Shlemon (1972) concluded that the periods of entrenchment were induced by the onset of glaciation and a eustatic sea level drop. Entrenchment was followed by the deposition of coarse channel deposits. A rapid influx of silt-dominated sediment ensued, reflecting the recession of the glacial fronts. This sequence of events is recorded in the Pleistocene stratigraphy, where coarse channel deposits are in very sharp contact with overlying silts of similar age.

2.2.7. Effect of Geology on Lower American River Behavior

The presence of Pleistocene-age alluvial strath terraces and surfaces along the Lower American River has greatly affected the dynamics of that system. Impinging on high bluffs of Turlock Lake Formation in the vicinity of Fair Oaks, the American River has migrated in a northerly direction through Quaternary time. Continued northward migration will be restricted due to the presence of the bluffs. Resistant outcrops are commonly present as low strath surfaces inset within the main channel. Where bend ways impinge on resistant Pleistocene-aged materials, they are commonly distorted (e.g., San Juan Rapids at RM 18; **Plate 18**). Exposures of strath surfaces in the channel bed constitute effective local grade control for the river. The general erosion risk associated with the presence of the Pleistocene deposits is lateral migration of the river over the strath surfaces or channel skating. The potential for channel skating is greatest where Pleistocene-age outcrop forms the channel bed at RM 9.8, RM 14.0 and RM 18.0. Retreat over strath surfaces that bound the river may occur at high stages. The presence



MAP LEGEND:

Qa: ALLUVIUM (Holocene): Unweathered gravel, sand, and silt deposited by present-day stream and river systems
Qb: BASIN DEPOSITS (Holocene): Fine-grained silt and clay derived from the same sources as modern alluvium
Qsc: STREAM CHANNEL DEPOSITS (Holocene): Deposits of open, active stream channels without vegetation
Qmu: MODESTO FORMATION (UPPER MEMBER): Unconsolidated, unweathered gravel, sand, silt, and clay
Qml: MODESTO FORMATION (LOWER MEMBER): Unconsolidated, slightly weathered gravel, sand, silt, and clay
Qru: RIVERBANK FORMATION (UPPER MEMBER): Unconsolidated but compact, dark-brown to red alluvium composed of gravel, sand, silt and minor clay
Qrl: RIVERBANK FORMATION (LOWER MEMBER): Red semiconsolidated gravel, sand, and silt
Qtl: TURLOCK LAKE FORMATION: Deeply weathered and dissected arkosic gravels with sand and silt

Figure 2.9. Geologic map of study area (Helley and Harwood, 1985).

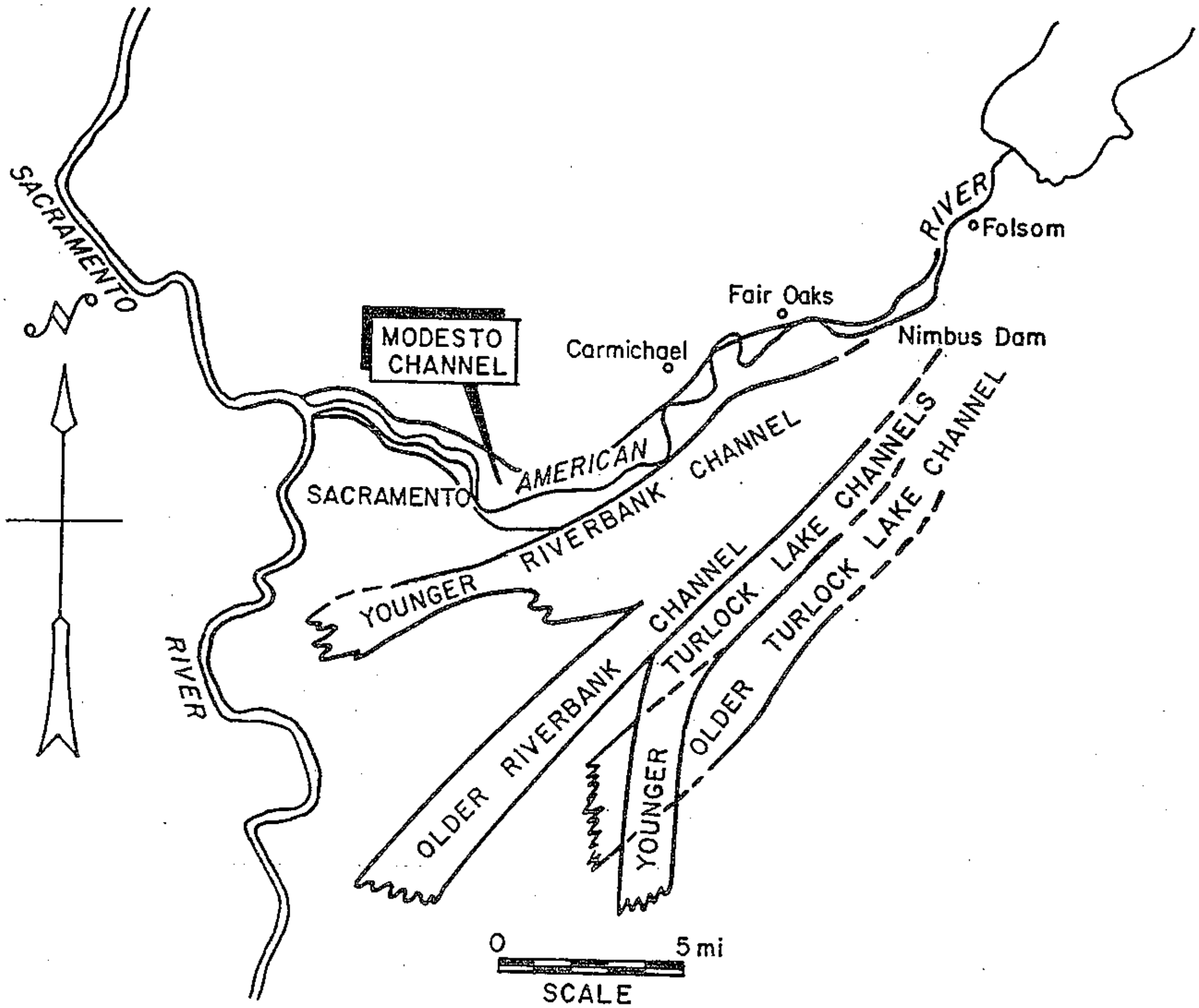


Figure 2.10. Pleistocene channels of the lower American River (Schlemon 1972).

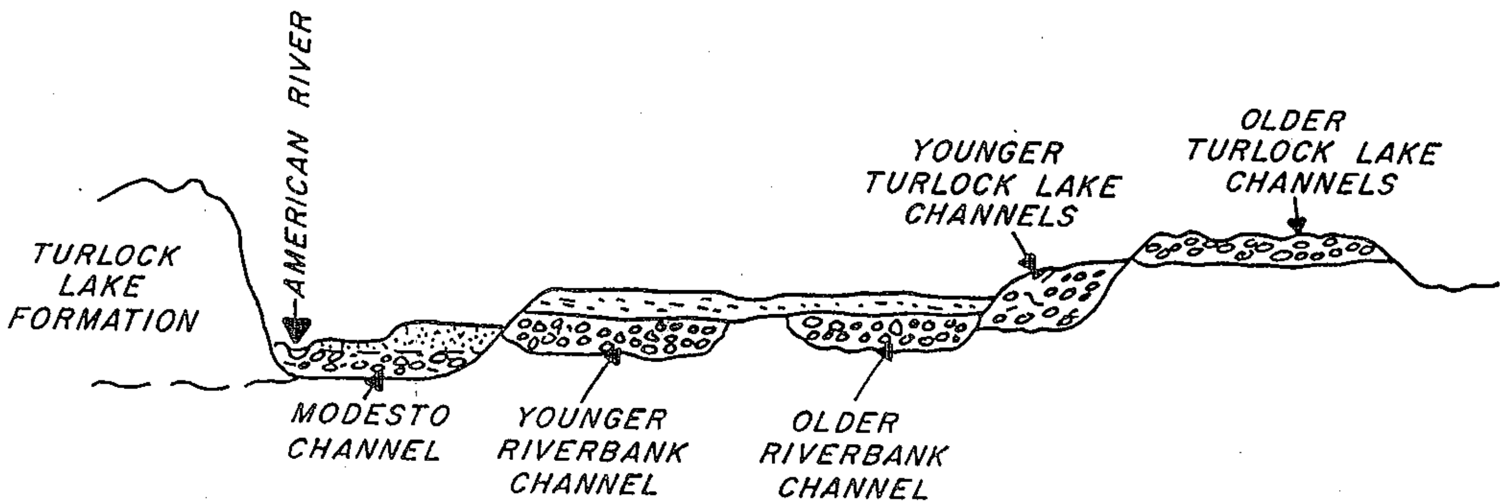


Figure 2.11. Generalized composite cross section from American River near Fair Oaks to the south toward Cosumnes River (Shlemon, 1972). View is upstream (east).

of strath surfaces and terraces along the river record historic lateral migration of the American River over the older units.

From Folsom to Fair Oaks, the American River floodplain is narrow. At Fair Oaks, the floodplain widens to about 1 to 1.5 mi, and the steep 125-foot high bluff of Turlock Lake Formation bounds the northern channel margin. Downstream, near Sacramento, the bluff height reduces to less than 10 ft and consists of Riverbank Formation. The southern channel margin consists of a terrace of Recent-age alluvium that is lower than the northern bluff. The levees that have been constructed along both banks of the lower river are, therefore, critical to flood control operations.

2.3. Hydraulic Mining in the American River Basin

Similar to the Feather, Yuba, and Bear Rivers, the American River has been greatly affected by hydraulic mining practices. In 1848, gold was discovered on the South Fork of the American River at Sutter's Mill. Between 1849 and 1909, approximately 255 million cu yd of mining debris entered the channels of the American River basin (Hagwood 1981). Manson (1882) suggested that most of the tailings from the largest operations were lodged in a "permanent way" in valleys within the upper watershed. Manson (1882) estimated that the volume of debris along the North Fork of the American River was 20 million to 25 million cu yd. He stated that there was a similar quantity on the Middle Fork, but none was on the South Fork. A majority of the gold-bearing, Tertiary-age gravels on the South Fork are overlain by volcanic rocks and were not mined hydraulically. Manson wrote, "The South Fork is practically clear of debris; the application of the hydraulic process to the ancient river gravel is limited, owing to the lava covering the greater portions of the ancient channels; those portions which were exposed under the edges of this lava cap have been nearly exhausted."

Hall (1880) suggested that the nature of the hydraulic mining debris that entered the American River channels was different in character from that entering the rivers to the north.

> "The materials put into the American River are of a lighter nature, contain more top soil on the average than those of the Feather, Yuba, Bear, or any other stream now being considered. At some points, almost the entire washing sometimes is in this red soil, which all goes off in suspension. At other points where the material is heavier, the great fall, which it has into the deep canyons and the immense pulverizing power of the floods there, grind it up to the finest atoms, and thus it is carried up in the waters."

Manson (1882) wrote the following:

> "The debris derived from the North and Middle forks of American River is of about the same nature, and is principally sand and gravel, mixed with cobbles and pipe clay. The material reaching the Sacramento is very fine gravel, sand, and sediment. The American, unlike Yuba and Bear Rivers, is muddiest during freshets, particularly the earlier ones; this is due to the sweeping out of the deposits of the past low-water season, to the greater extent of mining operations during the rainy season, and to the large amount of natural denudation. In this respect it resembles the Feather.

On the South fork the character of mined material is entirely different. The gravel deposits are largely covered by a lava-cap from fifty to several hundred ft in depth, which prevents profitable hydraulic mining."

During the initial phase of gold mining, Recent-age channel deposits were mined in the Lower American River in addition to the Tertiary-age gravel mined from higher in the watershed. Mendell (1881) describes this mining of Recent-age sediments:

> "The American River leaves the foothills at Folsom, the forks having united some 3 mi above. From Folsom to a point 1.5 mi below Alder Creek, or 5 mi below Folsom, the river flows between high gravelly banks. These banks are composed of cobble-stones, gravel, sand, and argillaceous material, packed together in various degrees of hardness, and overlaying a hard cement familiarly known as 'bedrock and hard-pan.' Gold occurs in all of the material above the cement, and in many places has been extensively mined in past years, and is even yet found to pay in a few localities."

With the evolution of channel dredges, mining of the American River channel deposits on the Lower American River continued. Some of the greatest successes of dredge mining since the turn of the century occurred on the American River near Folsom (Hagwood 1981).

The effects of the hydraulic mining operations on the American River were described relatively early in the hydraulic-mining period by Mendell (1881), who wrote the following:

> "Mining operations were commenced on the American River in 1849; the auriferous banks, near Alder Creek, were particularly rich. The tailings from such mines as were opened were dumped into the bed of the river. The filling of the channel consequent to these operations cannot now be accurately ascertained, and is variously reported at from 5 to 30 ft. The first noticeable effect of this filling was produced in 1862. The regimen of the river having been materially changed, much destruction was wrought. Large tracts of land were swept away in some places, and immense deposits caused in others. At one point, some 12 mi above Sacramento, and on the left bank, 400 ac were cut down to a depth of some 5 ft, and on the opposite bank half this area was similarly swept off. So far as damage to lands is concerned, they are not so destructive as deposits from Yuba and Bear Rivers. This is owing in part to the character of mining and in part to the large amount of natural wash from the watershed of American River. Lower down similar changes took place at 'the island' 9 mi above Sacramento. The river bed, as existing prior to 1862, was so filled up with mining detritus that a new channel was cut. Near the mouth, the old channel has been almost entirely obliterated and some 15 sq-mi of land covered with detritus, although the greater part of this area is not yet materially damaged. The river at its mouth was formerly subject to a tidal action of at least 2 ft. At present the tides do not reach within 3 mi of it and the plane of low-water has been raised at least 5-1/2 ft. The average filling of the bed for some 10 mi above the mouth is at least double and probably thrice, the rise in the plane of low-water."

The effects of hydraulic mining on the Lower American River are somewhat less than those on the lower Yuba and Bear Rivers. It is clear, however, that bed aggradation did occur as a result of increased sediment loads. As hydraulic mining activity decreased, degradation into the accumulated mining debris occurred. Since the river within the study reach has degraded down to Pleistocene-age outcrop in several locations, the removal of mining-derived debris from the bed of the Lower American River is locally complete. In the upper parts of the American River basin, construction of North Fork Dam (Lake Clementine) in 1939 effectively eliminated farther downstream transport of the mining debris. Hydraulic mining debris is still being transported in the Middle Fork and is being delivered to the North Fork at the confluence. Fuller descriptions of the effects of hydraulic and dredge mining on the American River are presented in Sections 2.4 and 2.5.

2.4. Geomorphology of Middle and North Forks, American River

2.4.1. Steepland Channel Literature Review

On a macro scale the streams and rivers that drain the headwaters of the Sierra Nevada are composed of some alluvial, but mostly canyon-bounded reaches. The channels in the alluvial reaches adjust over a period of years so that the sediment supplied is transported with the available discharge such that the dimensions of the channel can be considered to represent a condition of dynamic equilibrium (Andrews 1986). Such a condition does not exist in the canyon-bounded reaches where bedrock and other factors exert a major influence on the channel dynamics (Ashley et al. 1988; Webb et al. 1988).

In general the canyon-bounded reaches contain steep coarse-grained channels that can, in many cases, be considered to be courses of convenience; that is, they bear the imprints of numerous geomorphic processes that are not uniquely related to the present day watershed hydrology and sediment yield. Channel form and processes are related to infrequent hydrologic events, structural controls (faults, joints), landslides, man-induced impacts (e.g., hydraulic and placer mining, road construction) or discharges that occurred under different climatic regimes. Because of the caliber of the sediments that form the channel perimeter the large magnitude events (50-year or greater recurrence interval) are the discharges that control the primary structure of the channel (Grant et al. 1990). This is in contrast to alluvial reaches where the form and dynamics of the channels can be related to events of moderate magnitude and frequency that have been referred to as channel-forming or dominant discharges (Wolman and Miller 1960).

The Middle and North Forks of the American River exhibit significant bedrock control of the channel position, geometry, and gradient. In addition, landslides (colluvial and bedrock failures; SCS 1993), rockfalls and tributary-derived debris flows have placed materials with a large range of sizes within the channel; the large boulders in these deposits are not transported by typical modern day discharges. Also, the terraces and high-elevation boulder bars present throughout the study reach have been modified by mining practices and in the case of the Middle Fork by failure of the Hell Hole Dam on the Rubicon River in 1964.

Because many factors influence the form of the channel and the spatial distribution of meso-scale features such as rapids, riffles, pools, and bars within the canyon-bounded reaches, there is an indeterminant relationship between channel size and frequently occurring discharges (Lisle 1986, 1987; O'Connor et al. 1986). There is no consistent increase in channel size in the downstream direction, and neither bed material size nor channel slope diminish in a systematic fashion in the downstream direction. The frequency of the bankfull discharge is not constant along the length of the channel, and somewhat meandering. Generally lower gradient alluvial storage reaches are interspersed between steeper bedrock-controlled reaches that are essentially conduits that convey most of the supplied sediment downstream. This longitudinal variability in geomorphic character strongly impacts the sediment delivery and transport characteristics of the watershed drainage system. Relatively finer grained sediments tend to be deposited in zones influenced by large in-channel and channel margin structural features (boulder accumulations, bedrock outcrop). They are remobilized during high magnitude flows. Remobilization of sediments deposited during high flows as the result of local backwater conditions may only occur during lower magnitude discharges when the local hydraulic energy is greatest due to diminished effects of downstream controls (Harvey et al. 1991, 1993; Mussetter et al. 1993). Depending upon the local variability of conditions causing individual sediment deposits, remobilization of sediments may not occur under typical annual flow conditions.

Local controls (such as bedrock outcrop and boulder accumulations from landslides, debris flows, rockfalls, tributary fans) have significant effects on the meso-scale morphology and dynamics (hydraulics and sediment transport) of the channels (Leopold 1969; Graf 1979; Kieffer 1985; Howard and Dolan 1981). The longitudinal profiles of the channels are composed of steeper reaches interspersed with flatter reaches; this is referred to as step-pool morphology (Chinn 1989). The steps in the step-pool association are generally formed by clusters of large rocks that can only be transported by extreme flow events (Larrone and Carson 1976; Grant et al. 1990). Step-pool channel morphology is indicative of a sediment supply-limited stream (Grant et al. 1990). Both the Middle and North Forks exhibit step-pool morphology in reaches where the valley is narrow as a result of the presence of more erosion resistant bedrock (ultramafic rocks). Where the valley is wider and the rocks are less erosion resistant (metamorphosed sedimentary rocks), there is some storage of sediment within the reach the channel form is pool-riffle.

The hydraulics of steep mountain channels such as the North and Middle Forks of the American River, and therefore, the sediment transport capacity, are very much affected by the very coarse-bed materials, the step-pool morphology of the channels and bedrock outcrops (Jarrett 1985; Mussetter 1989; Wiberg and Smith 1987; Whittaker and Jaegi 1982; Harvey et al. 1993). The steep- and coarse-grained streams in which sediment transport capacity exceeds measured sediment transport rates by orders of magnitude can be classified as supply limited in contrast to lowland streams where sediment yield is controlled by available energy (Harvey 1982; Whittaker 1978; Mussetter 1989; Grant et al. 1990).

In a supply-limited channel system, sediment deposition tends to be controlled by local factors such as flow expansion and contraction zones, which in turn are controlled by the resistance to erosion of the lithologic units traversed by the river (Lisle 1986), or by depositional features such as tributary alluvial fans (Harvey et al. 1991). Because of the numerous factors that control sediment transport and deposition within the canyon-bounded reaches bed material transport is an episodic phenomenon. Sediments deposited because of local hydraulic controls during high discharges are reentrained during recessional flows of the same event and are deposited farther downstream. These same sediments can then be entrained during the rising limb of the next flood when incipient conditions are exceeded and deposited during the higher flows.

2.4.2. Geomorphology of the Middle Fork, American River

The Middle Fork of the American River was traversed by raft, and (where possible on the ground) from immediately upstream of the confluence with the North Fork of the Middle Fork (Indian Bar, RM 74) to the confluence with the North Fork at RM 50.3 (Figure 2.8). During the course of the field traverse sediment sampling (Wolman Counts; Wolman 1954) was conducted at seven locations. Each of the sampling locations was chosen to provide an indication of the sizes of sediment that were transported and deposited during the 1993 runoff season in which the peak discharge was on the order of 19,000 cfs. Observations on the morphologic and geomorphic characteristics of the channel and valley were made, and modes of sediment delivery to the channel were recorded.

Based on the observations made during the field traverse the following five subreaches were identified: (1) RM73 (Horseshoe Bar tunnel) to RM 70, (2) RM 70 to RM 61.2 (Canyon Creek/Landslide Rapid), (3) RM 61.2 to RM 59.3 (Greenwood Bridge), (4) RM 59.3 to RM 52.4 (Murderer's Gorge), and (5) RM 52.4 to RM 50.3 (Confluence with North Fork).

Subreach 1. commences at the downstream end of the tunnel at Horseshoe Bar (RM 73) and terminates at RM 70. The tunnel was excavated to cutoff the bedrock-incised bend (Horseshoe Bar) to enable placer mining of the channel. The tunnel effectively prevents any bed load transport from the upper watershed (Middle Fork, North Fork Middle Fork, Rubicon); therefore, sediment delivered downstream to the proposed dam site must be derived from that portion of the watershed that lies downstream of RM 73.

Between the tunnel and RM 70, the channel is characterized by very coarse bed material and rapids that are located within a very narrow, mildly sinuous and deeply incised canyon that is bounded by relatively resistant metavolcanic rocks. High relief coarse grained bars are located upstream of bedrock controls (**Plate 1**), but nearly all of the bars have been reworked during placer mining operations. Even though the bars have been (in places continue to be) placer mined the cobbles and small boulders that constitute the surface sediments are imbricated, which indicates that they have been fluvially transported. Failure of the Hell Hole Dam on the Rubicon River in 1964 resulted in a peak discharge on the order of 300,000 cfs in the subreach, and this event may well be responsible for the mobilization and subsequent deposition of the bar sediments. Finer grained sediments that were mobilized during the 1993 runoff season (peak discharge 19,000 cfs) tend to have a high proportion of quartz clasts, which indicates that they were probably derived from hydraulic mining operations. At RM 72.8 the median grain size (D_{50}) of the sediments deposited in a bar-head riffle is 230 mm (**Figure 2.12**).

Valley side slopes in the deeply incised canyon are on the order of 60 to 75 percent. Where the bedding planes in the metamorphic rocks parallel the slopes relatively shallow bedrock translational failures have delivered coarse material to the channel. Relatively small-scale rotational failures in colluvium have delivered both coarse and fine sediments to the channel. Large flood events such as the Hell Hole Dam failure in 1964, 1963 (113,000 cfs), 1986 (78,400 cfs) have eroded the detrital cones at the base of the valley

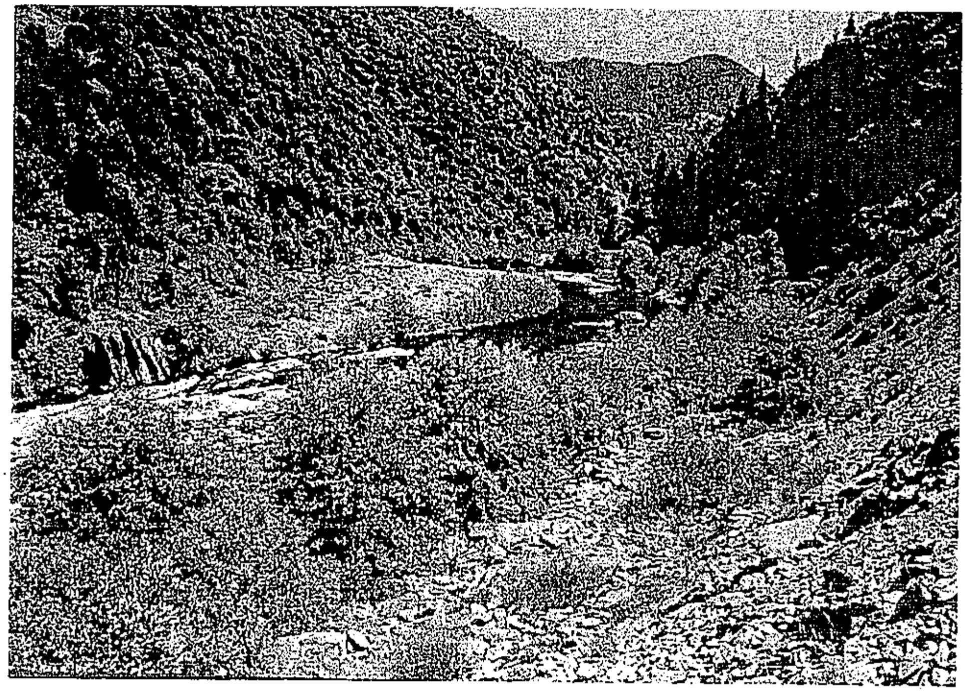


Plate 1. View upstream of Middle Fork American River (RM 71) showing bedrock controlled bar deposition, post 1986 riparian vegetation and self-armored toe of colluvial slope (Subreach 1).

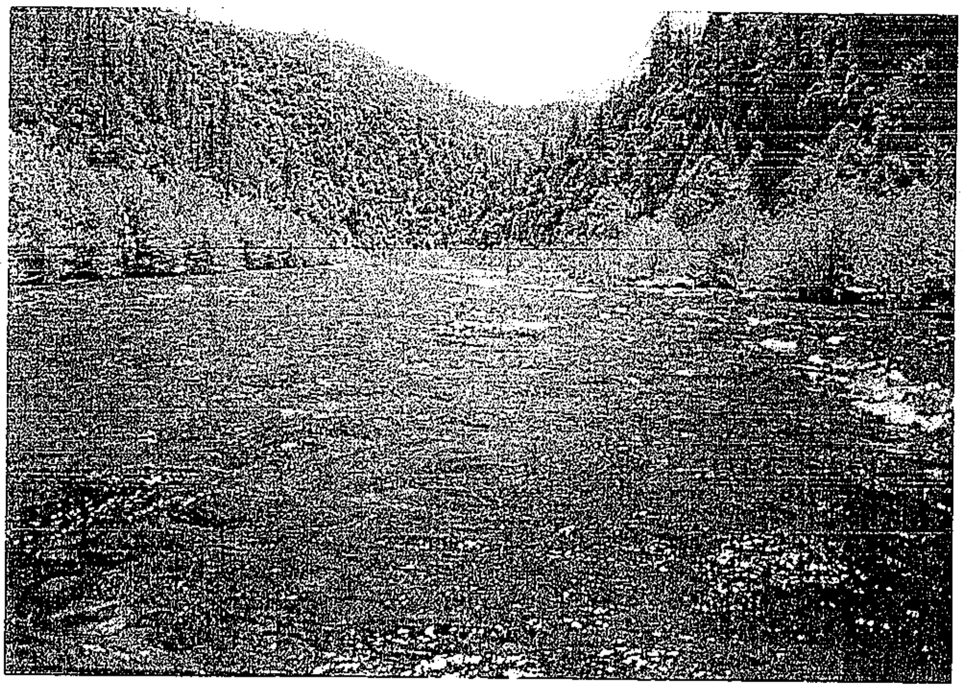


Plate 2. View downstream of Middle Fork American River (RM 68) showing channel in wider valley. Note difference in channel width and continuity of the riparian vegetation (Subreach 2).

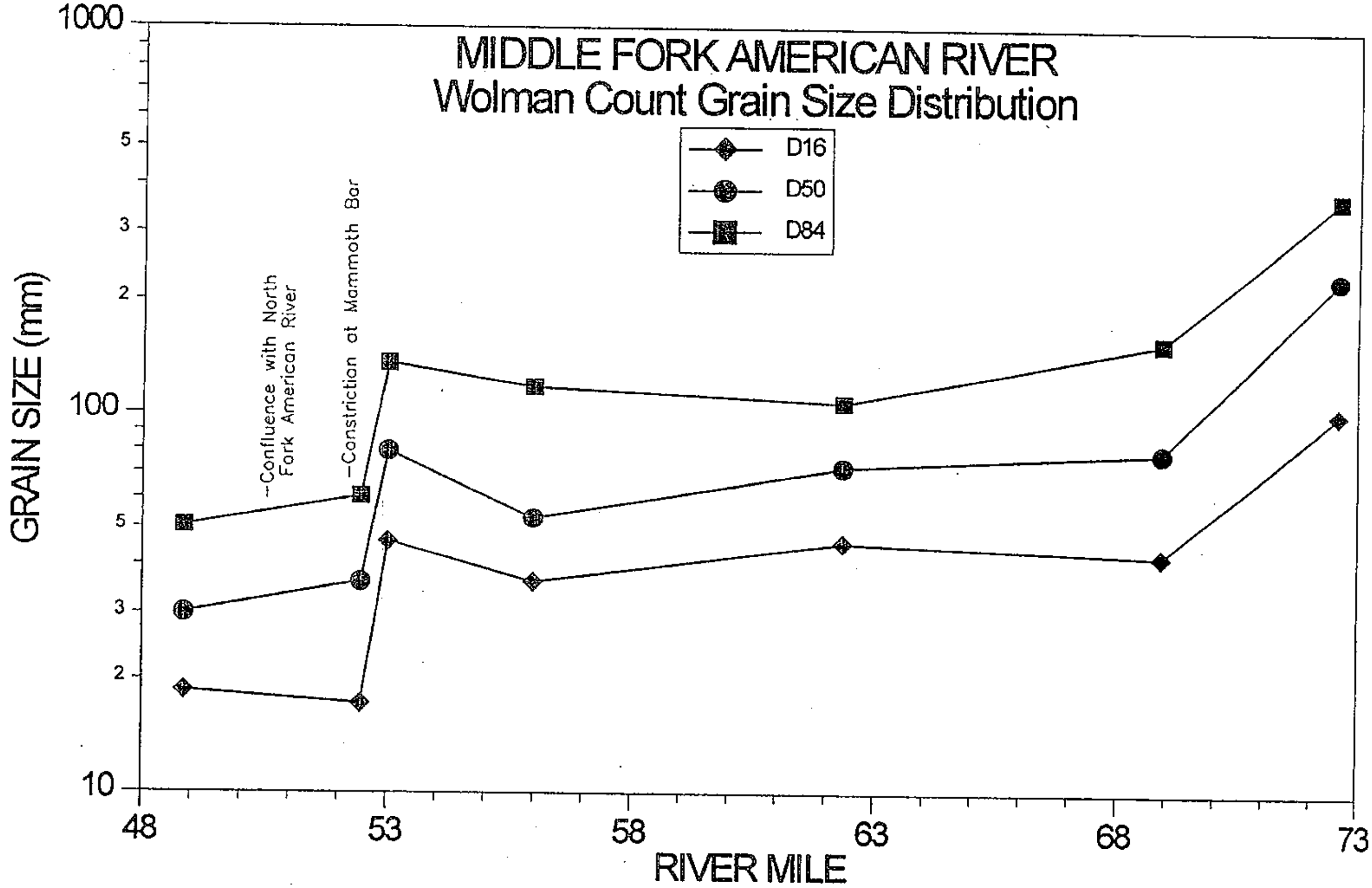


Figure 2.12. Grain size distribution parameters derived from Wolman Counts of bar-head sediments on Middle and North Forks of the American River.

side slopes (Plate 1). As a result the toes become armored with lag deposits that tend to prevent entrainment of detrital material except during the most extreme events. The belt of riparian vegetation located at the base of the slopes is dominated by even aged alders that post date the 1986 flood (Plate 1). In common with other canyon-bounded rivers the age of the riparian vegetation stands depends to a great extent on the recurrence interval of flood events that are capable of mobilizing the substrate (Lisle 1989).

Subreach 2 extends from RM 70 to Landslide Rapid at RM 61.2. The bed material in this subreach is somewhat finer than in the upstream reach primarily because the entrenched valley is wider and there is storage of alluvium in the reach. The channel is wider and its structure can be best described as pool-riffle **(Plate 2)**, but bedrock outcrop in the lower radius bends still have a significant local effect on the channel form. The wider valley is the result of less erosion resistant metasedimentary rocks. Slope angles tend to be somewhat lower (approximately 60 percent) and the majority of slope failures have occurred in colluvium.

The combined effect of reduced direct delivery of coarse bedrock to the channel and the finer nature of the landslide detritus is finer bed material in the subreach. Wolman Counts were performed at bar-head riffles at RM 68.9 and RM 62.3 and the D_{50}s for each location are 80 mm and 70 mm (Figure 2.12). The sediments are quartz dominated and are therefore derived primarily from hydraulic mining debris.

Most of the small tributary drainages in the subreach do not have distinguishable fans at their mouths, which tends to suggest that either sediment yield is low from the tributaries or that the Middle Fork flows are capable of transporting the delivered sediments. Field observations of the tributaries indicate that episodic debris flows may be the dominant sediment delivery mechanism **(Plate 3)**.

Landslide Rapid **(Plate 4)** was formed in 1940 as a result of a left bank failure of the colluvial slope. This is probably the result of construction activities associated with the California Debris Commission's (CDC) proposed Ruck-A-Chucky sediment retention dam (Hagwood 1981; SCS 1993). The valley/channel contraction at this site (RM 61.2) causes a backwater condition to extend upstream to about RM 62, which has resulted in significant deposition of sediment in the pool. The gradation of the pool sediments fines from cobbles and gravels at the upstream end to sand at the downstream end. The contraction is a very significant factor in controlling downstream delivery of bed material (Section 3.4).

Subreach 3 extends from RM 61.2 to the former site of the Greenwood Bridge (RM 59.3) that was destroyed by the Hell Hole Dam failure flood in 1964. The reach is characterized by an extremely narrow valley and channel and very little sediment storage. The valley narrowing is due to a change in lithology from relatively weak metasedimentary rocks to more erosion resistant ultramafic rocks (serpentenite). The channel is comprised of a series of rapids (Ruck-A-Chucky) that are formed by accumulations of large colluvially-derived boulders and bedrock outcrop in the bed and banks of the channel. Bed material that passes the Landslide rapid contraction is conveyed through the subreach. Very coarse grained alluvial fan remnants are located at the mouths of Todd Creek and Gas Canyon, two small north bank tributaries to the subreach. Sediment delivery from these drainages is dominated by debris flows. Mine dumps on the north bank in the vicinity of Greenwood Bridge may have contributed some sediment to the Middle Fork historically.

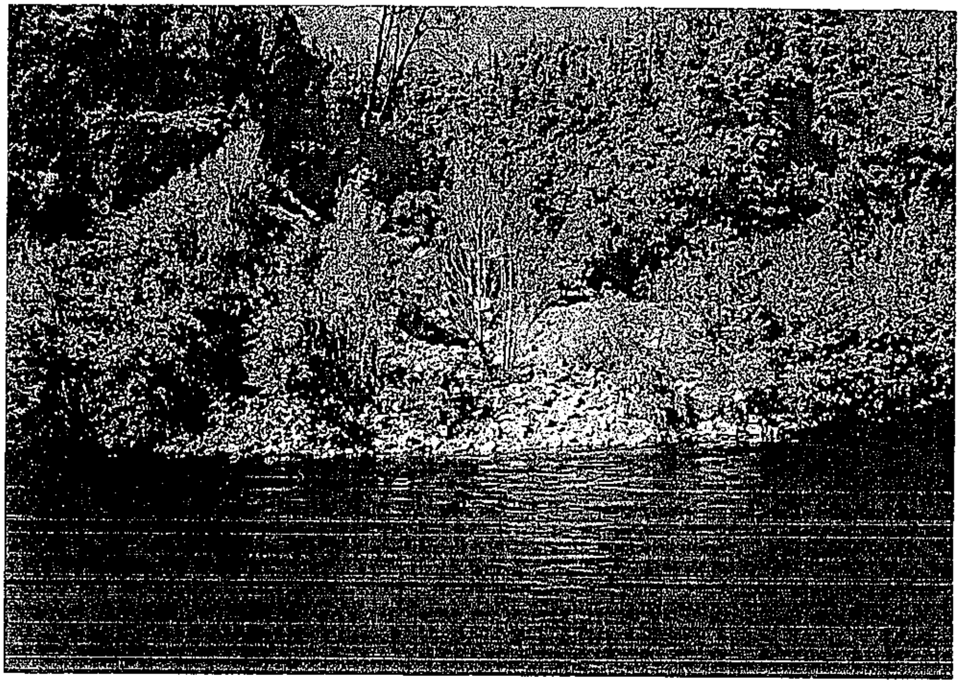


Plate 3. View of recent (1993) debris flow deposit at mouth of Slug Gulch (RM 62.2) a north bank tributary to Middle Fork American River (Subreach 2).

Plate 4. View upstream of Landslide Rapid at RM 61.2, Middle Fork American River. The rapid formed in 1940 as a result of construction-induced landslide at the Ruck-A-Chucky Dam site.

Subreach 4 extends from RM 59.3 (Cherokee Bar) to RM 52.4 at Murderer's Gorge a resistant bedrock (amphibolite) (and possibly prehistoric landslide) contraction, which is a major hydraulic control on the lower reach of the Middle Fork (**Plate 5**). The subreach has a much wider valley because the presence of weaker metasedimentary rocks and an alluvial channel that is characterized by the presence of 10 relatively coarse grained alternate bars that have all been extensively placer mined.

Investigations of aggregate resources for the Auburn Dam Project by the Bureau of Reclamation (1967-1981) provide bulk (subsurface) grain size data for the 10 alternate bars (**Figure 2.13**). The bar sediments have sand contents of between 15 and 52 percent and the maximum grain sizes are on the order of 300 mm (COE 1991). Median grain sizes for the bar sediments range from about 2 to 15 mm (Figure 2.13). In contrast, the median grain sizes of 3 sediment surface gradations determined by Wolman Counts range from 35 to 80 mm (Figure 2.12). Although the bars have been mined extensively the coarse surface sediments are imbricated, which indicates fluvial transport and reworked sediments since cessation of the placer mining. The higher elevations of Mammoth Bar (RM 53) are mantled with fluvially transported, horizontally laminated silty sands (D_{50} 0.17 mm) that were deposited during high magnitude discharges when the contraction at RM 52.4 was creating significant backwater.

The subreach is somewhat anomalous in terms of the average depths (approximately 27 ft) and volumes (approximately 800,000 yd^3) of stored sediments in the individual bars (Data from COE 1991) especially when the overall canyon setting is considered. The subreach can be subdivided into two separate reaches based on the dominant controls. From RM 59.3 to RM 57 (Poverty Bar) before resuming its general west-southwest course within a low sinuosity valley, the river flows in an anomalously straight valley that trends in a north-south direction. The valley strike (Figure 2.8) is controlled by an unnamed fault related to the West Branch of the Melones Fault Zone (Jennings 1977). The lower portion of the reach from RM 57 to RM 52.4 is base level controlled by the Murderer's Gorge contraction. The effects of the downstream contraction on sediment deposition at different flow levels can be seen on **Plate 6** and **Plate 7**.

Because of the significance of the contraction to downstream delivery of bed material a detailed topographic survey of the Mammoth Bar site was conducted to enable a site specific hydraulic model (HEC-2) to be constructed to evaluate the effects of the contraction on sediment storage and downstream delivery (Section 3.4).

Sediment delivery to the subreach appears to be somewhat higher on the basis of the size and frequency of tributary alluvial fans than for any of the other subreaches. This may be the result of a combination of factors that include weaker rocks, thicker colluvial deposits, fires, road construction, and off-road vehicle traffic. Numerous relatively small-scale rotational slope failures in colluvium were observed on the valley side slopes (**Plate 8**), and many of the hillslopes show signs of rilling and gullying. Of greater significance is the presence of a major slope failure scarp on the left bank (south) at Poverty Bar (RM 57). It appears that there has been historic slumping of an intervolcanic paleochannel fill (Bateman and Wahrhaftig 1966) that overlies more competent metasedimentary rocks (Figure 2.8). It is not known whether the unnamed fault has had any effect on the instability of the hillslope at this location. Cemented fluvial gravels that comprise the paleochannel sediments crop out at waters edge along the left bank until the bend apex where the metasedimentary rocks crop out. Saturation of the hillslope as a result of construction of the dry dam could result in a major slope failure at this location.

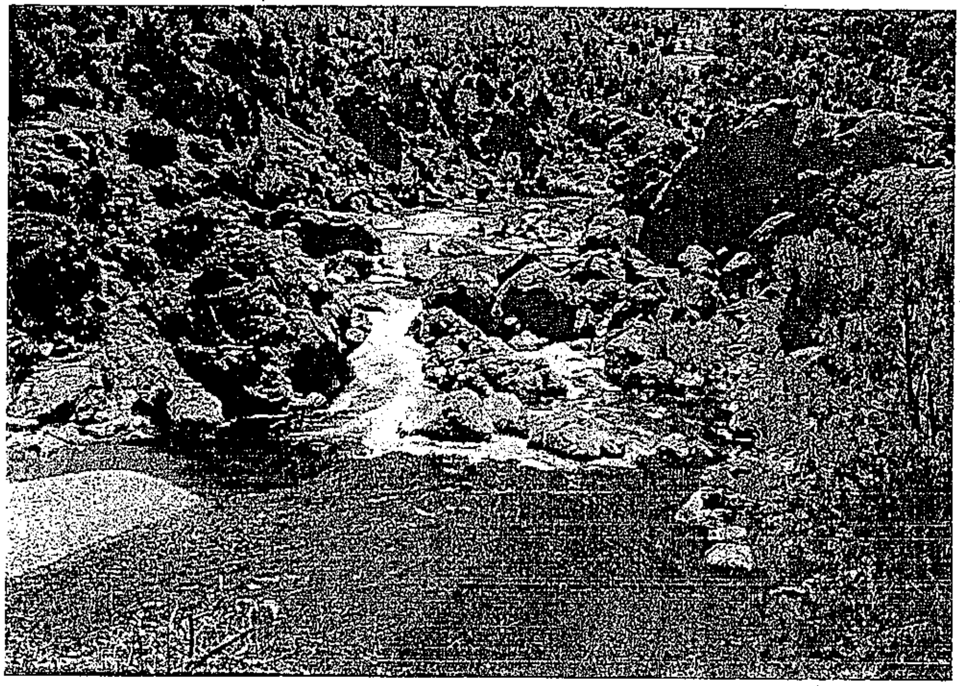


Plate 5. View downstream of Murderer's Gorge, the downstream hydraulic control for Subreach 4 and Mammoth Bar at RM 52.4, Middle Fork American River.

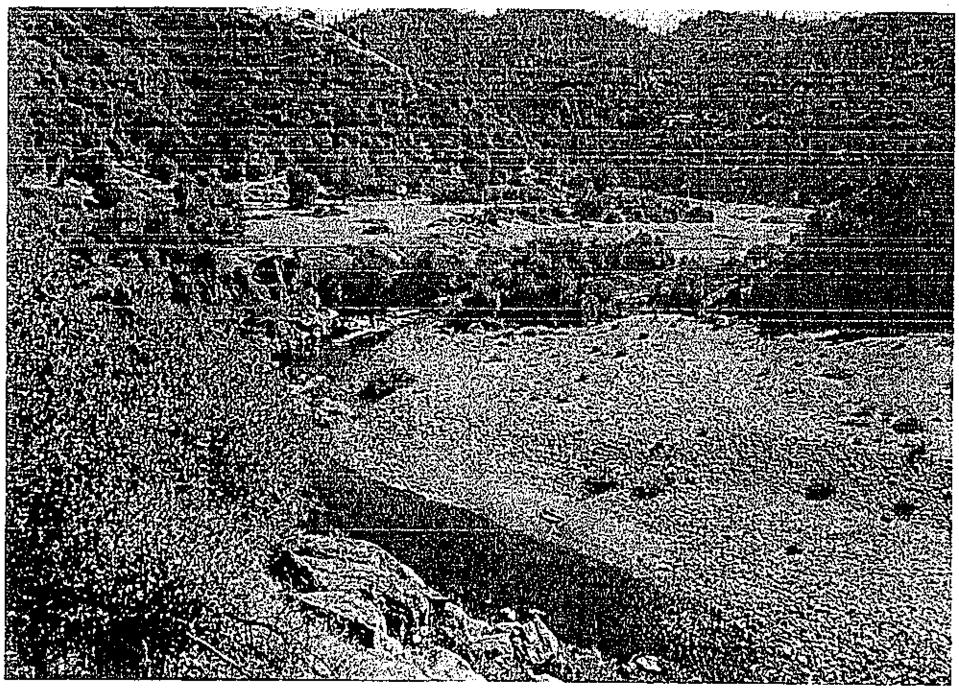


Plate 6. View upstream of Mammoth Bar at ~ RM 53, Middle Fork American River. The bar was placer mined historically, but the sediments have been reworked by subsequent flows.

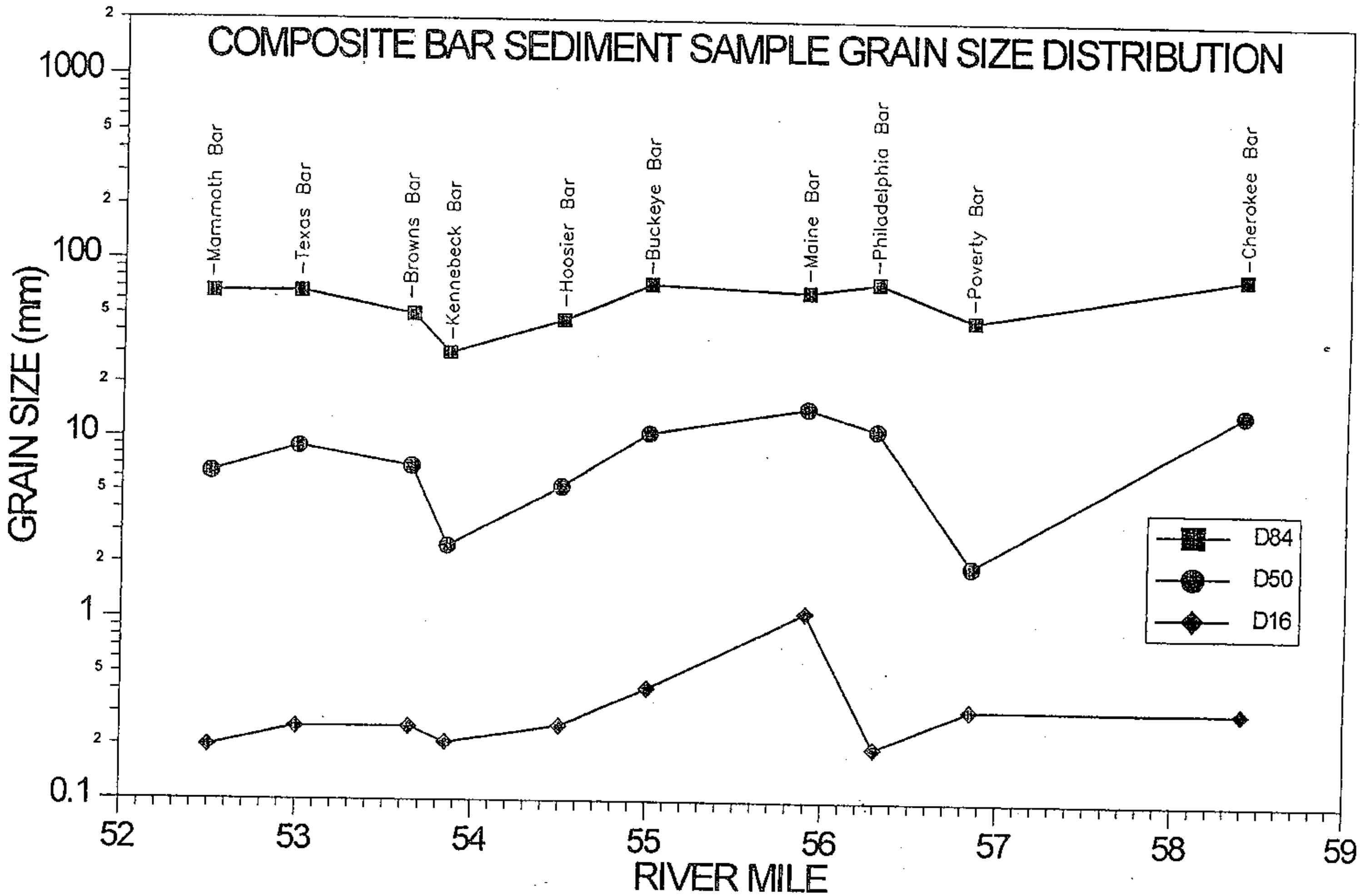


Figure 2.13. Composite bar subsurface sediment gradient parameters for the alternate bars in Subreach 4 of the Middle Fork American River. The data were obtained from Appendix M, COE (1991).

Plate 7. View of fluvially emplaced, horizontally laminated, silty sand dominated sediments on the upper surface of Mammoth Bar at RM 53, Middle Fork American River. The sediments provide confirmation of backwater conditions at the bar during high magnitude flows.

Plate 8. Small recent landslide failure scarp in colluvial material on left bank of Middle Fork American River at RM 53.2. The failure occurred at the bedrock outcrop - colluvium interface located approximately 20 feet above the bed of the river (Subreach 4).

Subreach 5 extends from RM 52.4 to RM 50.3, which is the confluence with the North Fork American River. The valley width is narrower and the channel has less alluvial storage characteristics. Quarry waste rock from the left bank Cool Quarry has and continues to supply gravel-sized angular material to the reach. In contrast to the upstream subreach, there is a single bar identified in this reach (Louisiana Bar) at RM 50.7. The bar is controlled by a downstream bedrock outcrop contraction that causes upstream backwater under a wide range of flows (**Plate 9**). The bar has been extensively placer mined in the past.

Sediment delivery to subreach 5 depends on the upstream hydraulic control at RM 52.4 (Murderer's Gorge) and local tributary contributions (**Plate 10**) that result from hillslope erosion as the result of fires and road construction. Aerial photography dated c.1938 (SCS 1993) indicates that considerably more sediment was in storage in the lower reach of the Middle Fork at that time. Reduction in sediment storage in the reach is probably related to reduced supply from upstream as a result of a number of factors. Recovery of the system from the accelerated sedimentation caused by hydraulic mining is inevitable (Gilbert 1917; Pitlick 1989; Fischer and Harvey 1991). However, creation of Landslide Rapid in 1940 may have accelerated the recovery process and cessation of placer mining in subreach 4 may also have contributed to the recovery.

2.4.3. Geomorphology of North Fork, American River

For the purposes of this project the North Fork of the American River can be subdivided into two subreaches: Auburn Dam site (RM 47.6) to North Fork Dam (RM 52.5) and Lake Clementine (RM 52.5 to RM 57). The North Fork Dam constructed by the CDC as a sediment retention dam in 1940 (Hagwood 1981) effectively prevents any bed load delivery to the lower reaches from the upper watershed even though hydraulic mining debris is still being flushed from the upper watershed.

Subreach 1 extends from the Auburn Dam site at RM 47.6 (**Plate 11**) to the North Fork Dam site at RM 52.5 (**Plate 12**). The confluence with the Middle Fork is at RM 50.3. Upstream of the confluence there is very little sediment in storage within the deeply incised narrow canyon-bounded channel. The hydraulic mining debris that was very apparent on the c.1938 (SCS,1993) aerial photos has been totally evacuated from the reach as a result of clear water releases from the dam. Downstream of the confluence with the Middle Fork the channel is bedrock confined, and there is little sediment in storage within the reach. A Wolman Count at RM 48.2 indicates that the bed material in transport in this reach is relatively fine grained with a D_{50} of 30 mm (Figure 2.12). The gradation of the bed material is almost identical to that at the downstream end of Mammoth Bar and is indicative of the caliber of sediments that are passed through the contraction at RM 52.4 on the Middle Fork.

Failure of the coffer dam at the Auburn Dam site in 1986 caused rapid drawdown in the upstream pool that resulted in a number of landslides downstream of the Middle Fork confluence. The largest of these landslides formed an alluvial fan at RM 49.2L. The fan has probably persisted because the existing tunnel at the coffer dam location causes backwater that extends beyond the fan at discharges when the fan would be eroded under non-backwatered conditions. Moderately thick sand deposits are located within the backwatered reach upstream of the tunnel. However, there is no evidence under existing conditions that sediment delivery to the dam site is high. There are two possible explanations for the relative lack of sediment deposition upstream of the breached coffer dam under existing conditions. First, there is a relatively low supply of sediment to the area, and second there have been very few sediment transporting events since failure of the coffer dam in 1986. The weight of field evidence indicates that the sediment yield is

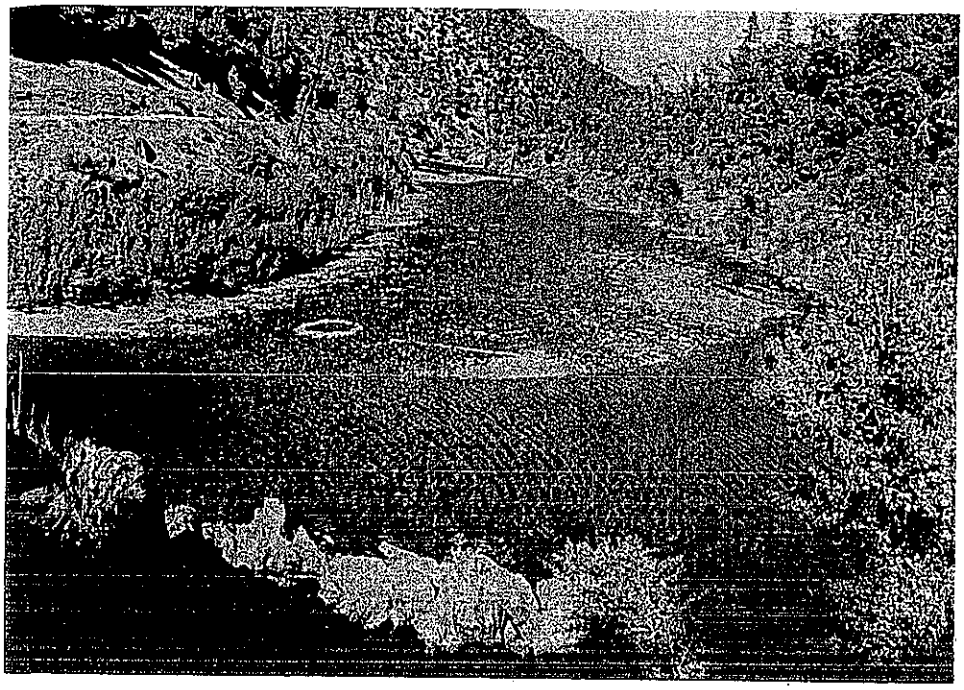


Plate 9. View upstream of backwater-induced sediment deposition in channel of Middle Fork American River at RM 50.7. Bedrock outcrop on both banks creates a sharp bend that causes backwater during high discharges (Subreach 5).

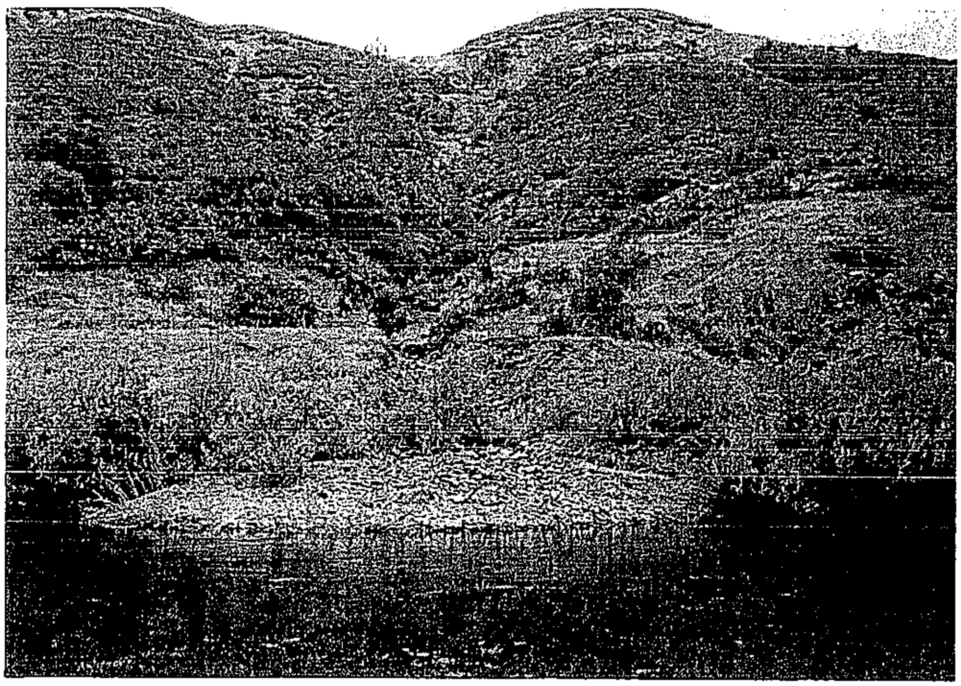


Plate 10. View of small alluvial fan prograding into the Middle Fork American River at RM 51.8 (R). Gullying of the hillslope as a result of road construction and concentration of flows is the source of the fan sediments (Subreach 5).

Plate 11. View upstream of the breached coffer dam at the Auburn Dam site at RM 47.6 North Fork American River. Breaching of the coffer dam has exposed the trace of a fault in the Maidu Fault zone in the outcrop at the base of the right hand valley wall at the breached section.

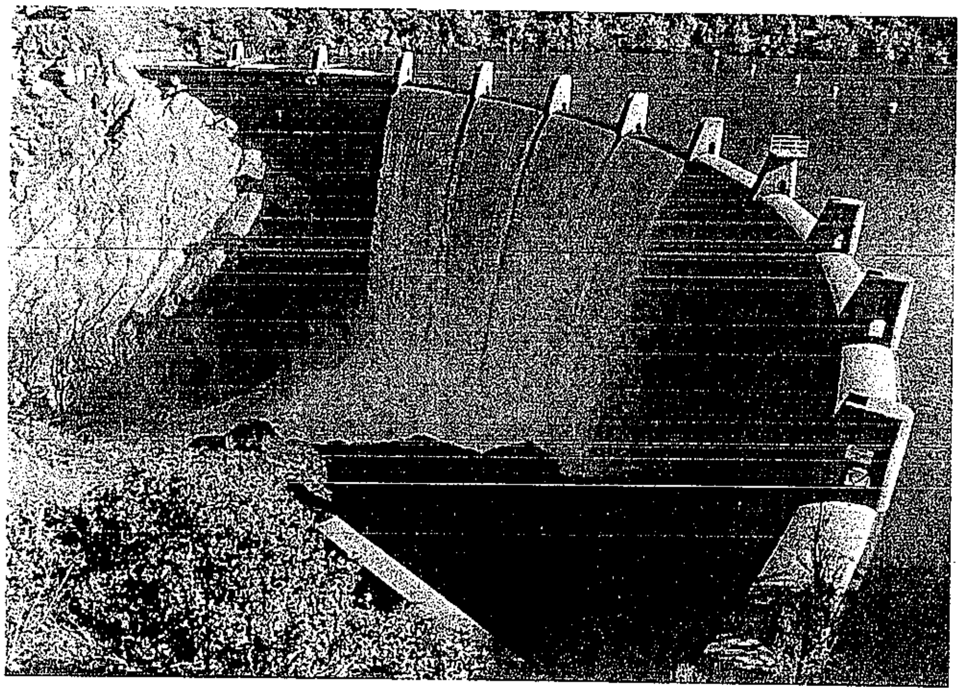


Plate 12. North Fork Dam at RM 52.5 North Fork American River. The dam was constructed by the California Debris Commission in 1940 to retain hydraulic mining sediments.

low, because the North Fork no longer contributes bed material load due to the high trap efficiency of the North Fork Dam, and sediment yield from the Middle Fork is low because of the combined trap efficiencies of the tunnel at RM 73, Landslide Rapid at Rm 61.2, and Murderer's Gorge at RM 52.4.

Subreach 2 encompasses Lake Clementine that extends from RM 52.5 to RM 57. The dam was built by the CDC in 1940 as a sediment retention structure to trap hydraulic mining debris. Initial surveys of the channel were conducted in 1926 and 1935 and a low resolution resurvey of the reservoir was conducted in 1963. During this investigation, 10 cross sections of the reservoir were surveyed, and a centerline profile of the bed of the reservoir was established with a boat-mounted fathometer. The combined information from all of the surveys is presented in **Figure 2.14**. The data show that there is a large wedge of sediment within the reservoir that extends downstream for a linear distance of about 16,400 ft to within 6,000 ft of the dam. However, the data also indicate that there is considerable storage capacity for sediment left in the reservoir; therefore, bed load sediment delivery to the proposed dry dam site is unlikely in the foreseeable future from the North Fork.

Figures 2.15 to 2.19 present cross section profiles of the reservoir from the upstream end (Cross section 1) to cross section 10 located towards the downstream end of the sediment wedge (refer to Figure 2.14 for cross section locations). In the upper parts of the reservoir there is a meandering thalweg at low lake levels (Figures 2.15, 2.16). At about the crest of the sediment wedge, the sediment profile is more symmetrical (Figure 2.17) and the shape of the section remains similar at the remaining sections (Figures 2.18, 2.19), but the depth of water increases progressively downstream. Eight sediment samples were obtained within the reservoir, and three samples were obtained upstream of the reservoir (**Figure 2.20**). The gradation data show expected downstream fining trends from the river to the reservoir and within the reservoir itself. Upstream of the reservoir, the D_{50} of the bar-head sediments is between 25 and 50 mm. Within the reservoir, the D_{50} ranges from 1.5 to 0.09 mm.

The sedimentation data for the reservoir were used to establish sedimentation rates for 2 time periods: (1) 1939-1963 (24 yr) and (2) 1939-1993 (54 yr). The data are presented in **Table 2.2**. The sedimentation rate for the first period is 0.59 ac-ft/mi^2/yr. and that for the second period is 0.22 ac-ft/mi^2/yr. At face value these numbers appear reasonable since it would be expected that the rate would be higher in the earlier period. However, comparison of the total estimated volume of sediment in the reservoir for the 2 time periods suggests that the approximate method of resurvey used in 1963 may have resulted in an over estimation of the sediment volume since the 1963 volume exceeds that from the more precise 1993 survey by approximately 1,000 ac-ft. The lower value is therefore considered to provide a better estimate of sediment yield in a watershed that was severely impacted by hydraulic mining.

2.5. Geomorphology of Lower American River

The project reach of the Lower American River extends from its mouth at Sacramento to Nimbus Dam (**Figure 2.21**).

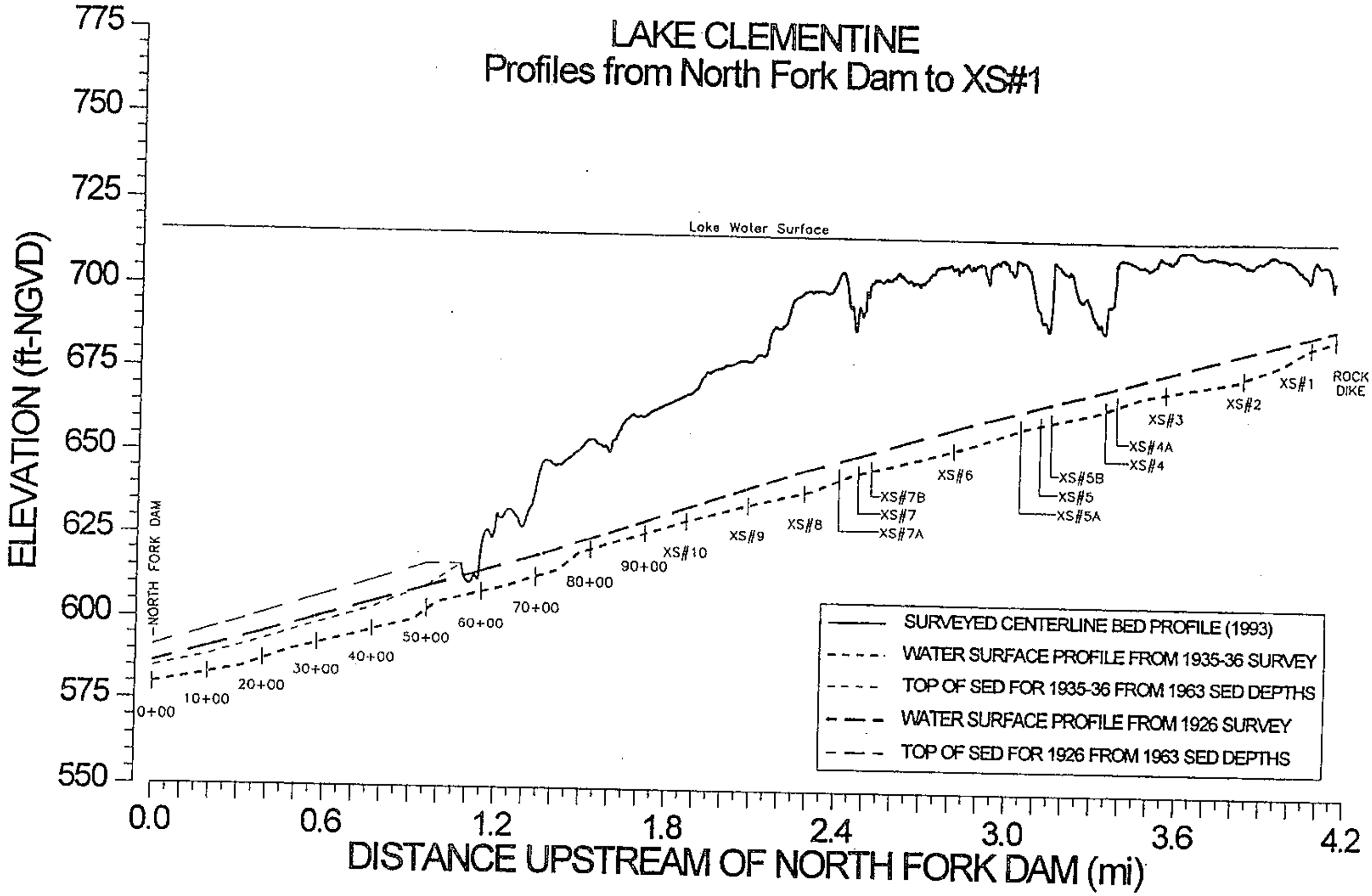


Figure 2.14. Plotted profiles of Lake Clementine on the North Fork American River showing sediment accumulation in the reservoir.

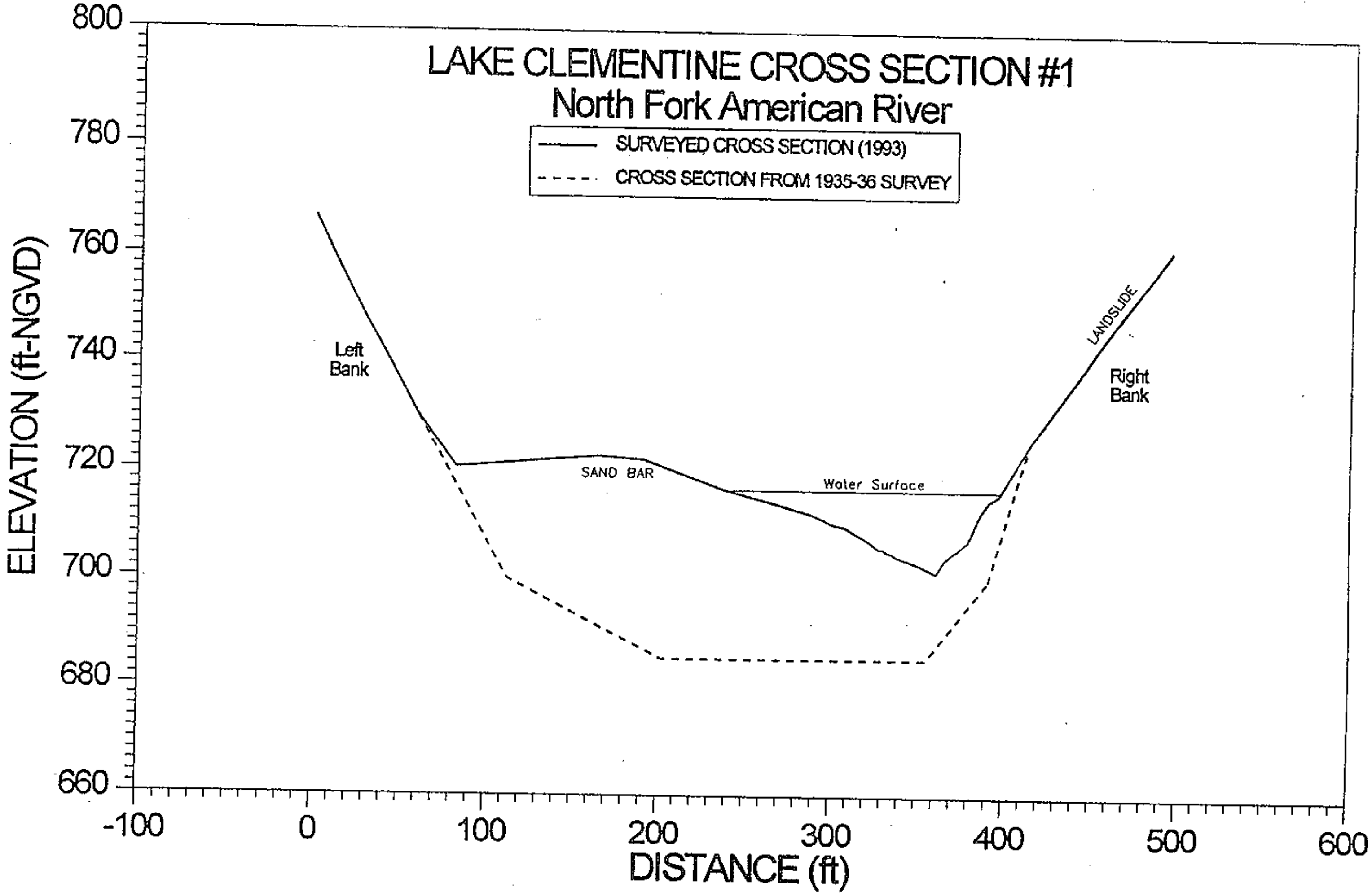


Figure 2.15. Cross section No. 1 profiles for 1993 and 1935/36 surveys of Lake Clementine (refer to Figure 2.14 for location of cross section).

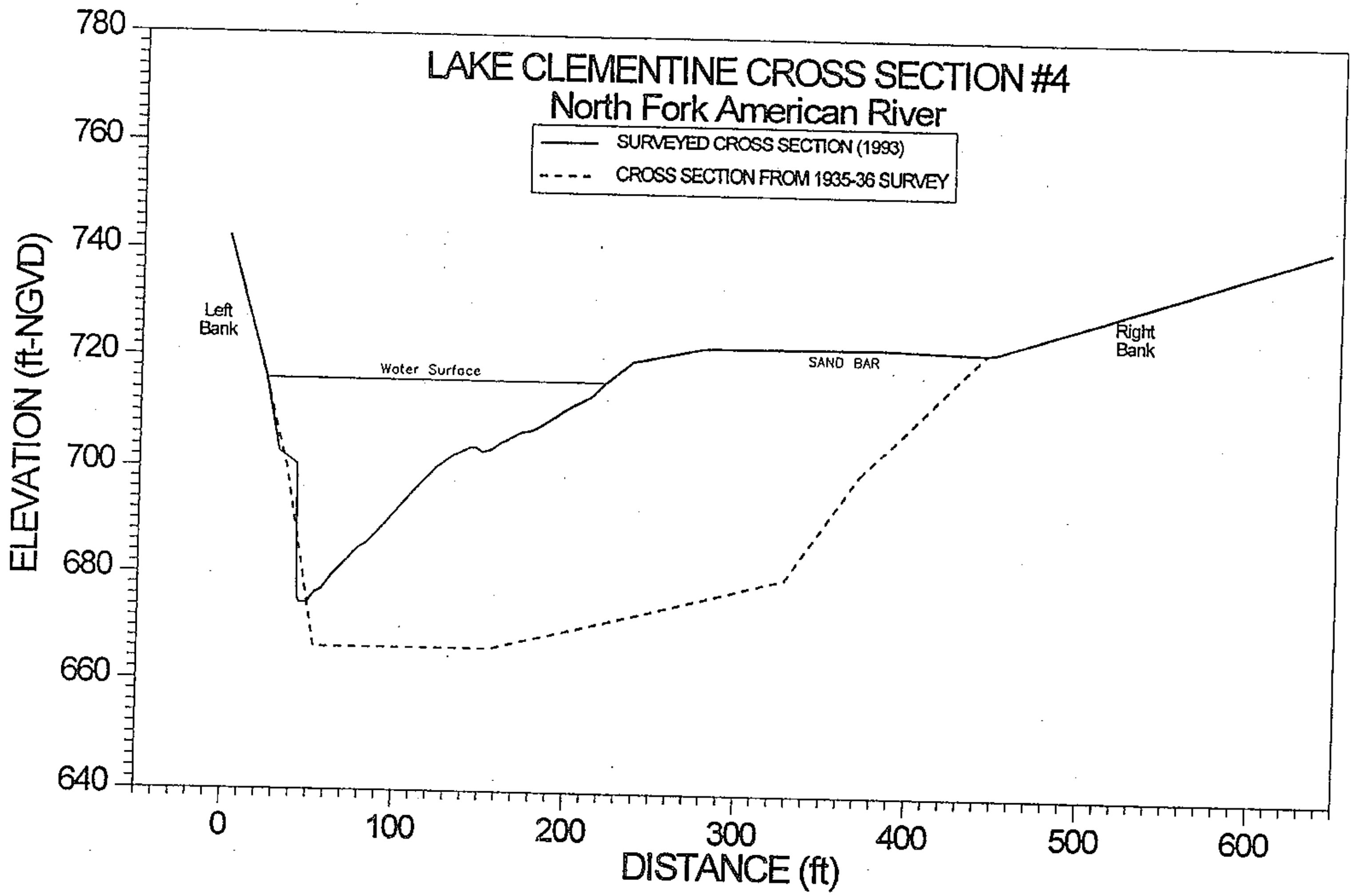


Figure 2.16. Cross section No. 4 profiles for 1993 and 1935/36 surveys of Lake Clementine (refer to Figure 2.14 for location of cross section).

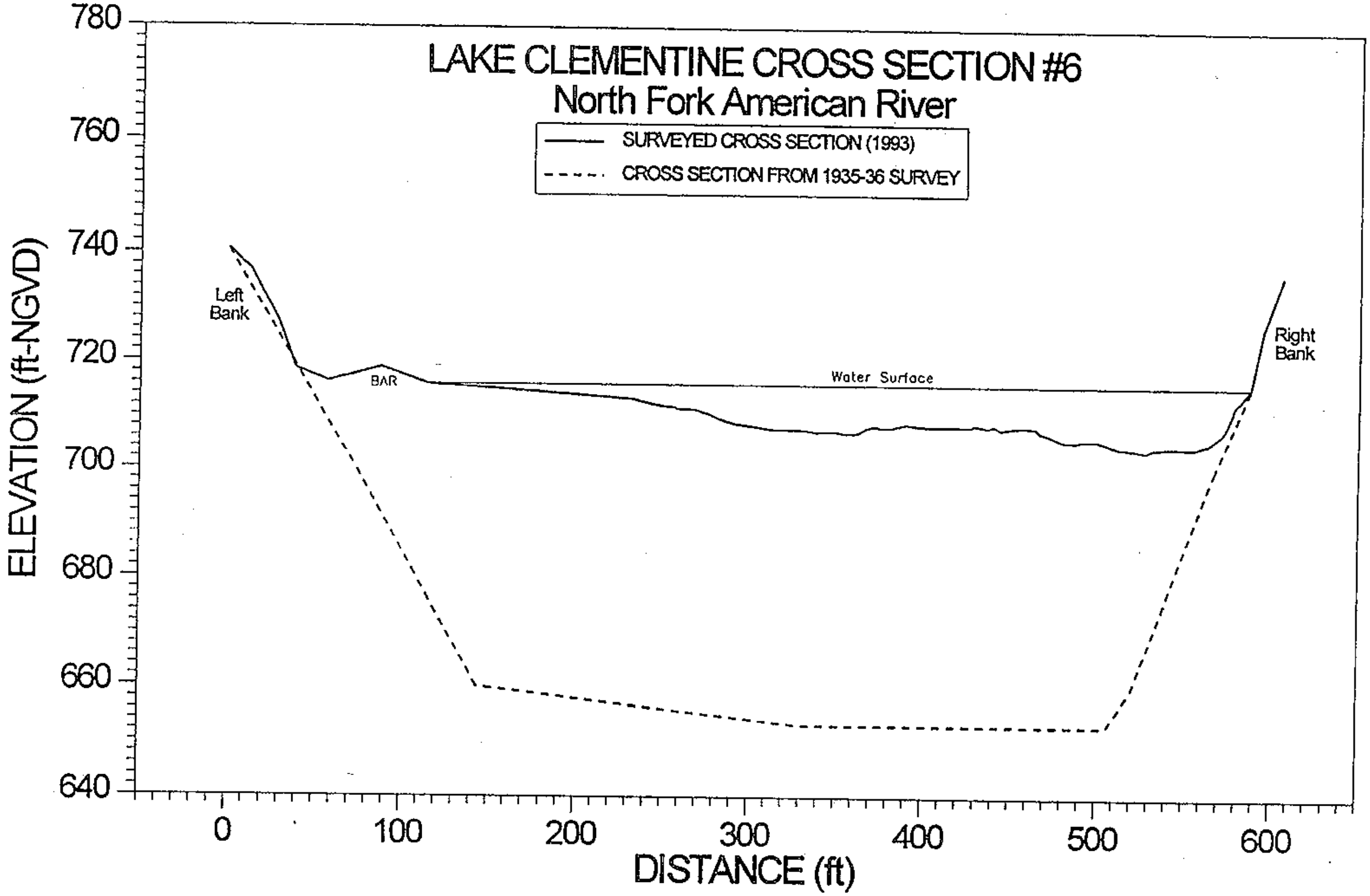


Figure 2.17. Cross section No. 6 profiles for 1993 and 1935/36 surveys of Lake Clementine (refer to Figure 2.14 for location of cross section).

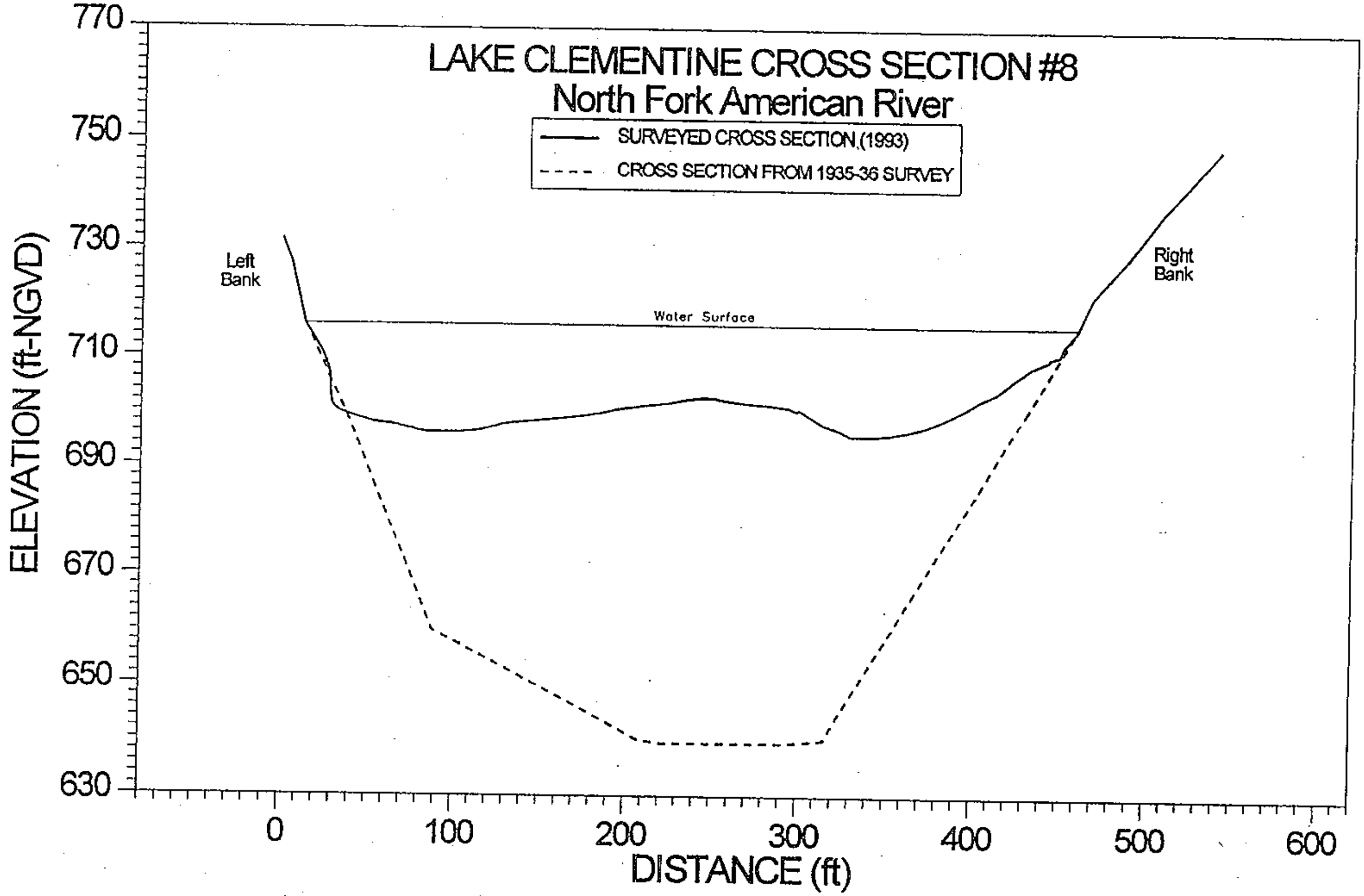


Figure 2.18. Cross section No. 8 profiles for 1993 and 1935/36 surveys of Lake Clementine (refer to Figure 2.14 for location of cross section).

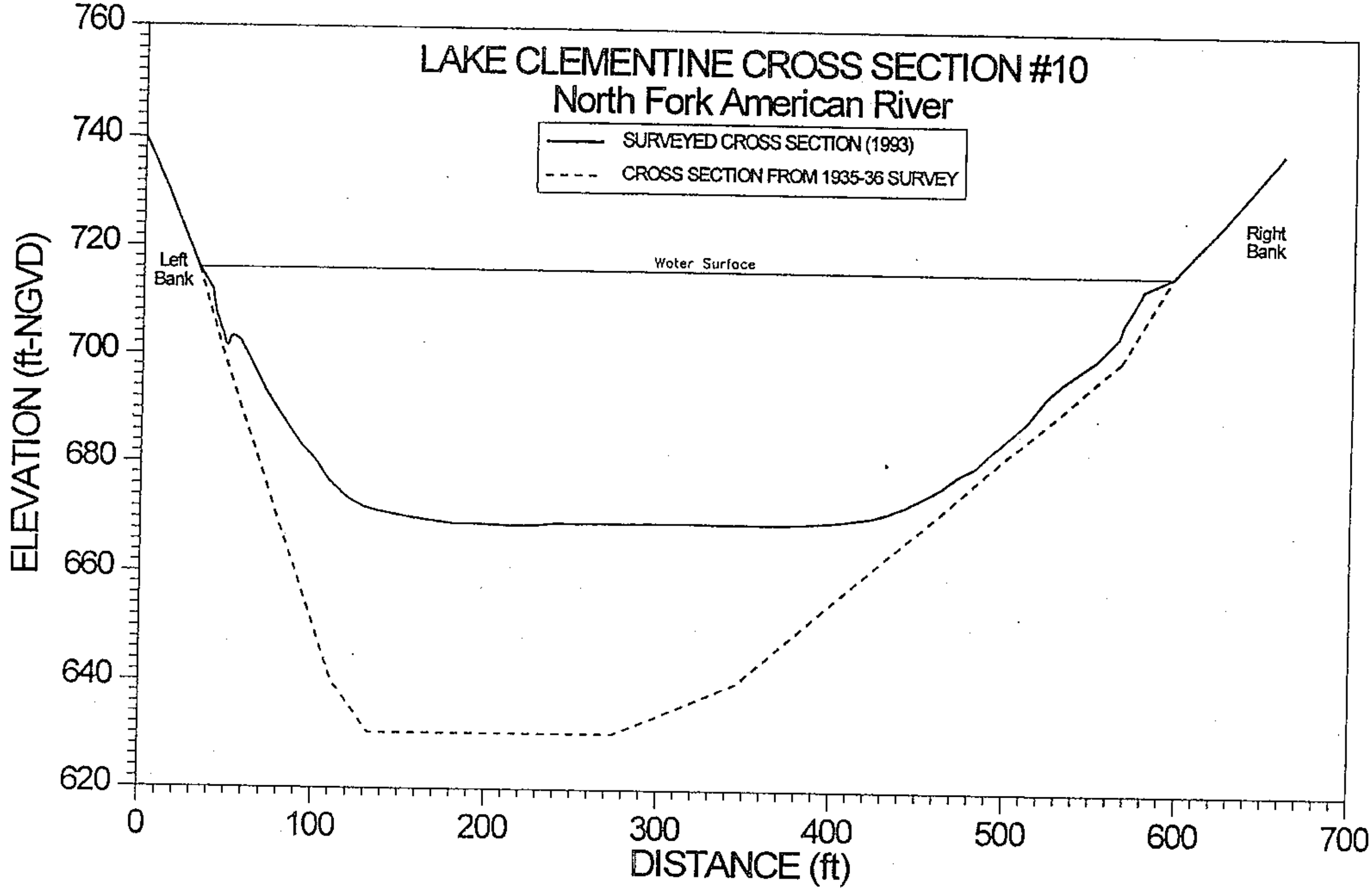


Figure 2.19. Cross section No. 10 profiles for 1993 and 1935/36 surveys of Lake Clementine (refer to Figure 2.14 for location of cross section).

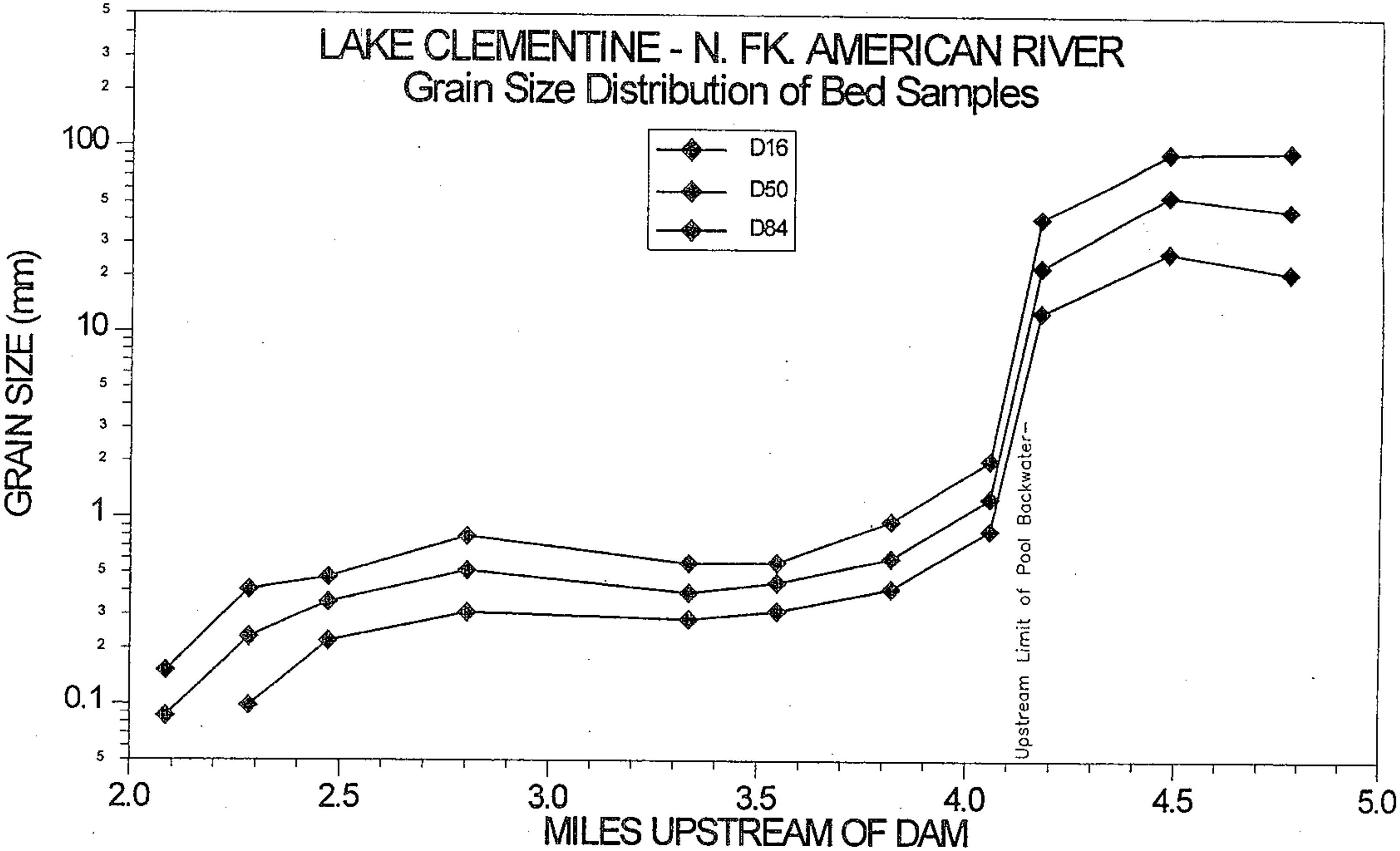


Figure 2.20. Grain size distribution parameters for sediment samples obtained from within and upstream of Lake Clementine.

Table 2.2. Lake Clementine Sedimentation Rates.					
Year Surveyed	Number of Years Since Dam Closure	Drainage Area (mi^2)	Total Estimated Volume (ac-ft)	Estimated Filling Rate (ac-ft/yr)	Estimated Sedimentatio n Rate (ac-ft/mi^2/yr)
1963[1]	24	343	5,000	203	0.59
1993[2]	54	343	4,062	75	0.22

[1] Based on sounding to top and bottom of sediment and visual estimates at 16 cross sections. Volumes calculated using capacity-elevation curves.

[2] Based on hydrographic survey of 10 cross sections with an additional 15 cross sections added using top of sediment from surveyed centerline profile. Volumes calculated by comparing 1926 and 1935 topographic surveys with 1993 hydrographic survey at cross section locations.

2.5.1. Man-Made Modifications to Lower American River

Flood control works on the Lower American River include Folsom Dam at RM 30 and a series of levees and channel improvements between Folsom Dam and the Sacramento River. Folsom Dam was completed in 1955. The dam was constructed by the USACOE and is operated by the USBR as part of the Central Valley Project (USACOE 1989). The dam regulates runoff from about 1,860 sq mi of drainage area. The normal full storage capacity of Folsom Lake is 1,010,000 ac-ft.

The Sacramento River Flood Control Project extends up the American River, and includes 10.8 mi of levee improvements along the south (left) bank upstream of the Sacramento River and 6 mi of levee improvements along the north (right) bank of the river (Figure 2.21). The south bank levee was constructed in 1948 and the north bank levee in 1955. Both levees were constructed to USACOE standards with a crown width of 20 ft, 1V:3H riverside slopes, and 1V:2H landside slopes (USACOE 1989).

Extending from the end of the Sacramento River Flood Control Project levee upstream about 8 mi to the Charmichael Bluffs at (RM 14), the American River Flood Control Project consists of a levee along the north (right) bank of the river (Figure 2.21). The project was constructed by the USACOE in 1958. The levee dimensions are the same as those constructed as part of the Sacramento River Flood Control Project.

Several levees have been constructed on the Lower American River by private land developers. These levees were not constructed to USACOE standards. Non-federal levees are present intermittently on the left bank of the American River from the Mayhew Drain (RM 11) upstream to Sunrise Boulevard (RM 20).

The present day characteristics of the Lower American River have been significantly affected by historical mining activities and upstream dam construction. Hydraulic mining caused approximately 15 to 20 ft of aggradation through the project reach. Dredge

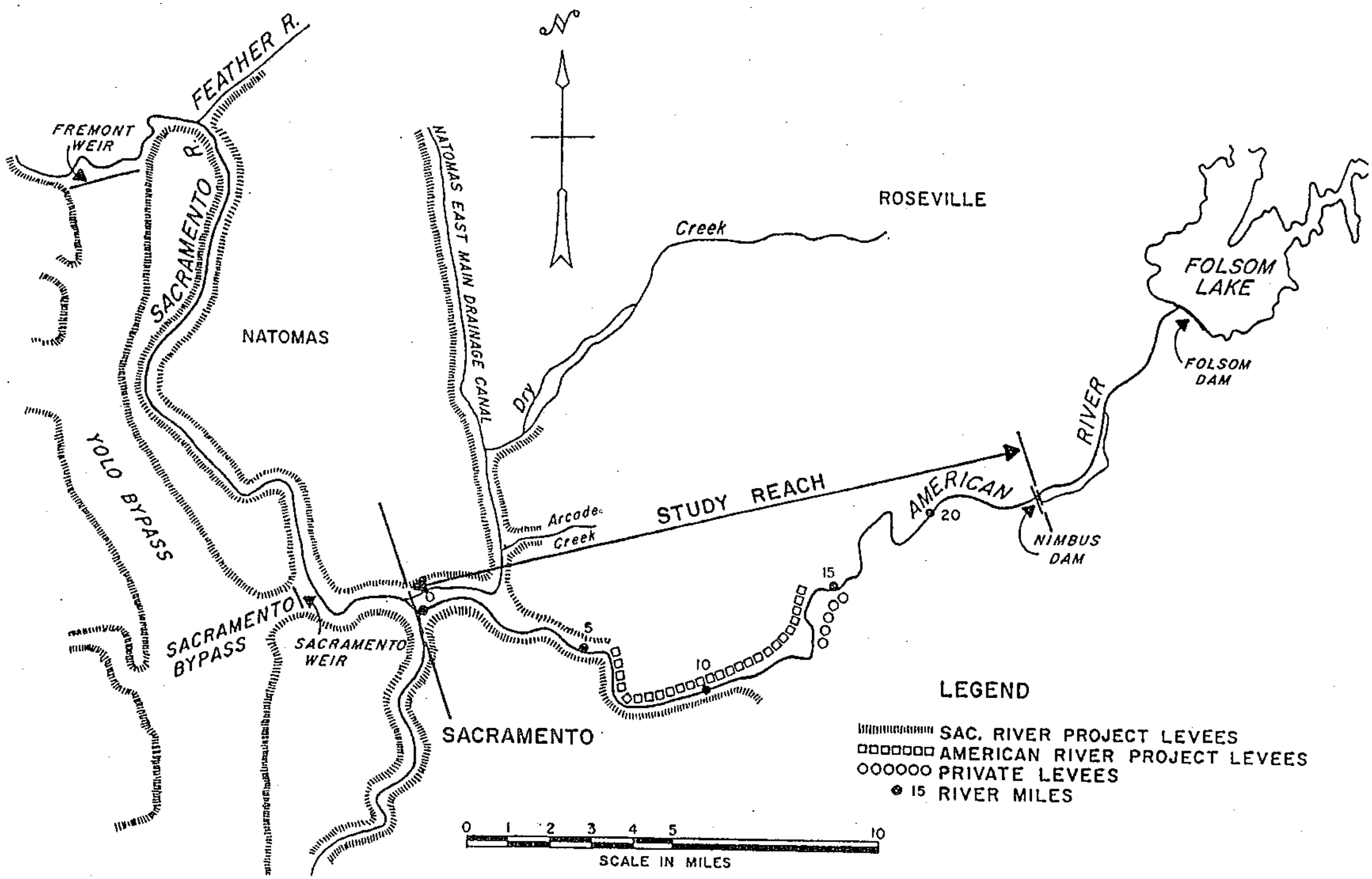


Figure 2.21. General location map of project reach (WET, 1991)

mining for gold as far downstream as Goethe Park (RM 13.5) caused reworking of the floodplain and bars, and it significantly altered the out-of-bank topography. Sand and gravel mining both in the river and on the floodplain has resulted in the development of numerous split flow reaches. Construction of Folsom Dam and Nimbus Dam has effectively eliminated any sediment supply from the upper watershed to the Lower American River. The combined effect of baselevel lowering resulted from recovery of channel bed elevations in the Sacramento River by about 1930 (Meade 1982). Sand, gravel mining, and dam construction has been channel degradation and possibly exhumation of pre-existing bedrock topography within the reach. Channel degradation has required reinforcement of numerous bridge piers especially in the lower reaches where the oldest bridges are located (**Plate 13**).

Based upon general geomorphic characteristics, the project reach was divided into five study subreaches (**Figure 2.22**) (WET 1991). Subreach 1 extends from RM 0 at the Sacramento River to RM 4.8 near Cal State Expo. This subreach is characterized by little sediment storage in bars and intermittently eroding banks. It is strongly influenced by backwater conditions generated at the confluence with the Sacramento River. Subreach 2 extends from RM 4.8 to RM 8.0 and is characterized by sediment storage in large point bars and several midchannel bars. A large percentage of the left bank is revetted within this subreach, which is adjacent to Sacramento State University. Subreach 3, extending from RM 8.0 to RM 11.5, is characterized by a large proportion of split flow and small sloughs, which result from sand and gravel mining. Modesto Formation outcrop is present in minor amounts within this subreach. Subreach 4 extends from RM 11.5 to RM 17.0, and it encompasses sediment storage sites in point bars and midchannel bars that are underlain by Pleistocene-age outcrop. The sediment commonly forms a relatively thin veneer over the strath surface. From RM 17.0 to the upstream study limit at RM 23, Subreach 5 of the Lower American River is characterized by high bluffs of Turlock Lake Formation on the north bank and large, coarse grained bars that commonly consist of dredge spoils.

2.5.2. Sedimentology of Lower American River

The sediments that comprise the active channel and floodplain deposits of the Lower American River can be divided into Recent and Pleistocene ages. The Pleistocene-age units, which form bounding terraces and bluffs along the study reach, are described in Section 2.2. The Recent sediments record deposition in several environments, including floodbasin, floodplain and channel deposits. Sedimentary deposits formed in these environments: floodbasin silt and clay, abandoned channel fill, and vertical and lateral accretion sediments.

Wolman Counts (Wolman 1954) were conducted at bar-head locations within the project reach with the objective of determining the caliber of sediment that had been transported and deposited by the 1993 discharges, which were the highest since the 1986 flood (**Plates 15 and 16**). The peak discharge in the Lower American River in 1993 was 16,200 cfs. Grain size distribution parameters for the sampled sediments in the project reach are shown in **Figure 2.23**. The most downstream sample (RM 0.2) was obtained with a pipe dredge. The data show a general downstream fining trend from Nimbus Dam to RM 4. The D_{50} diminishes from 90 mm at RM 22.3 to 30 mm at RM 4. Downstream of RM 4 the backwater effects from the Sacramento River cause deposition of sand-sized sediment on the bed of the river (D_{50} 0.6 mm). Coarser sediments are probably located beneath the surface sands.

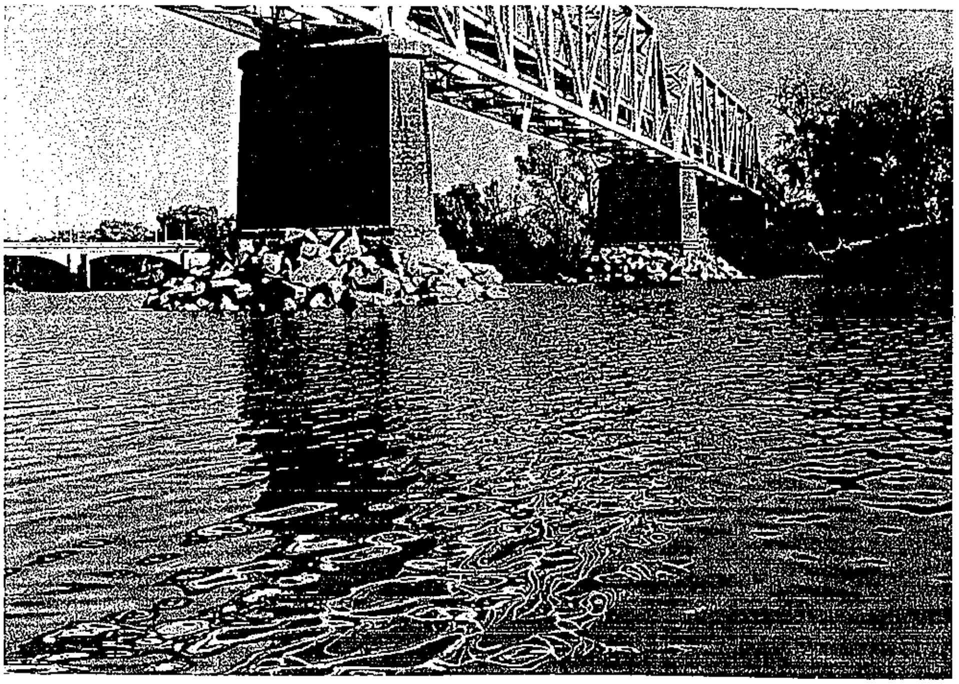


Plate 13. Rock reinforcement of abandoned railroad bridge piers at RM 2.2, lower American River. General bed degradation of at least 15 feet has occurred since 1906 at this location in response to recovery from aggradation resulting from the introduction of hydraulic mining debris.

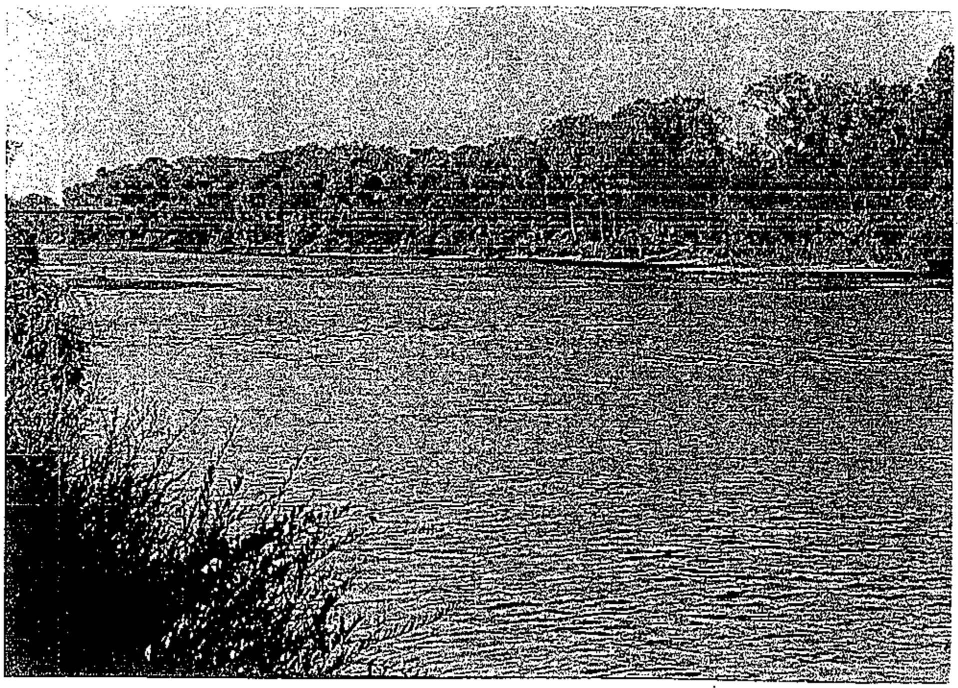


Plate 14. View downstream of a riffle created by Pleistocene-age outcrop in the bed of the channel of the lower American River at RM 9.9. Watt Avenue bridge is located downstream.

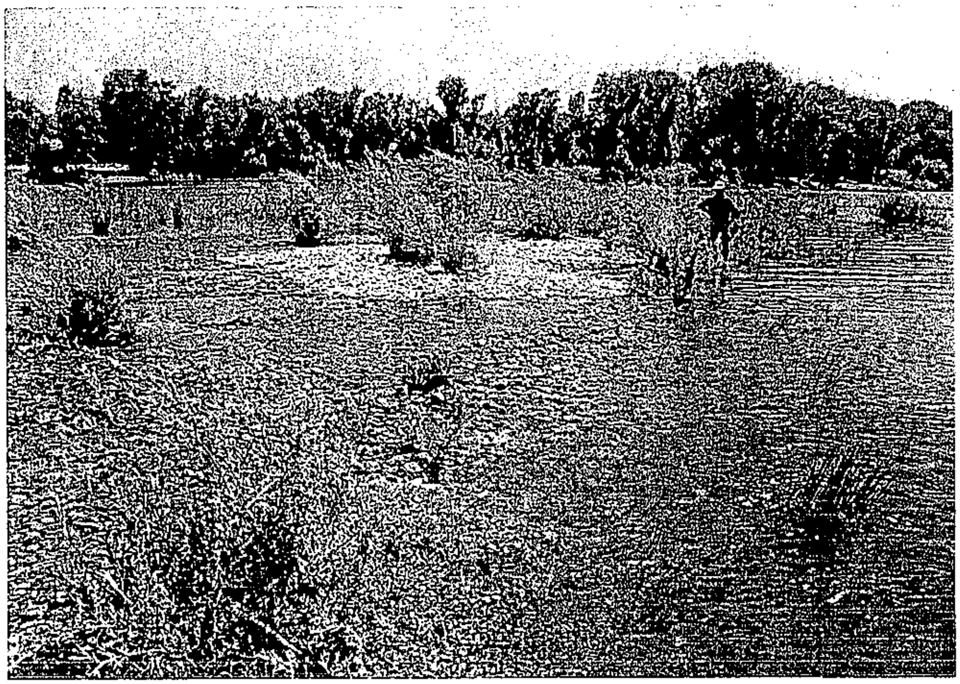


Plate 15. View of mid-channel bar at RM 9.2, lower American River. The gravels and cobbles have been deposited over the willows indicating recent deposition during the 1993 high flows.

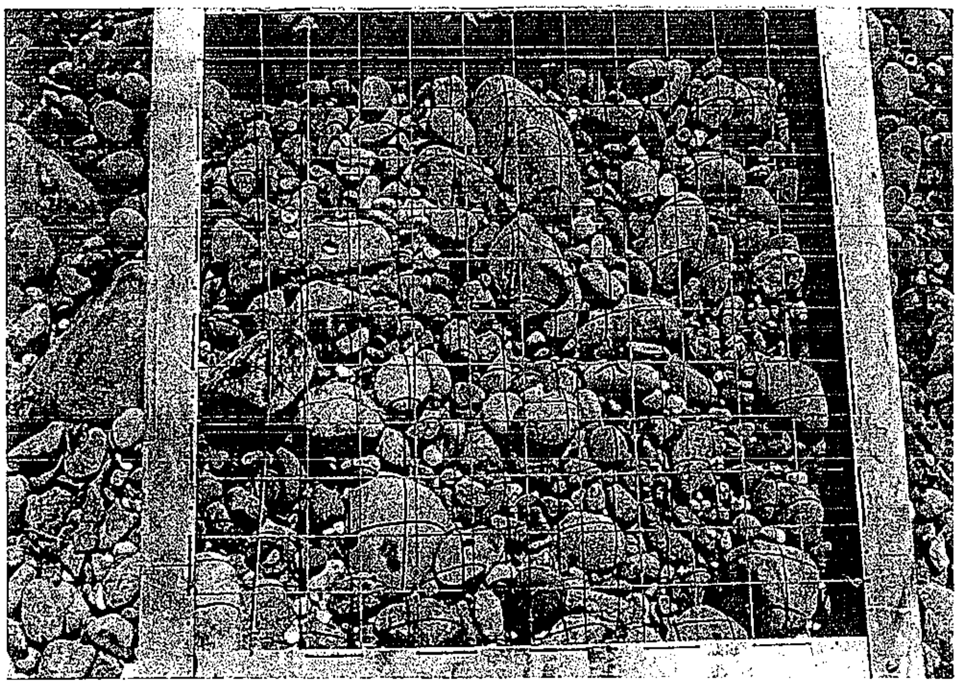


Plate 16. Bar sediments at RM 9.2 lower American River. Site of Wolman Count No. LAR2. Median grain size (D_{50}) is 41 mm. The grid is 2 inch square.

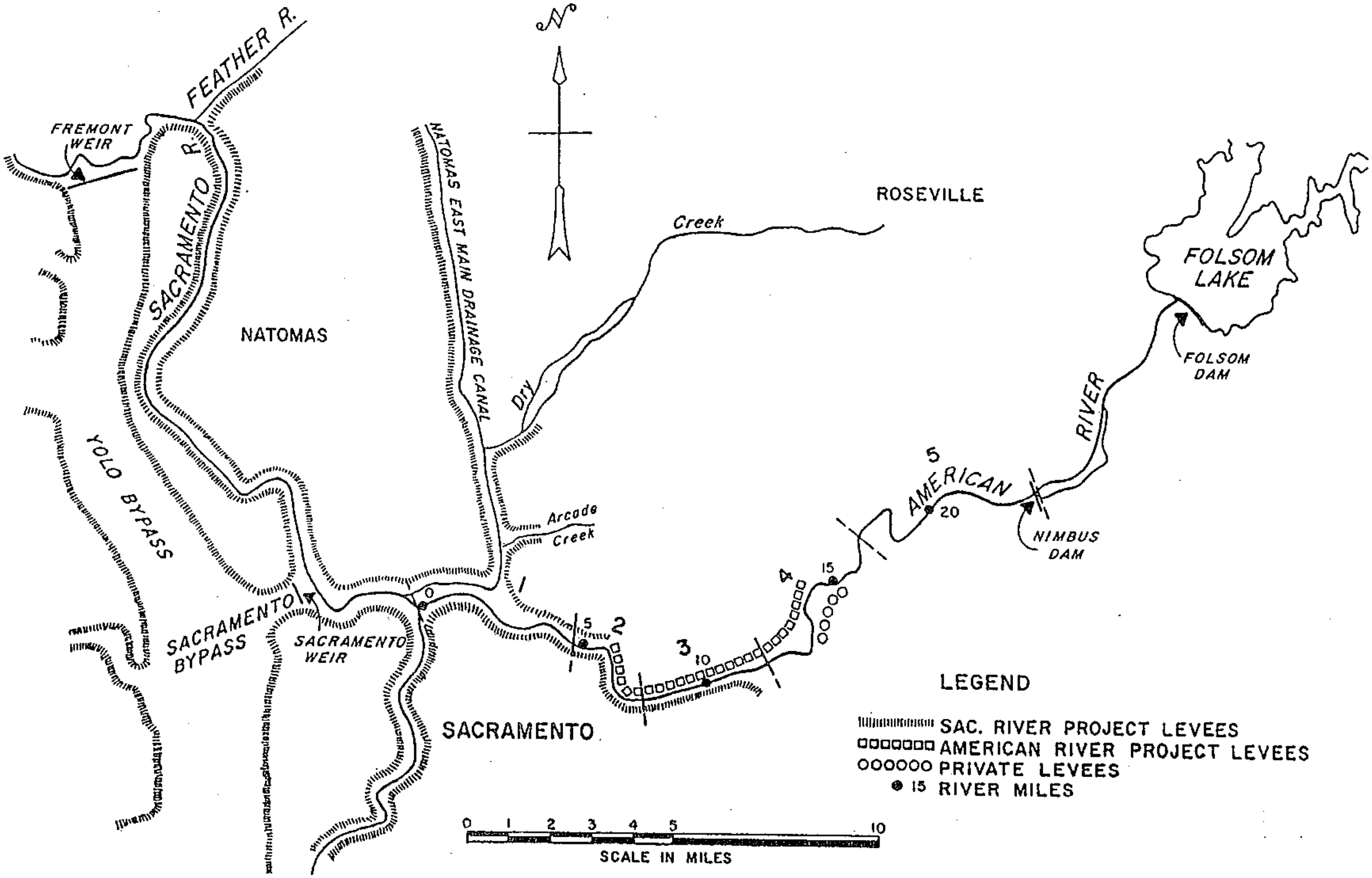


Figure 2.22. General location map of study subreaches (WET, 1991)

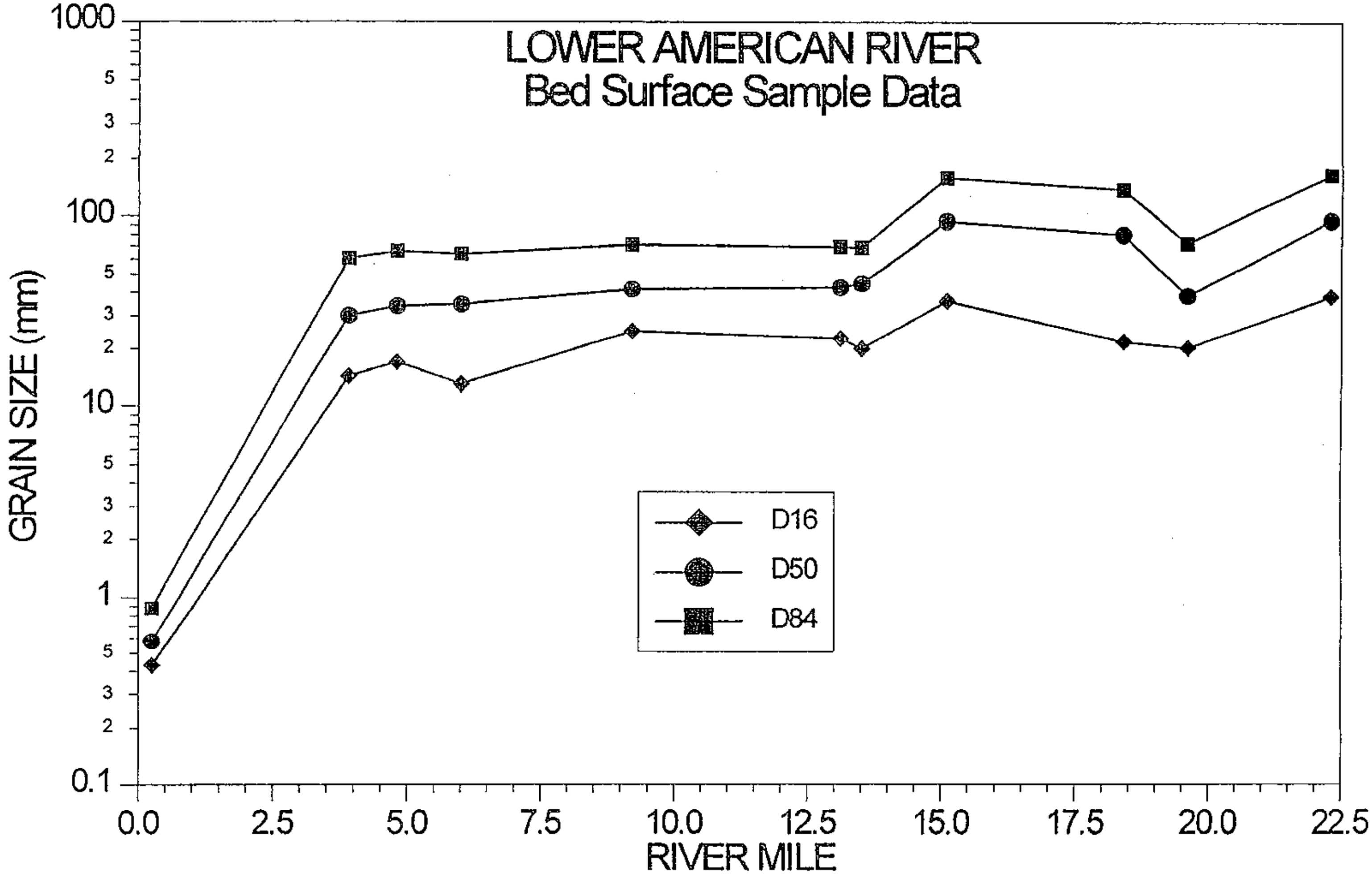


Figure 2.23. Grain size distribution parameters for bar reach sediments that were transported and deposited by 1993 flows in the Lower American River.

2.5.3. Bank Erosion and Channel Migration

Locations of mapped bank erosion on the Lower American River are shown in **Table 2.3.** These sites are broken down in terms of subreach in **Figure 2.24.** Subreaches 1 and 5 contain the largest percentages of eroding bankline, which reaches 13 percent on the right bank in Subreach 1 and 12 percent on the left bank in Subreach 5. Whereas bank erosion within Subreach 1 involves eroding Recent-age American River alluvium, erosion within Subreach 5 generally involves slowly eroding indurated Pleistocene-age alluvial deposits.

The percentages of Pleistocene-age outcrop exposed within the bed and banks of each study subreach are shown in **Figure 2.25**. Individual sites of Pleistocene outcrop are tabulated in **Table 2.4.** The extent of outcrop decreases downstream from more than 30 percent on the right bank in Subreach 5 to 0 percent in Subreaches 1 and 2. This downstream reduction in Pleistocene outcrop corresponds to the gradual emergence of the American River from confining Pleistocene-age alluvial terraces onto a broad floodplain of Recent-age sediments. Furthermore, the general lack of project flood control levees within Subreaches 4 and 5 is related to the presence of high terraces within those subreaches. These upstream subreaches are least susceptible to extensive flooding due to the relatively large channel capacity between the older terraces.

In order to determine historical channel migration through time, aerial photographs containing bank lines of the Lower American River from 1968 and 1986 were compared by WET (1991). Migration rates varied from 1.1 to 13.9 ft/yr within the study reach.

Erosion rates at 30 sites between RM 0 and RM 9.25 (Watt Avenue) were determined from aerial photographs for 4 time periods between 1963 and 1986 by Mitchell Swanson and Associates (MSA 1993). Photography was available for 1963, 1971, 1981, and 1986. At each of the sites the rate of erosion was extensively variable within the different periods. Bank erosion rates varied from 0 to a maximum of 11.2 ft/yr (depending on the site and the time period). Rates were generally highest in the 1981-1986 period and lowest in the 1975-1981 period.

2.5.4. Thalweg Profiles for the Lower American River

Thalweg profiles of the project reach of the Lower American River are available for 1906, 1955, and 1987. The data were compiled and elevations were corrected to a common datum (NGVD) by Murray, Burns, and Kienlen (MBK 1993). The plotted profile for the entire reach are shown in **Figure 2.26** and for the reach contained within the levees in **Figure 2.27.** The profiles indicate that the channel bed elevation downstream of Goethe Park (RM 13) was about 15 to 18 ft higher in 1906 than in 1987. Upstream of RM 13 the 1906 profile does not differ greatly from the 1987 profile except that the 1906 profile is less irregular and variable. The aggradation shown in the 1906 profile is the result of introduction of hydraulic mining debris to the Middle and North Forks of the American River (Hagwood 1981). The 1906 profile coincides with the maximum aggradation of the Sacramento River at Sacramento, which is the baselevel control for the American River (Meade 1982).

Table 2.3. Locations of Significant Bank Erosion, Lower American River.			
RM	Bank	Length (ft)	Comment*
0.25	TRB	250	Int–Discovery Park
0.5	TRB	950	Int
2.1	TLB	100	Int–Scalloped vertical accretion deposits
2.4	TLB	800	Int--Massive failure of clay toe; wavewash, undercutting
3.3	TRB	1,850	Int–Erosion over cohesive toe
3.8	TRB	300	Int--Point bar deposit
4.9	TLB	175	Steep section at channel constriction
5.4	TRB	420	O/S bend way; D/S sewage treatment plant outflow
8.4	TLB	400	Sloughs
8.5	TRB	200	Little levee setback
9.2	TRB	475	D/S Watt Avenue Bridge
12.0	TLB	200	Modesto Formation
16.2	TLB	200	Block failure at toe
17.7	TRB	1,700	Int–Locally severe erosion of Turlock Lake Formation
18.7	TLB	1,175	Int--Modesto Formation on O/S bend way
19.3	TLB	2,100	Int--Vc cobbles; self-armoring toe
20.6	TRB	1,400	Int--Erosion of high bluff of Turlock Lake Formation
20.9	TLB	450	

* Int = Intermittent
O/S = Outside
D/S = Downstream

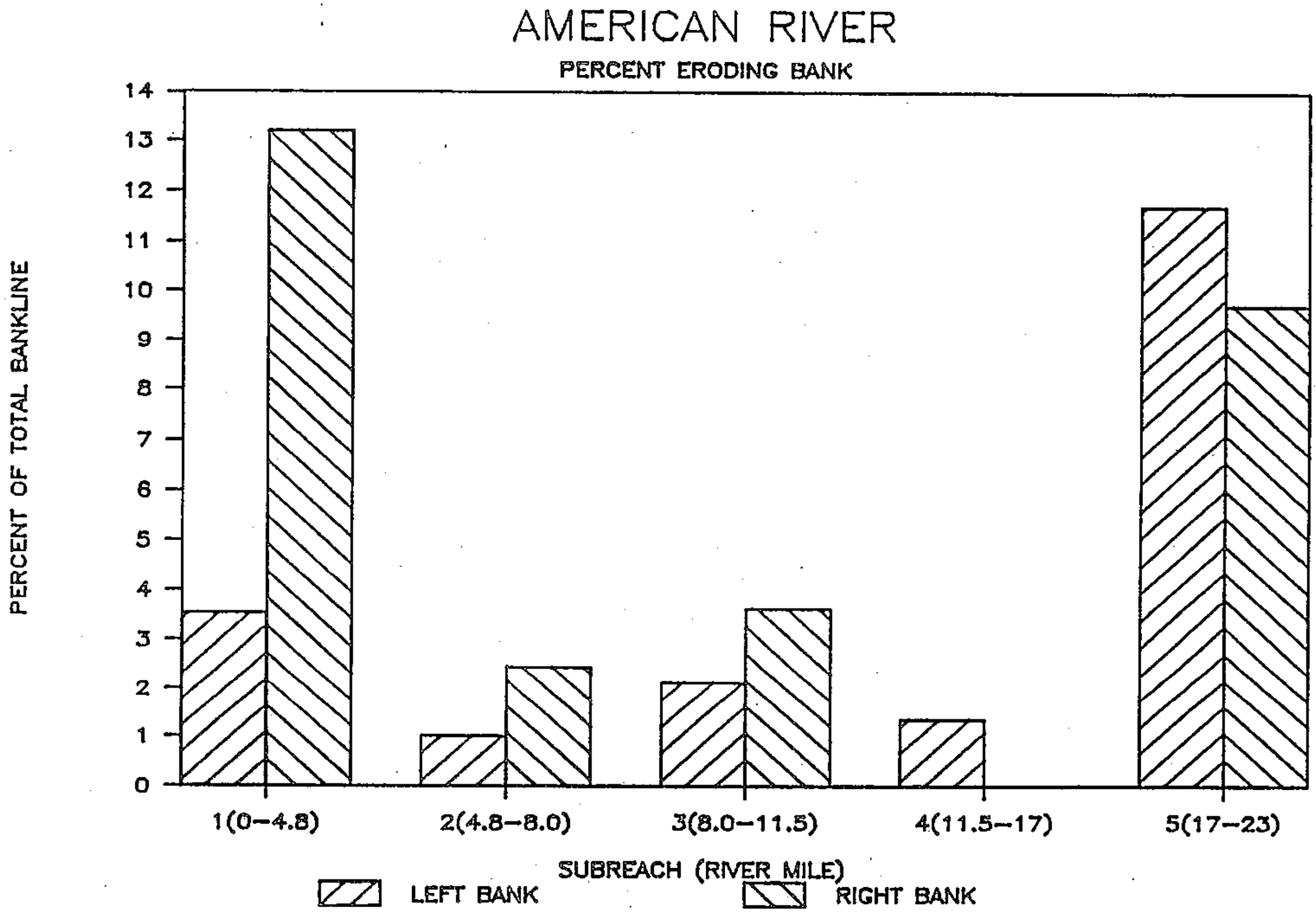


Figure 2.24. Percent eroding bank within study subreaches on Lower American River (WET, 1991).

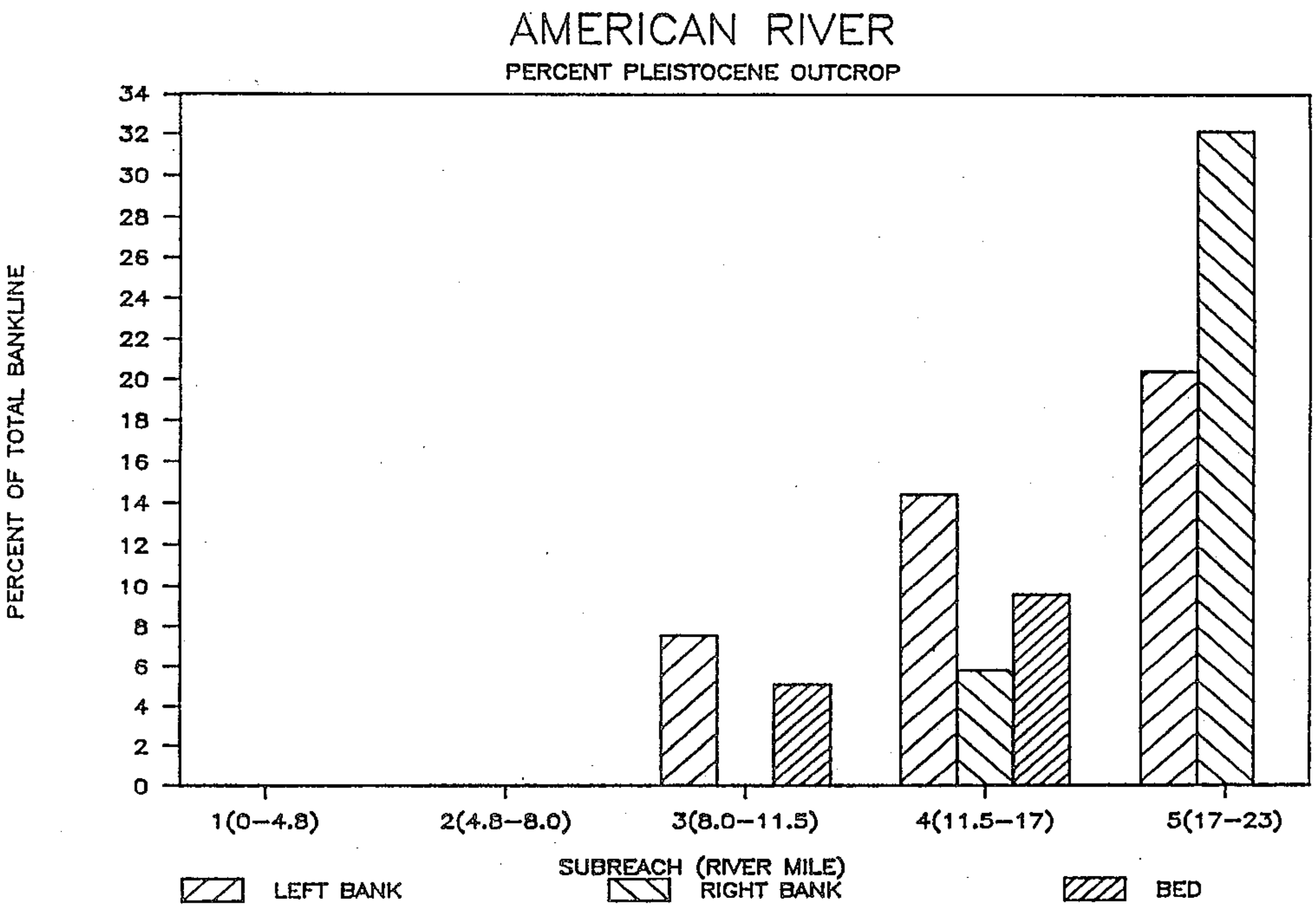


Figure 2.25. Percent Pleistocene-age outcrop exposed in banks and bed of study subreaches, Lower American River (WET, 1991).

Table 2.4. Locations of Pleistocene-age Outcrop in Channel Banks and Bed, Lower American River.			
RM	Bank	Length (ft)	Comment*
9.7	BED	950	Clay bed; Modesto
10.3	TLB	1,400	Block failure; Modesto
12.2	TLB	2,300	High Bank
13.1	TRB	800	Modesto; overlain by cobble/boulder veneer
13.5	TRB	350	Modesto; overlain by bar
13.8	BED	2,800	Fluted bed, high erosional remnants; Modesto
14.4	TRB	1,150	Lower bank; Modesto
15.5	TLB	700	Strath; Modesto
16.0	TLB	1,200	Strath; Modesto
17.5	TRB	6,400	D/S San Juan Rapids; Turlock Lake Formation
18.7	TLB	2,300	Outside bendway; Modesto
20.6	TRB	3,800	High bluff U/S Sunrise Blvd.; Turlock Lake Formation
22.0	TLB	3,000	Conglomerate; proximal fan; Turlock Lake Formation (?)
22.6	TLB	1,200	Turlock Lake Formation (?)

The 1955 profile shows that the river had degraded by between 10 and 18 ft between RM 13 and RM 1.8 (Figure 2.27). Between RM 11 and RM 6, the 1955 and 1987 profiles are not appreciably different, which indicates that degradation in this reach was essentially complete by 1955. Downstream of RM 6, the 1955 and 1987 profiles are different, which indicates that further degradation occurred between the two surveys. Degradation between 1906 and 1955 was probably induced by baselevel lowering at the Sacramento River confluence since the Sacramento River bed elevation reached its present elevation by about 1930 (Meade 1982). Degradation may have been accelerated by sand and gravel extraction from the river. Figure 2.27 shows that further degradation occurred between 1955 and 1987 between RM 13 and RM 11, and downstream of RM 6. Depending on the location, it is apparent that bed degradation up to 30 ft has occurred between 1906 and 1987. The degradation between 1955 and 1987 was probably accelerated by construction of Folsom and Nimbus Dams that cut off the upstream sediment supply to the Lower American River.

Both Figures 2.26 and 2.27 indicate that the present (1987) profile of the river is highly irregular. Upstream of RM 14, the channel bed elevation is controlled by irregularities in the Pleistocene-age materials that crop out intermittently all the way to Nimbus Dam (Plate 18). Downstream of RM 14 the bed irregularity can also be attributed to outcrops

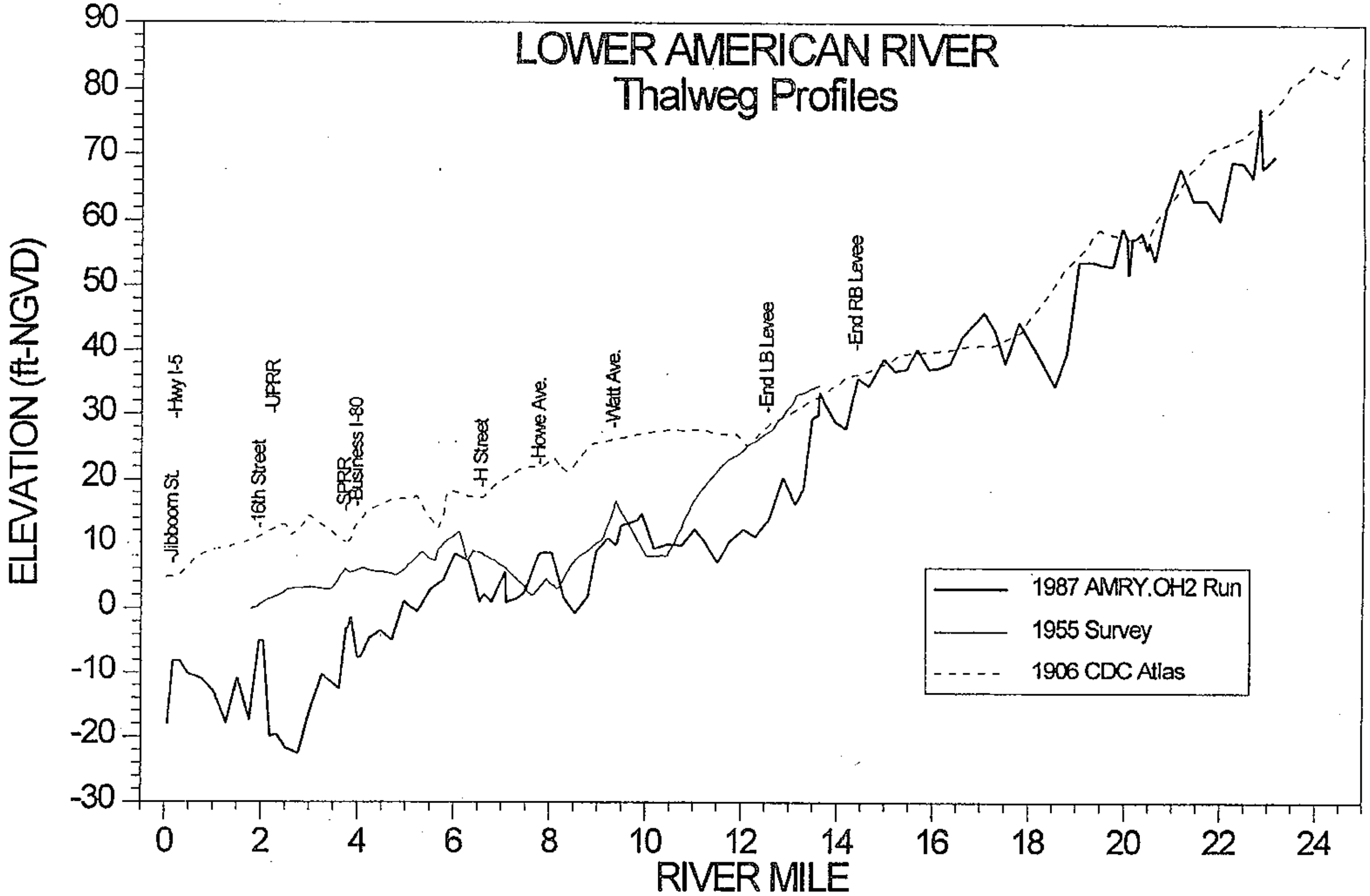


Figure 2.26. Thalweg profiles for 1906, 1955, and 1987 for Lower American River. Data and profiles provided by Murray, Burns, and Klenlen (1993).

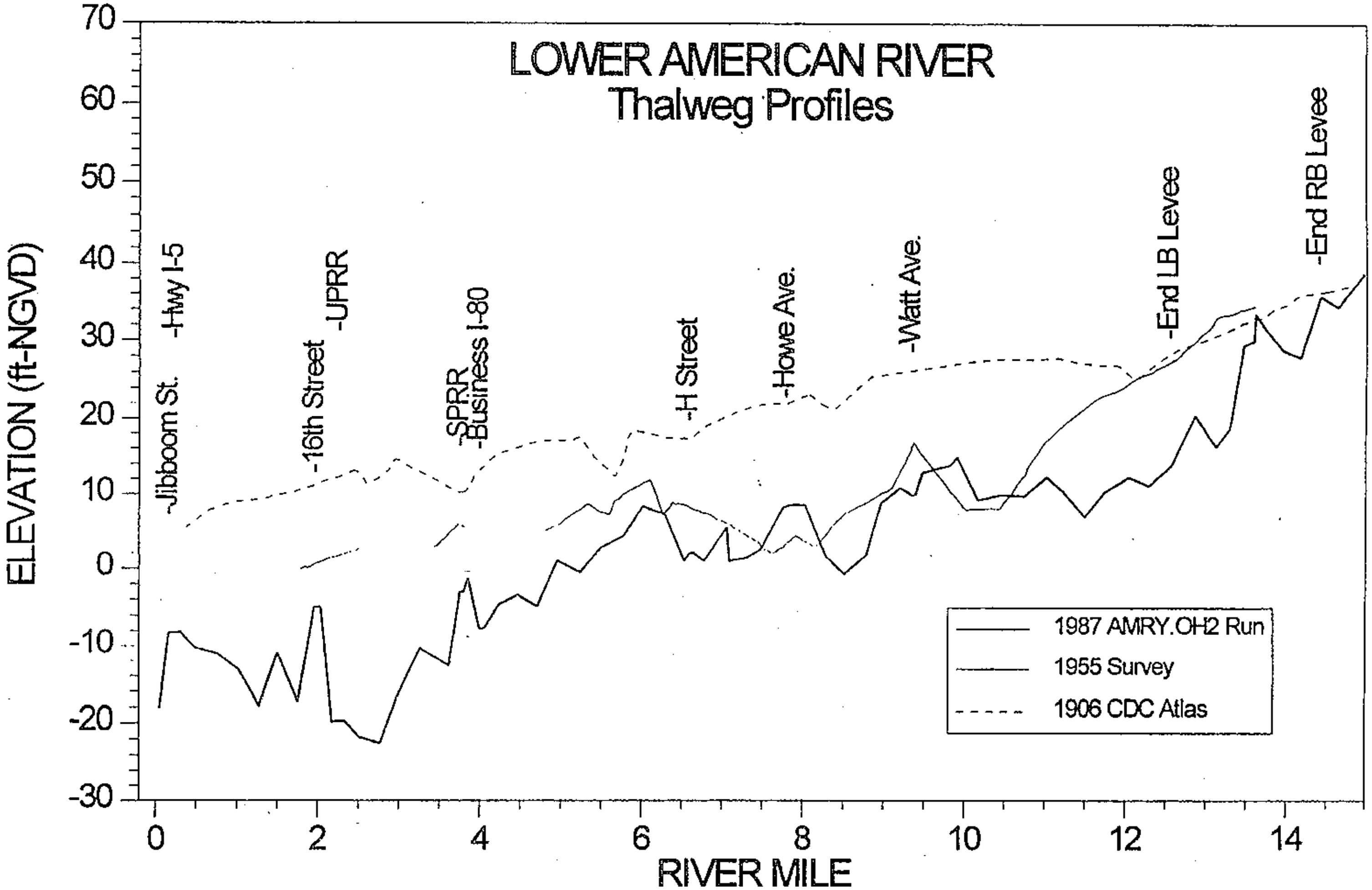


Figure 2.27. Thalweg profiles for 1906, 1955, and 1987 for lower 14 miles of Lower American River. Data and profiles provided by Murray, Burns, and Kienlen (1993).

of erosion-resistant materials. Pleistocene-age materials crop out in the bed of the river at RM 13.5 (**Plate 17**) and RM 9.9 (**Plate 14**), and these appear to be providing local baselevel control for the channel. Fine-grained abandoned channel fills are located at RM 6 and RM 4.5. At RM 4.5 the channel fill has been eroded and no longer provides a local baselevel control. It is likely that the channel fill at RM 6 will also erode eventually, and this will cause some local bed adjustment upstream. Currently, there is bed material storage immediately upstream of RM 6, and bed erosion will lead to downstream transport of these sediments. However, the bend downstream also exerts a major control on the hydraulic energy in this reach, and provided the existing planform is maintained, the rate of erosion of the channel fill may be slow.

The future status of the bed in the project reach is dependent on the inflow of sediment downstream of Nimbus Dam (**Plates 19, 20**), the caliber of the bed materials in the reach, and the erosion-resistance of the Pleistocene-age outcrop in the bed of the channel. Since the Sacramento River bed elevation has remained relatively constant since 1930 (Meade 1982), it is unlikely that baselevel will change significantly in the future. Because of drought conditions, flows in the Lower American River have been low since the 1986 flood. However, field evidence indicated that the 1993 flows transported and deposited bed material at different locations (**Plate 15**).

In an attempt to determine whether there had been any significant changes in bed elevation since 1987, 10 cross sections were selected for resurvey in 1993. The cross sections were located at RM 3.9, 5.4, 5.67, 6.01, 6.53, 9.21, 13.47, 13.59, 15.2, and 19.73. The original surveys in 1987 had been conducted by California Department of Water Resources (DWR) (RM 0 - RM 14) and the COE (RM 14 - RM 22.5). In the lower reach, exact recovery of the cross sections was not possible but attempts were made to reproduce the cross sections as closely as was possible. With these limitations, the resurvey data indicate that there was essentially no significant change at RM 3.9, 5.4, 5.67, 6.01, 6.53, 13.59, 15.2, and 19.73. Minor aggradation was recorded at RM 9.21 and minor degradation at RM 13.47.

2.5.5. Existing Bank Protection

Bank protection methods presently employed within the project reach of the Lower American River include rock revetment, river cobble revetment, concrete walls, saccrete, gabions, stone dikes, and concrete rubble. **Table 2.5** lists existing bank protection emplacements as retrieved from USACOE records (WET 1991). **Figure 2.28** shows the extent of existing protection within each study subreach. The extent of revetment is greatest within Subreach 2 (RM 4.8 to RM 8.0) where the levees are relatively close to the channel and Sacramento State University is located landward of the left bank levee. Some of the rock revetment adjacent to Sacramento State University was constructed in 1986 under emergency status. Figure 2.28 shows that the extent of bank protection presently in place on the Lower American River is greatest in Subreaches 1 and 2 where Pleistocene outcrops are not present to provide lateral channel stability.

Bank protection performance on the Lower American River has in general been satisfactory. **Table 2.6** lists locations of damaged bank protection along the river (WET 1991). The only damaged rock revetment mapped is located at RM 0.45R where rocks have been dislodged due to extensive river access. The majority of damaged protection consists of toe failure of river cobble riprap, or toe failure of non-USACOE concrete rubble or concrete walls. A total of 1,500 linear ft of bank protection was mapped as damaged.

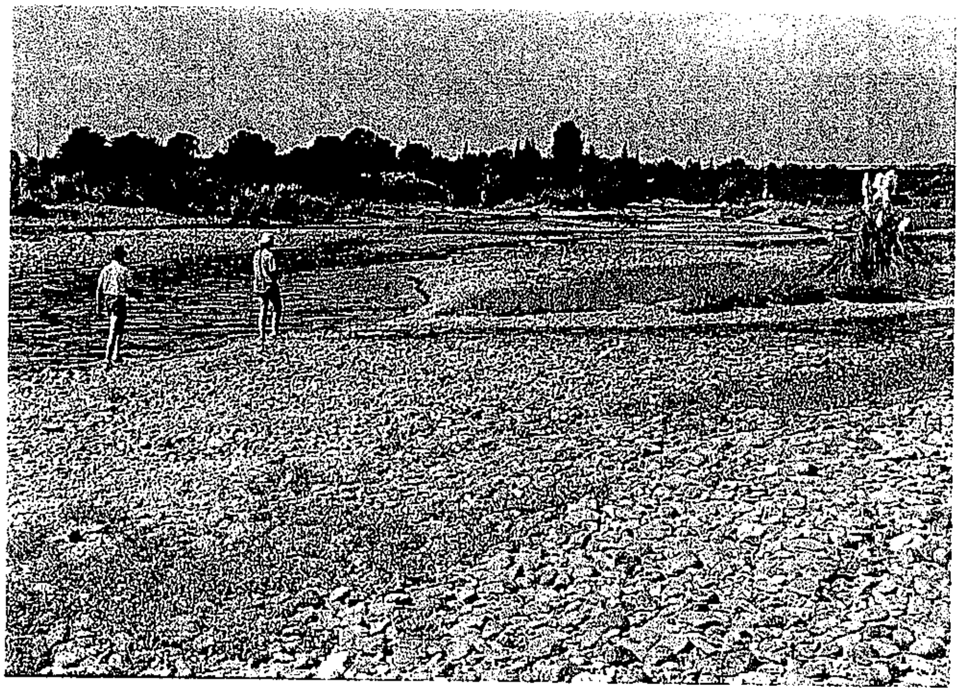


Plate 17. View upstream of low-water braided reach of lower American River at RM 13.5. The gravels form a veneer over outcrop of Pleistocene-age bedrock. Goethe Park Bridge piers in the background are founded on outcrop.

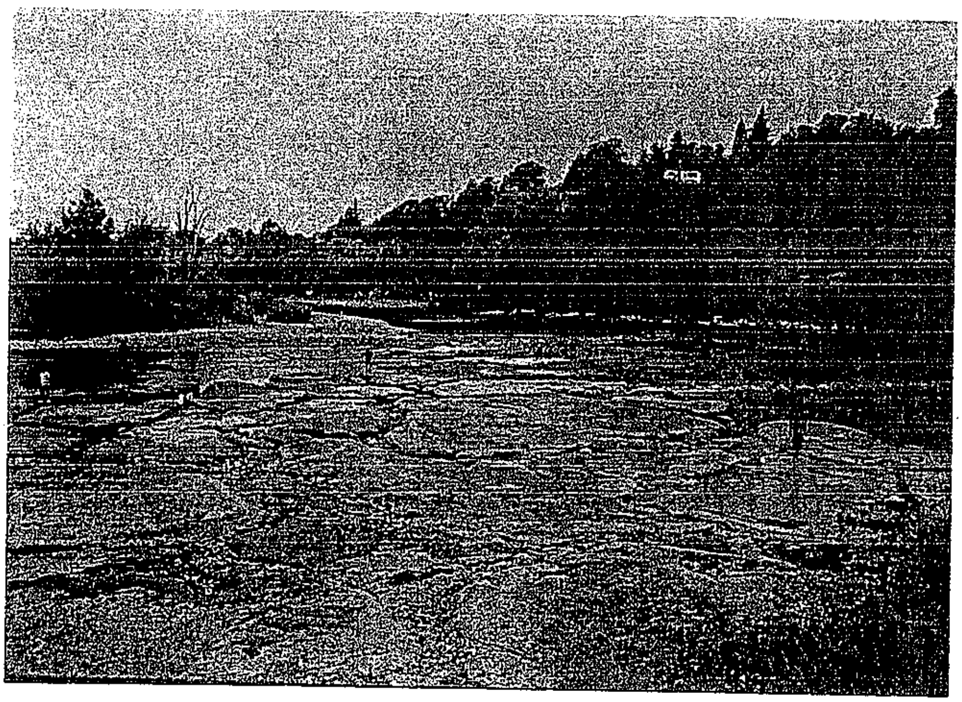


Plate 18. View downstream of San Juan rapids at RM 18.0 on the lower American River. Pleistocene-age bedrock outcrop forms the rapid. Turlock Lake Formation underlies the high terrace on the right bank.

Plate 19. View of right bank of lower American River at RM 18.4. The bank is composed entirely of sand that was probably concentrated by floating dredges. Bank erosion is the primary source of sediment for the lower river because of the effects of both Folsom and Nimbus dams.

Plate 20. View of right bank of lower American River at RM 20.6 where there are minor failures of the high terrace. Terrace erosion is probably the result of lawn irrigation.

Table 2.5. Existing Bank Protection, Lower American River. Revetment Dates Retrieved From USACOE Records (WET 1991).

River Mile	Bank	Length (ft)	Type	Date
0.65	TLB	3,120	Local covered riprap	1988
0.90	TLB	110	Cobble at outlet	?
1.50	TRB	3,000	Scattered concrete debris	?
1.95	TLB	868	River cobble	1952
1.95	TRB	800	Concrete debris	?
2.45	TLB	8850	River cobble	1951
2.55	TLB	1,375	River cobble	1948
4.20	TLB	1,537	Emergency bank protection/rock	1986
4.80	TLB	495	River cobble	1969
5.10	TRB	950	River cobble	1967
5.45	TRB	110	Concrete	?
5.55	TRB	60	Concrete	?
5.60	TRB	2,287	River cobble	1959
5.95	TLB	1,100	River cobble	1952
6.22	TLB	1,550	River cobble	1970
6.55	TRB	1,075	River cobble	1959
6.65	TLB	6,500	River cobble	1959
6.80	TLB	810	Emergency bank protection/rock	1986
7.25	TLB	7,400	River cobble	1959
7.30	TRB	960	Rock	Date and origin unknown
7.60	TRB	1,600	Rock	?
7.75	TRB	240	Bridge bank protection	?
8.80	TRB	400	River cobble	Date and origin unknown
9.05	TRB	2,302	River cobble	1951
9.20	TRB	280	Rock	Date and origin unknown
13.65	TRB	200	Concrete and cobble debris	?
13.80	TRB	180	Private rock	?
13.90	TRB	240	Private concrete bank	?
14.20	TRB	910	Concrete bank	?
14.40	TRB	1,160	Concrete wall	?
15.00	TLB	1,320	Rock/gabions	Date and origin unknown
15.3	TLB	1,750	Cobble/dikes	1970

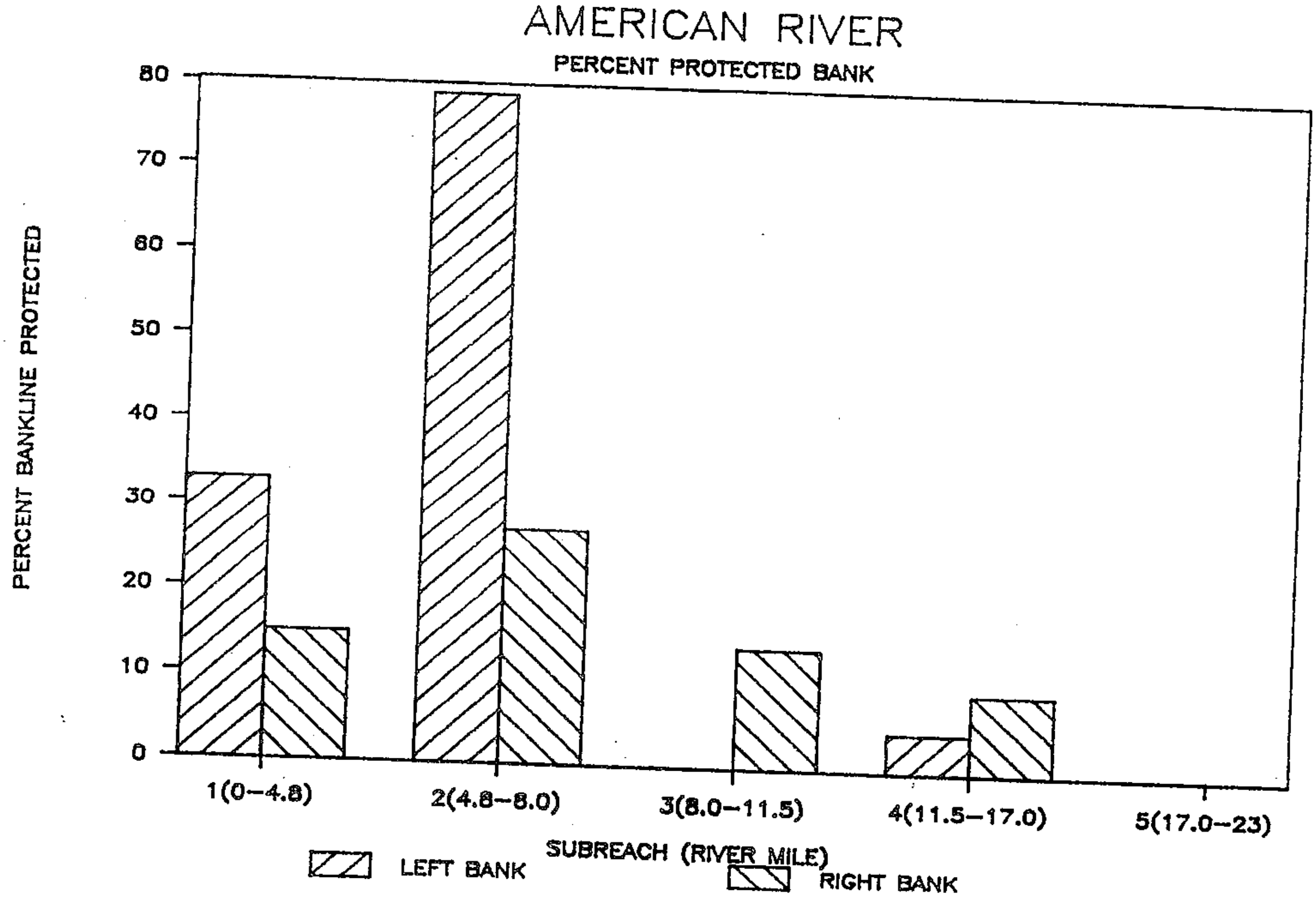


Figure 2.28. Percent protected bankline within study subreaches, Lower American River (WET, 1991).

Table 2.6. Locations of Mapped Damaged Bank Protection, Lower American River (WET 1991).

River Mile	Bank	Length (ft)	Date	Comment
0.45	TLB	200	1988	Localized damage from river access
1.35	TRB	100		Int. failure of concrete rubble
4.80	TLB	100	1969	Localized failure of river cobble and riprap
7.00	TLB	500		Int. toe failure of river cobble and riprap
8.70	TRB	400		Int. failure of non-COE dumped rock
13.80	TRB	200		Private concrete bank; cracking at toe

Rock revetment has performed satisfactorily on the American River and should provide effective bank protection where further bank retreat is unacceptable. Less stringent methods can be applied where some additional retreat is acceptable. Through a large portion of the study reach, project levees are absent, so levee threat is absent. Where levees are present, they are generally setback from the channel. Consequently, long segments of bank line requiring highly dependable forms of bank revetment such as rock riprap have not been identified on the American River. The primary erosion control problem on the Lower American River is not bank line erosion into the levee section, but potential levee failure related to seepage.

3. MAIN DAM ELEMENT

3.1. Hydrologic Analysis

The characteristics of sediment accumulation in the American River channel upstream of the Dry Dam and the potential impact of sediment on the Dry Dam sluices were evaluated by performing sediment routings using the HEC-6 computer model. The sediment routings focused primarily on the 200-year return period design event. The antecedent flow record was used as a warmup flow period to allow the model to stabilize prior to simulation of the design event. The succedent flow record was used to evaluate the potential for reworking of the sediment deposits during normal flows after the design event has passed.

Inflow, outflow, and pool elevation hydrographs for the design event (200-year) were provided by the COE (**Figure 3.1**). The inflow hydrograph represents the combined total inflow to the pool area from both the Middle and North Forks. The outflow and pool elevation hydrographs were determined by the COE by routing the inflow hydrograph through the pool assuming 12 sluices and closure of the existing tunnel. The routings were performed using the DWOPER model. The peak inflow of 295,805 cfs occurs 54 hr into the 408 hr storm hydrograph. The corresponding peak outflow of 86,103 cfs occurs 76 hr into the storm hydrograph. Figure 3.1 shows that the major peak flow is reduced by 71 percent while a secondary peak of approximately 49,000 cfs, occurring at 340 hr is reduced by only 17 percent.

To perform the hydraulic analysis and sediment transport routings for the study reach upstream of the proposed dry dam, it was necessary to separate the COE-provided total inflow hydrograph into two parts, which represent inflows from the Middle and North Forks. This was accomplished using the following relationship provided by the COE:

$$Q_{NF} = 0.6091 * Q_{MF} - 170 \quad (Q_{MF} >= 6000\ cfs) \tag{3.1}$$

$$_{NF} = 0. \quad 1 * \ _{MF} \quad (\ _{MF} < \quad f \ ) \tag{3.2}$$

where Q_{NF} is the discharge (cfs) in the North Fork and Q_{MF} is the discharge (cfs) in the Middle Fork. The resulting hydrographs indicate that between 62 and 63 percent of the total inflow is derived from the Middle Fork basin.

The antecedent flow record consisted of the recorded flows for water years 1984 and 1985. The succedent flows were based on recorded flows for water years 1983 and 1985. These periods were selected to evaluate conditions during a relatively dry year and a relatively wet year with peak flows sufficiently low to minimize impacts associated with backwater at the outlet sluices (i.e., redistribution during run-of-the-river conditions). **Table 3.1** shows characteristics of the selected years and average annual values. By combining either the 1983 or 1984 flow records with the 1985 flows, the modeled antecedent and succedent periods are between 20 and 40 percent higher than mean annual conditions. Plots of the 1983, 1984, and 1985 flows at the Forest Hill Gage on the Middle Fork American River are shown in **Figures 3.2 through 3.4**. Equations 3.1 and 3.2 were used to develop flow hydrographs for the North Fork.

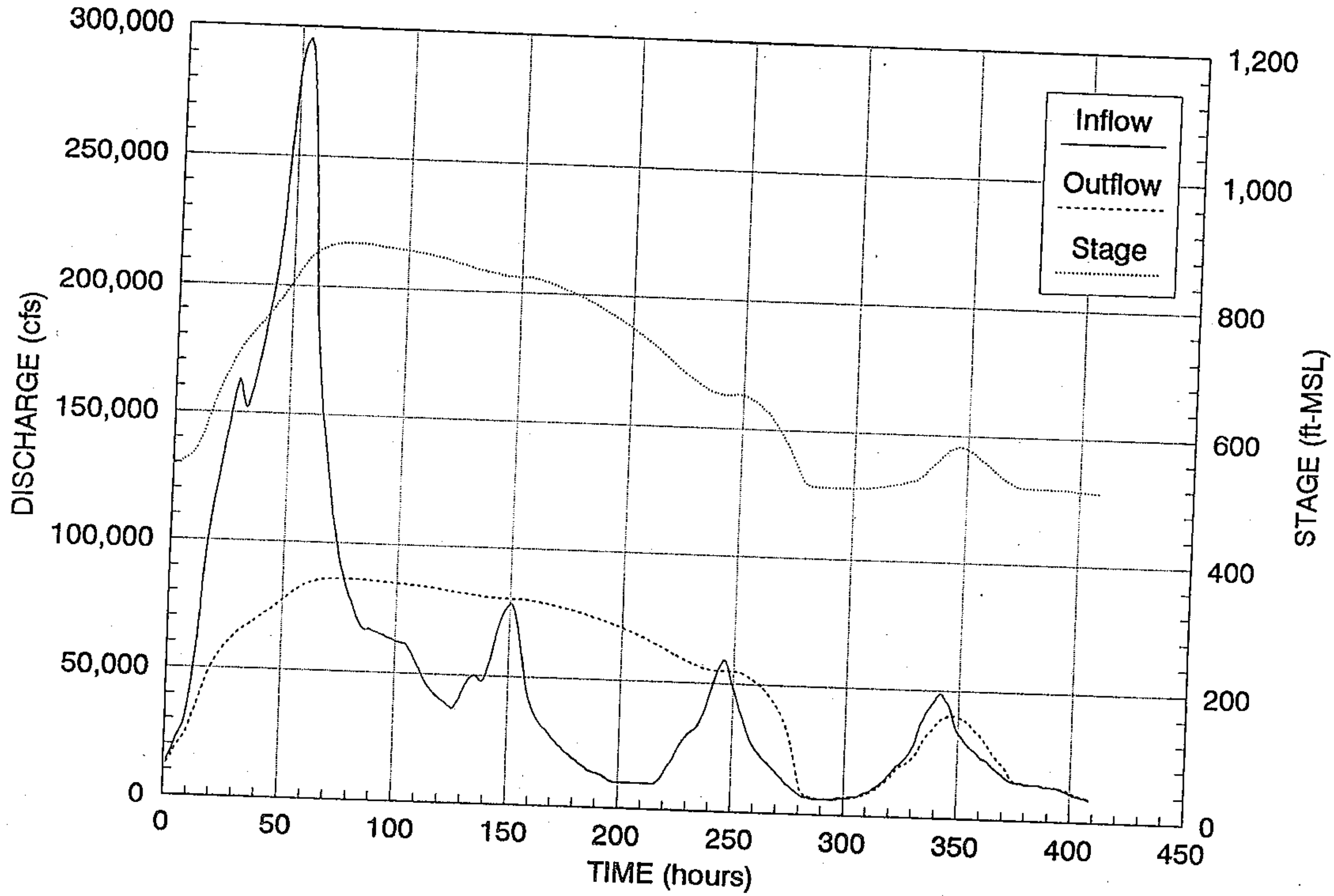


Figure 3.1. Total inflow, outflow, and stage hydrographs for the 200-year recurrance interval design event.

Table 3.1. Flow Characteristics at the Forest Hill Gage, Middle Fork American River, for Period used to Develop Antecedent and Succedent Flow Record.

Year	Runoff Volume (ac-ft)	Peak (cfs)
Average Annual	812,177	----
1983	1,797,899	18,600
1984	1,397,044	18,400
1985	524,570	2,770

Pool elevations for antecedent and succedent flow conditions were estimated based on the outlet rating curve and the inflow discharges. The rating curve is shown in **Figure 3.5**. Figure 3.1 shows that inflows less than about 50,000 cfs experience minimal attenuation. Routing effects were therefore neglected in the antecedent and succedent flow periods. To determine whether significant tributary inflow occurs on the Middle Fork American River between the upstream study limit (Forest Hill Gage) and the confluence with the North Fork American River, comparisons were made between mean daily flow records at the Middle Fork Gages near Forest Hill and Auburn.

Figure 3.6 shows the flow duration curves for the two gages for water years 1959 through 1985 (the common period of record). Between the 30 and 70 percent exceedence levels, the two gage records are virtually indistinguishable. For higher and lower exceedence levels, the Auburn gage recorded higher flows, though generally within 10 percent of the upstream gage. The average discharge for this period of record (1959 and 1985) are 1,190 and 1,260 cfs for the Forest Hill and Auburn Gages. Therefore, on average the flow increased by 6 percent in the 20.4 river miles separating the two gages. This minor amount of additional runoff is not considered significant. For the sediment transport analyses, flow rates were held constant over the Middle Fork study reach.

The Auburn Dry Dam, under the scenario considered in this study, will be operated as an ungated structure (storing water when the inflows exceed the outlet capacity of the sluices and returning to run-of-the-river conditions when the inflows fall below the outlet capacity). The extent of the pool upstream of the dam will therefore vary with time during passage of the hydrograph (reaching its maximum when the discharge in the falling limb of the inflow hydrograph recedes to the capacity of the outlet works). Because the deposition and movement of sediment through the pool area is an important aspect of this study, it was necessary to consider the storage routing effects associated with the variable pool level to establish the discharge profile between the dam and the upstream end of the pool. The equation used to estimate discharge at any point within the pooled reach is:

$$Q_i = (V_i/V_{tot}) * (Q_{in} - Q_{out}) + Q_{out} \tag{3.3}$$

where Q_i is the discharge at the desired location cross section i, V_i is the reservoir volume downstream of that location, V_{tot} is the total volume in the pool, Q_{in} is the reservoir inflow, and Q_{out} is the reservoir outflow. The storage versus location relationships were determined by running the HEC-2 model for the range of pool levels and extracting the incremental storage volumes from the output. Because the proportion of the downstream volume to the total volume will change with stage, these values were determined at vertical intervals of 20 ft, and Equation 3.3 was applied for the appropriate stage increment.

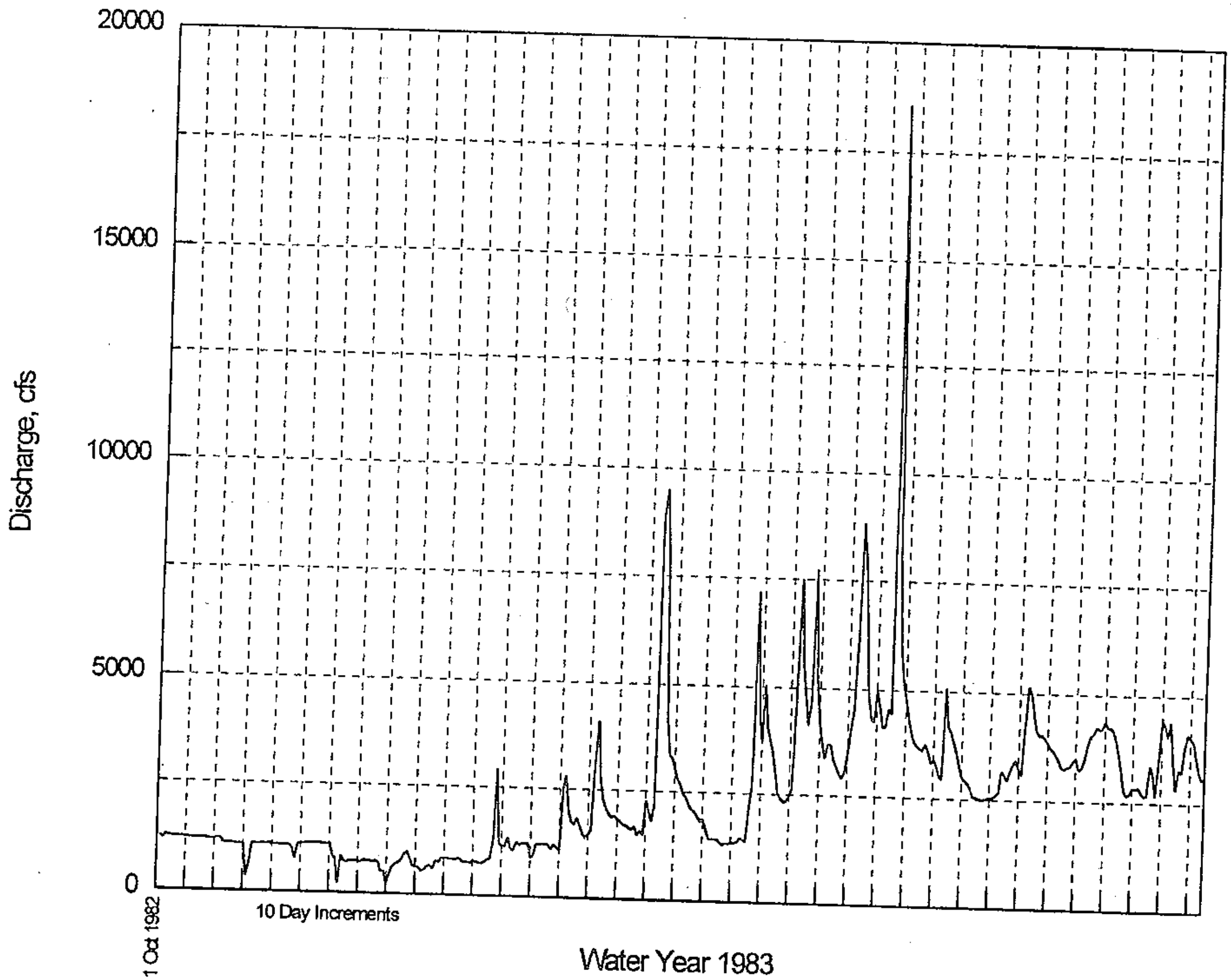


Figure 3.2. Mean daily discharges at Forest Hill gage, water year 1983.

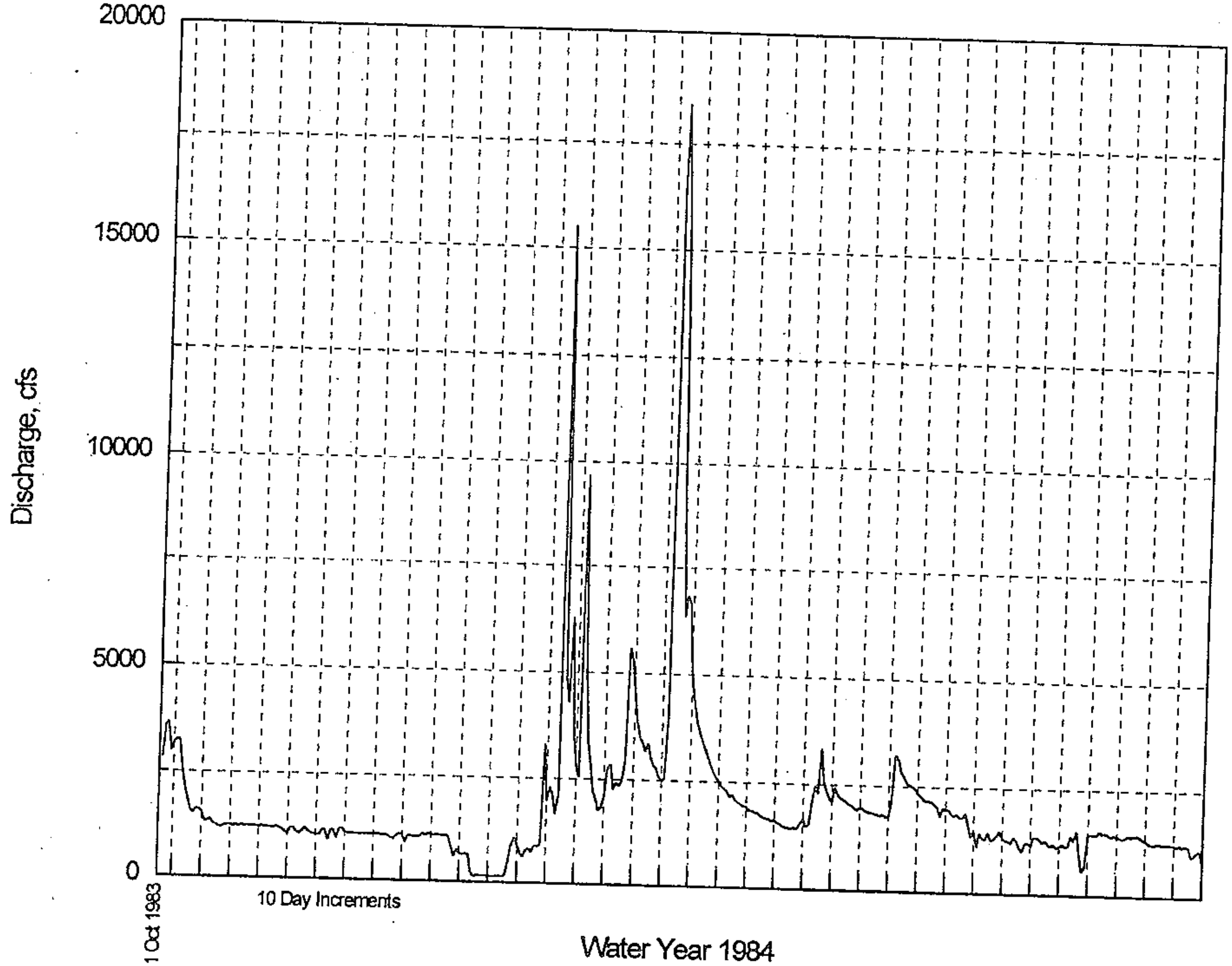


Figure 3.3. Mean daily discharge, water year 1984.

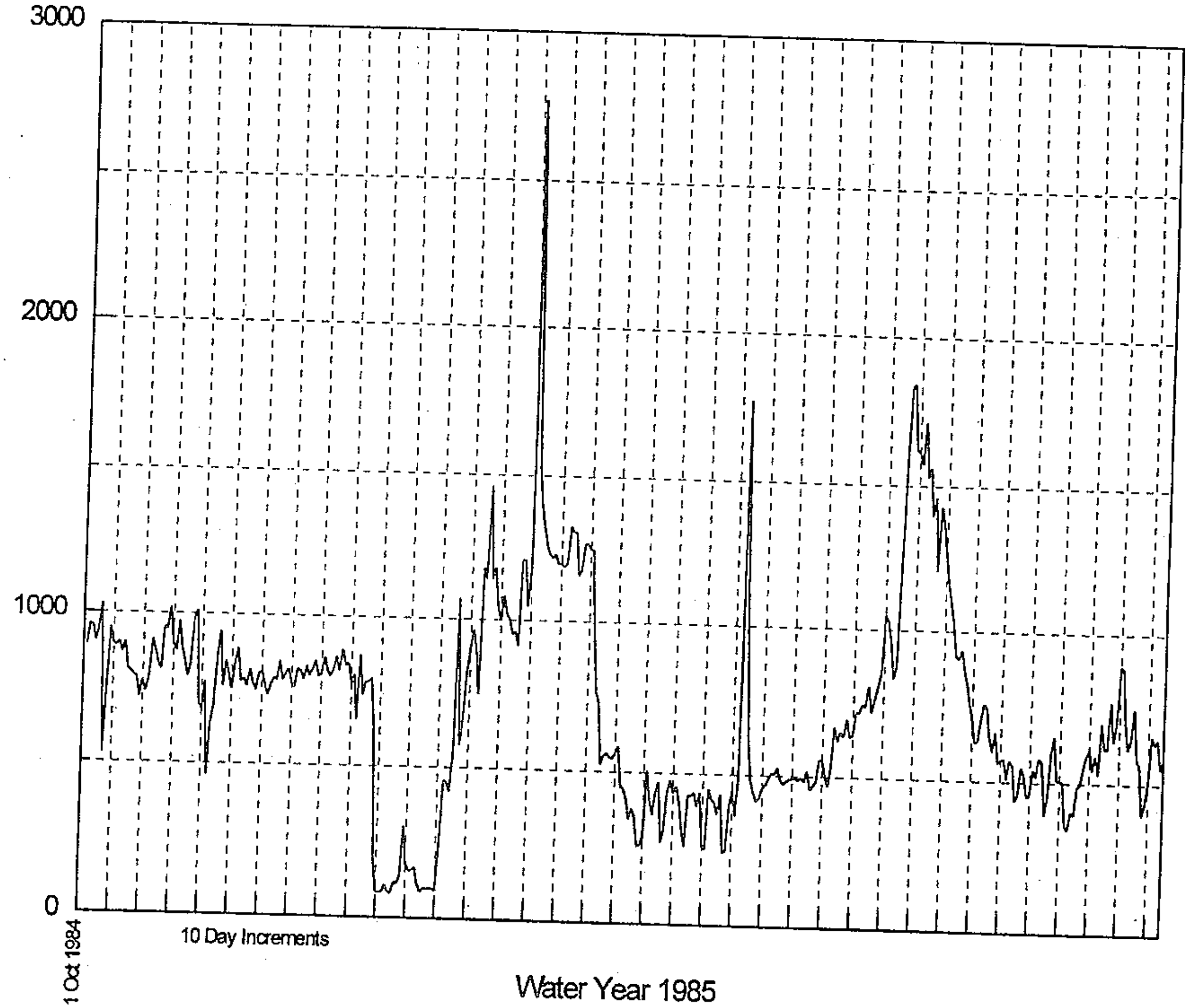


Figure 3.4. Mean daily discharge, water year 1985.

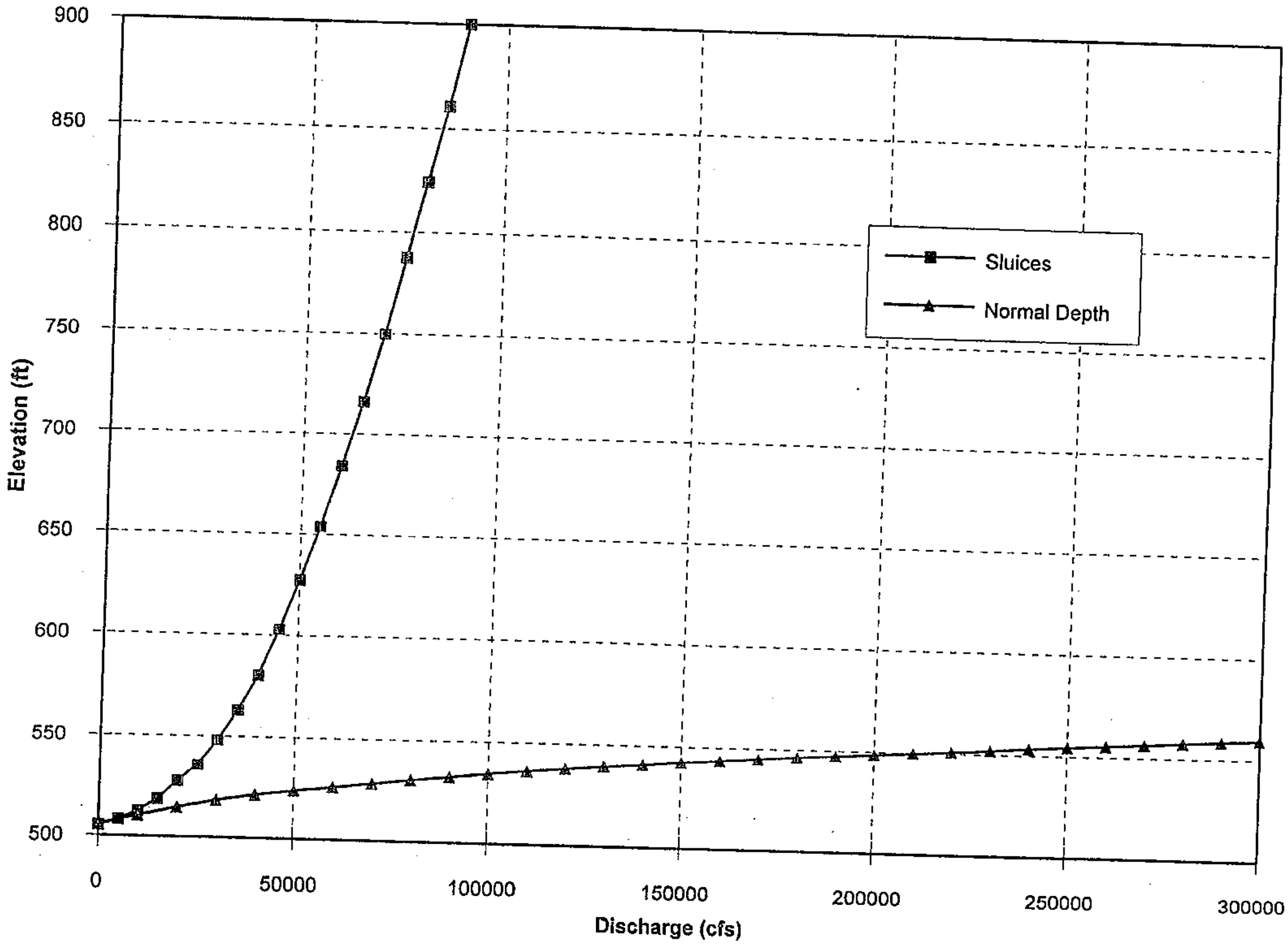


Figure 3.5. Rating curves for the proposed dry dam outlet and normal depth rating curve used for base condition simulations.

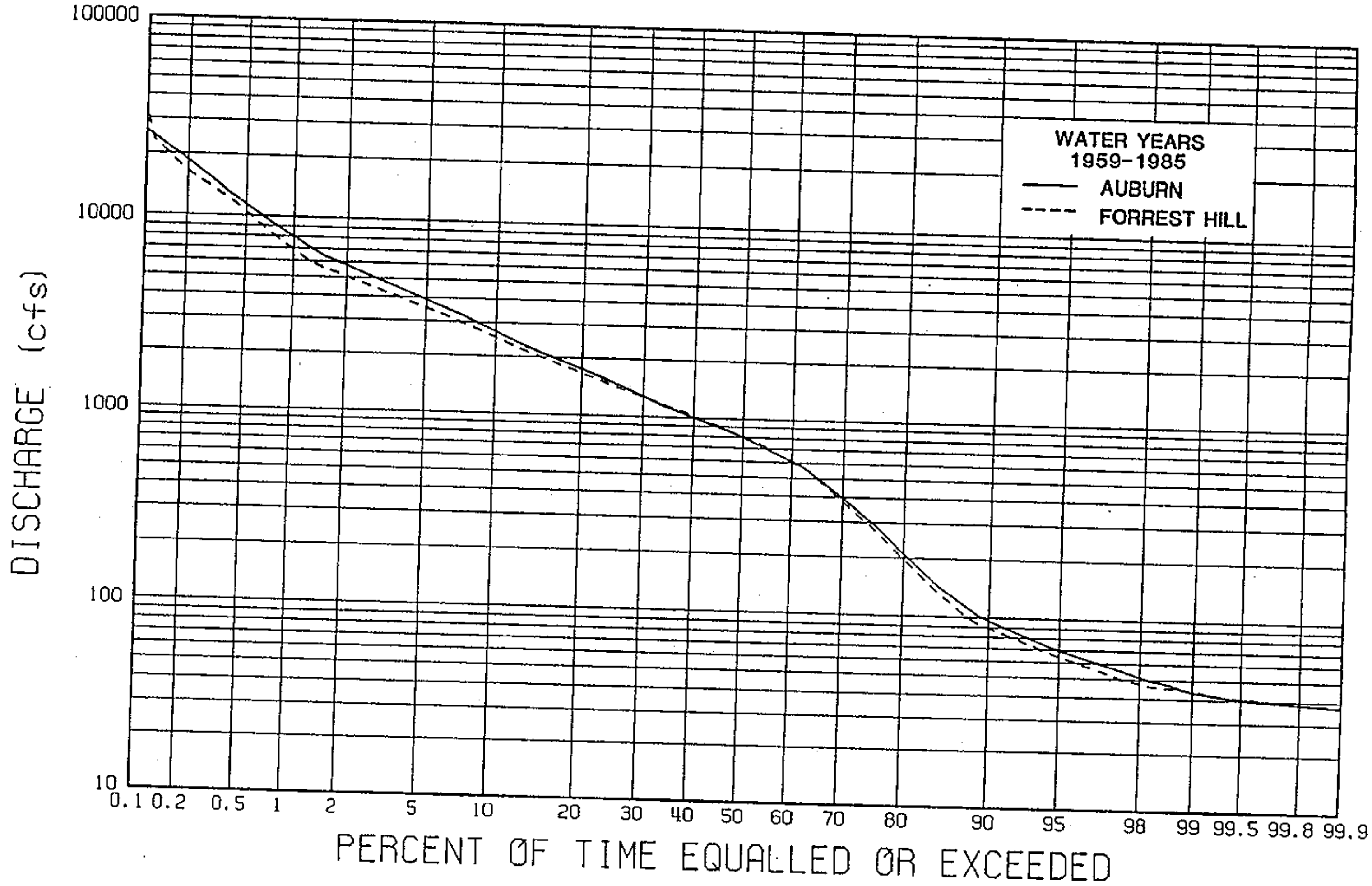


Figure 3.6. Flow duration curves, Middle Fork of the American River.

3.2. Hydraulic Analysis

3.2.1. Main Dam Element Study Reach

A preliminary analysis of hydraulic conditions in the study reach was performed using the U.S. Army Corps of Engineers HEC-2 program (COE 1990) to aid in developing input for the HEC-6 sediment routing model. A schematic of the study reach showing basic reach definitions used in the evaluation and significant features is presented in **Figure 3.7**. The study reach on the North Fork of the American River extends from about RM 20 (approximate location of the dam) to RM 25.3 (North Fork Dam). The study reach on the Middle Fork of the American River extends from RM 0 (confluence with the North Fork) to RM 22.6. Cross sections for the model were developed using 1:24,000 scale USGS quad maps, topography based on USBR surveys, cross-sectional shape determined from detailed surveys at Mammoth Bar, and observations during the field reconnaissance. Average cross section spacing was approximately 1/2 mi, with closer spacing in areas requiring greater detail. Because the contour intervals on the USGS quad sheets were 20 to 40 ft, the valley topography and channel location only were obtained from this source. The cross sections were augmented by adding a 5 ft deep trapezoidal channel below the waters edge depicted on the quad sheets. The width of the channel at each location was based on the quad map and was generally about around 100 ft with the side slopes extended from above water topography. To prevent unrealistically shallow flow depths at low flow, a 30 to 35 ft wide by 2 to 3 ft deep low flow trapezoidal channel was added to the below water channel. This generalized channel shape is consistent with observations made during the field reconnaissance. **Figure 3.8** shows a conceptual illustration of the assumed channel geometry used in the model. Cross sections in the Mammoth Bar reach were, of course, based on the field surveyed data. A discussion of the detailed analyses of the hydraulic conditions at Mammoth Bar is presented in the next section.

Manning's n values ranging from 0.035 to 0.045 were used for the model covering the overall study reach for the Main Dam Element. These values were estimated based on field observations since no data are available with which to calibrate the model except in the vicinity of Mammoth Bar. The preliminary HEC-2 model was run for discharges ranging from 6,000 cfs through 300,000 cfs. The results indicate significant variation in main channel velocity, because primarily the step-pool and riffle-pool structure of the study reach. Velocities for the higher flows were generally in the range of 20 to 25 fps with local areas reaching over 30 fps and depths were generally on the order of about 40 ft. At the lower range of flows between 6,000 and 15,000 cfs, velocities ranged between 5 and 10 fps and depths were in the 5 to 10 ft range. Plots of the bed profile and water surface profiles for run-of-the-river conditions at the dam site for discharges of 6,000, 100,000 and 300,000 cfs are presented in **Figure 3.9**.

3.2.2. Mammoth Bar Study Reach

Based on the geomorphic characteristics of the study reach, it is apparent that the constriction and vertical control created by Murderer's Gorge at RM 52.4 will significantly influence the amount of bed material delivered to reaches between that location and the dry dam site. This constriction causes significant backwater during high flows, which induces sediment deposition in the upstream channel. The sediment deposits associated with this process are referred to as Mammoth Bar. Because of the apparent importance of this site to the sedimentation processes near the dam site, a detailed model of the

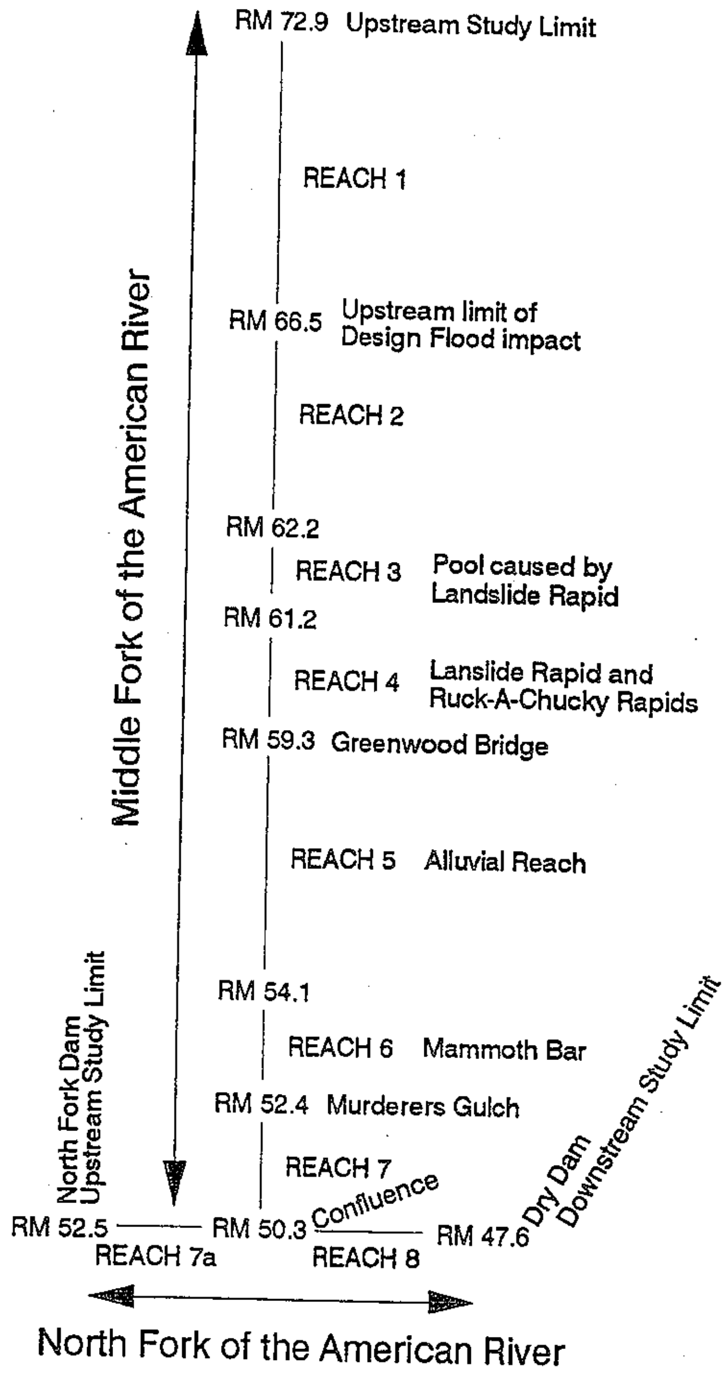


Figure 3.7. Schematic of study reach for main dam element.

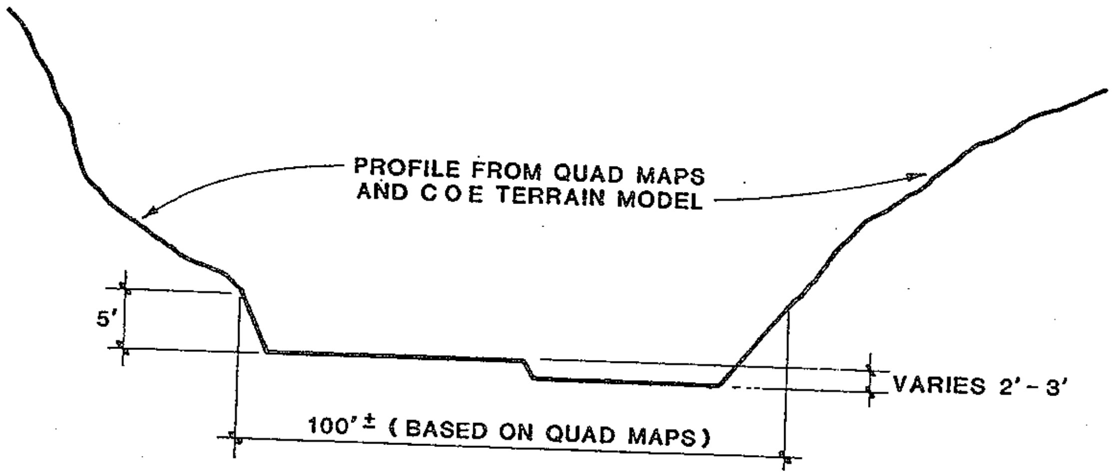


Figure 3.8. Typical cross section in study reach upstream of Auburn dry dam site.

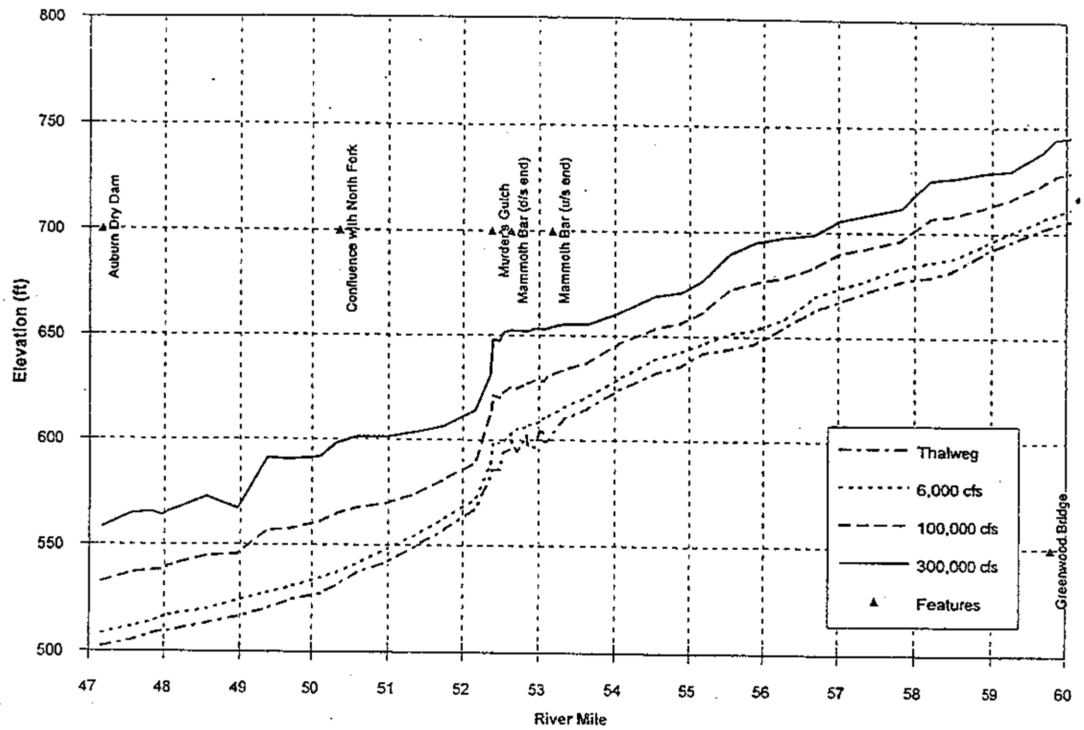


Figure 3.9a. Thalweg and computed water surface profiles for run-of-the-river conditions for three discharges along the main dam element study reach, Dry Dam (RM 47.2) to Greenwood Bridge (RM 59.8)

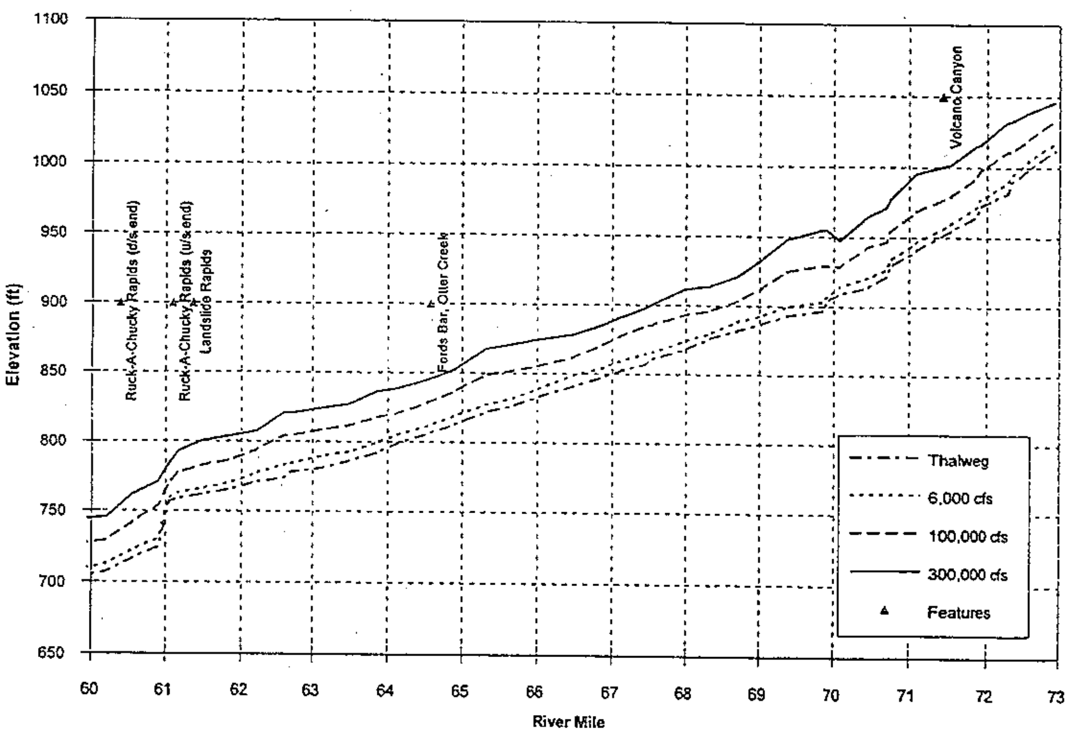


Figure 3.9b. Thalweg and computed water surface profiles for run-of-the-river conditions for three discharges along the main dam element study reach, Greenwood Bridge (RM 59.8) to upstream end of study reach (~RM 73).

Mammoth Bar reach was developed to facilitate a more detailed evaluation of the hydraulic and sediment transport conditions in this area than would be possible using the less detailed model for the overall study reach. A topographic map of the Mammoth Bar study reach showing the location of surveyed cross sections and other significant features is presented in **Figure 3.10**.

A detailed HEC-2 model of the Mammoth Bar reach was developed from field surveyed data collected by the study team in May 1993. The model was calibrated to observed low flow conditions at the time of the field survey (approximately 1,500 cfs), high-water marks from the 1993 peak flow (approximately 19,000 cfs on the Middle Fork), and historic high-water marks, which are believed to correspond to flows during the 1986 flood. Calibration of the model was performed by adjusting the channel roughness values and expansion/contraction losses within physically reasonable limits to obtain a reasonable fit to the observed high-water marks. In addition, it was necessary to estimate some additional cross sections in areas where field survey data was not available. Most of these cross sections were in the contraction created by Murder's Gorge.

Field surveyed topography was available for only the above water cross section at the upstream end of the contraction. Due to the severity of the hydraulic conditions in the gorge at the time of the survey, it was not possible to obtain data for the below water cross section and other midchannel features within the Gorge. During the initial attempts to calibrate the model, it was determined that the hydraulic control shifts from the entrance to the gorge at low flows to a point approximately 150 ft downstream at high flows. The reason for the shift relates to submergence of the smaller boulder at the entrance to the gorge and increased losses at the larger boulder in the middle of the channel at the downstream location. Manning's n values used in the calibrated model were 0.04 for the main channel and 0.05 for the relatively unvegetated overbank areas along the bar.

Figure 3.11 shows the water surface profiles from the calibrated model for 1,500 cfs, 19,000 cfs, and 46,000 cfs along with the observed water surface elevation and high-water marks from the field survey. The three computed profiles reproduce the observed high-water marks relatively well. It should be noted that the peak and two-day average discharges at the Forest Hill Gage (approximately 15 miles upstream of Mammoth Bar) during the 1986 flood were approximately 78,400 cfs and 46,000 cfs. Because the canyon is confined and the channel gradient relatively steep between the two locations, the flood probably attenuated little in that reach. Thus these discharges are believed to be representative of the flows at Mammoth Bar, as well. The high water marks corresponding to the 46,000 cfs flow in Figure 3.11 represent a line along the left side of the channel where the colluvium has been scoured from the underlying bedrock. Because a reasonable duration of flow would be required to accomplish the scouring, it is reasonable to assume that these high water marks would correspond better to a more sustained flow rather than the instantaneous peak. Since the model calibrated well with the lower flows and the profile for the 46,000 cfs discharge is reasonably similar to the high-water mark profile, the calibration is believed to be reasonable.

The calibrated model was used to predict the hydraulic conditions within the Mammoth Bar reach for discharges ranging from 1,000 cfs to 150,000 cfs. The results were then used to predict the variation in bed shear stress and the ability of the flows to mobilize the bed material within the reach. These computation indicate that relatively steep energy gradients occur in the Mammoth Bar reach during low flows when the effect of the constriction at Murderer's Gorge is small which results in relatively high bed shear and thus the ability to mobilize the full range of sediment sizes present in the bed. At higher flows, the effect of the constriction becomes more pronounced that creates backwater conditions, which reduce the bed shear, reduce the ability of the flows to mobilize the bed material, and induce deposition within the reach. This effect is clearly shown in the sediment routing results presented in the following section.

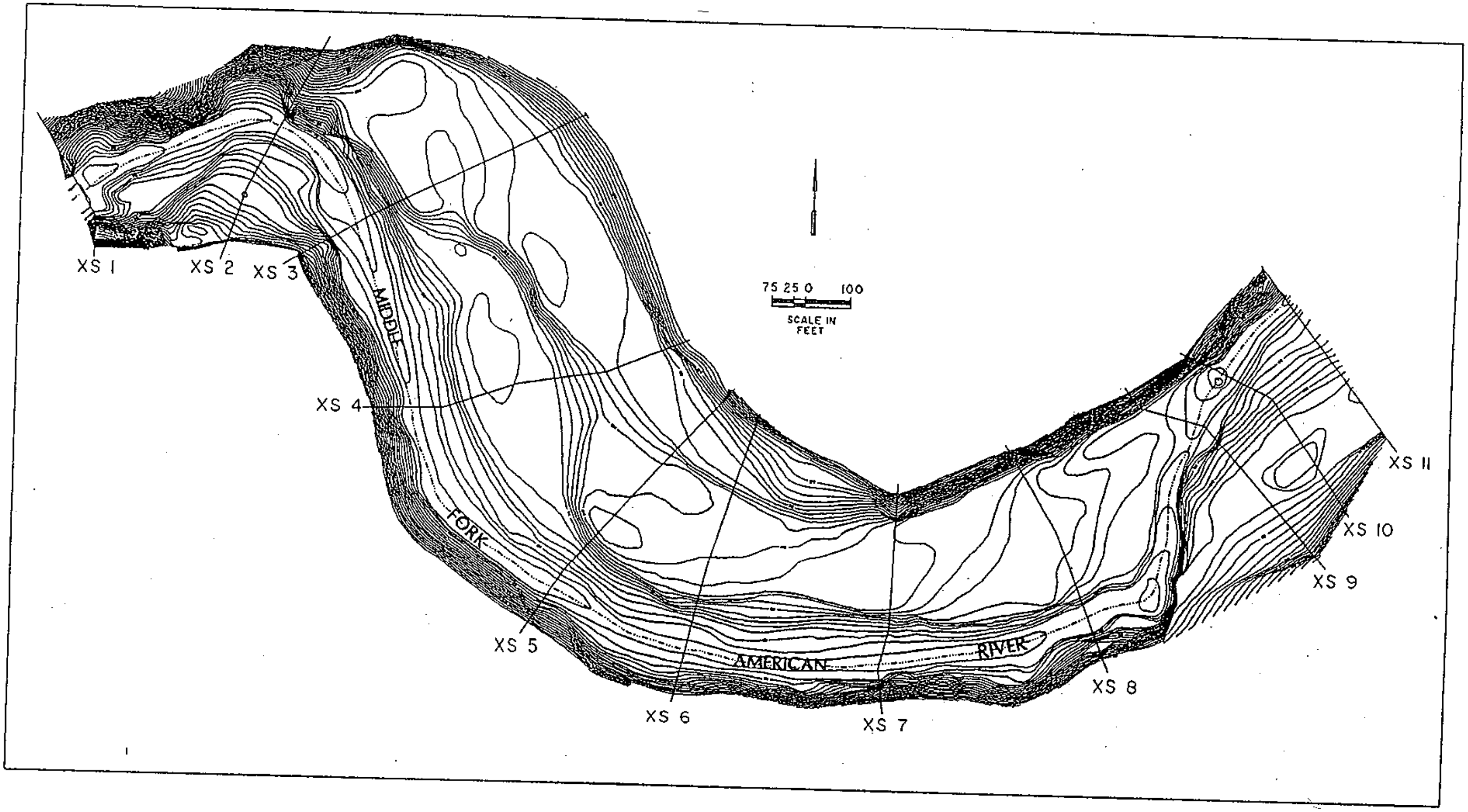


Figure 3.10. Map of the Mammoth Bar Study Reach.

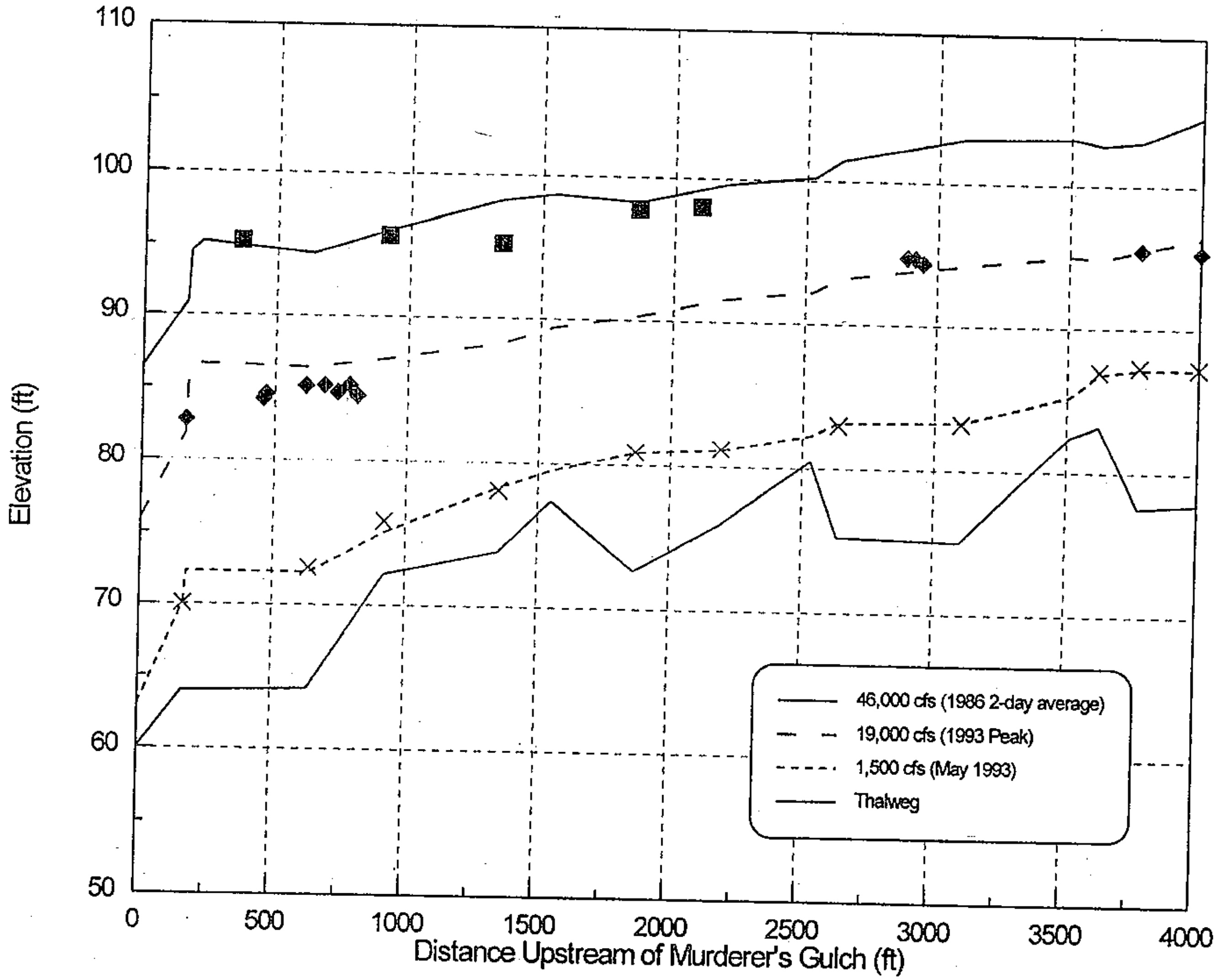


Figure 3.11. Thalweg and calibrated water surface profiles and high water marks for the Mammoth Bar study reach.

3.3. Watershed Sediment Yield

The stability of the study reach (both laterally and vertically) depends upon the supply of sediment from upstream, the ability of the channel to transport the incoming sediment load, and the erodibility of the material comprising the channel boundary. The first step in performing a channel stability analysis is the determination of the upstream sediment yield. An investigation was conducted to identify appropriate analytical and/or empirical methods for estimating the upstream sediment yield on an average annual and single flood event basis. "Sedimentation Investigations of Rivers and Reservoirs" (EM 1110-2-4000; COE 1989) and "Sedimentation Engineering" (ASCE 1975) were used as resource documents for guidance on appropriate methods.

3.3.1. Sediment Yield Methods

In evaluating possible methods for use in the investigation, it is important to consider the modes of transport for sediment delivered from upstream sources since the appropriate computational method(s) are dependent on the delivery mechanism. The sediment load in the river can be separated into two components: wash load and bed material load. Wash load is that portion of the total sediment load made up of particle sizes finer than those found in significant quantities in the bed. The quantity of wash load in the river is determined by the availability of material from bank erosion and upstream watershed sources. Since it is controlled by the availability of material, wash load is generally not carried at the capacity of the stream. For this reason, estimates of the wash load component of the total sediment load must rely on measured data or extrapolation from similar watersheds. It cannot be computed based on the hydraulic conditions in the river.

Bed material load, on the other hand, is that part of the total sediment load that is made up of sediment sizes represented in the bed and is carried both in suspension and as bed load. Bed material load is usually carried at approximately the capacity of the flow, and thus the quantity being carried is a function of the flow energy and caliber of the sediment. The bed material sediment yield can best be estimated by computations of the transport capacity of the river. It is important to note that transport of bed material is normally the most significant factor in determining the stability of the river channel.

Considering the above discussion, the following methods for estimating the total sediment load were investigated: (1) review of the Soil Conservation Service (SCS) sediment yield maps for the Western United States (SCS 1974), (2) use of the Pacific Southwest Interagency Committee rating procedure for evaluating factors affecting sediment yield (PSIAC 1968), (3) review and compile available reservoir sedimentation data for reservoirs in this or similar drainage basins (COE 1990), and (4) develop sediment load rating curves using measured sediment load data, and (5) compute sediment rating curves using available sediment transport relationships.

Methods 1 through 3 provide estimates of the mean annual total sediment yield from the watershed on a per-unit-area basis. Values derived from the SCS sediment yield maps are general in nature and thus provide only order-of-magnitude estimates of the total sediment yield. The PSIAC method is also general in nature (but considers in a qualitative sense) the influence of the important factors affecting sediment yield from a specific watershed. The results from this method should thus be more accurate than those obtained from the more general sediment yield maps. To the extent that reservoir sediment data are available from the watershed (or nearby, similar watersheds), this method can provide excellent information on historical total sediment yields.

When sufficient data are available, method 4 provides a basis for estimating the total measured sediment load on either a mean annual or per storm event basis. Although a reasonably sized set of measured suspended load data are available for the gage on the Middle Fork of the American River at Auburn, this method is only appropriate for estimating the wash load component of the total sediment load. The sampling technique used to collect these data captures little or no the sediment being transport as bed material load.

Method 5 provides the basis for estimating the bed material load only. To compute the total sediment load using this method, independent estimates of the wash load must be made using one of the previous methods. The disadvantage of this method is that an appropriate sediment transport equation must be selected from the dozens available in the literature. These relations can give results that differ by an order of magnitude or more. For this reason, it is critical that the range of conditions for which the selected relations were developed be as similar as possible to the conditions in the present study. In spite of this disadvantage, this is the preferred method for estimating the bed material component of the total sediment load since it directly considers the hydraulic conditions in the stream channel for the specified discharges and the characteristics of the bed material. The Parker bed load equation (Parker, et al. 1982) is the only relation available in the literature that provides satisfactory results for the very coarse material found in the study reach of the American River.

3.3.2. Sediment Yield Results

Sediment Yields based on SCS Maps, PSIAC Method and Reservoir Sedimentation Surveys

Figure 3.12 is a copy of the portion of the SCS sediment yield map of the Western U.S. (SCS 1974) showing the American River watershed. This map indicates that the portion of the watershed within the study reach has an annual sediment yield of 0.1 to 0.2 ac-ft/mi^2 (.34 to .68 tn/ac) and the portions upstream of the study area have an annual sediment yield of less than 0.1 ac-ft/mi^2 (<0.34 tn/ac). Based on the relative amount of the watershed in each classification unit contributing to the study reach, this information indicates that the watershed sediment yield is probably be less than 0.1 ac-ft/mi^2 (0.3 tn/ac).

The PSIAC method was applied to the watershed area upstream of Lake Clementine between North Fork and Ralston Afterbay on the Middle Fork (the approximate upstream study limit) at the Auburn Dam Site. The PSIAC results using the best available information are summarized in **Table 3.2**. The estimated unit sediment yield for these three locations is 0.22 ac-ft/mi^2 (0.75 tn/ac), 0.18 ac-ft/mi^2 (0.61 tn/ac) and 0.32 ac-ft/mi^2 (1.09 tn/ac) for the net contributing watershed area upstream of each location. The total sediment yield at the Auburn Dam site based on the PSIAC method was then estimated by compiling the percentage of the upstream yield passing through the Lake Clementine and Ralston Afterbay and adding the results to the incremental yield between these locations and the Auburn Dam site. The fine sediment (wash load) trap efficiency of Lake Clementine and Raltson Afterbay was estimated to be approximately 70 and 20 percent by using the Brune procedure as described in ASCE (1977).

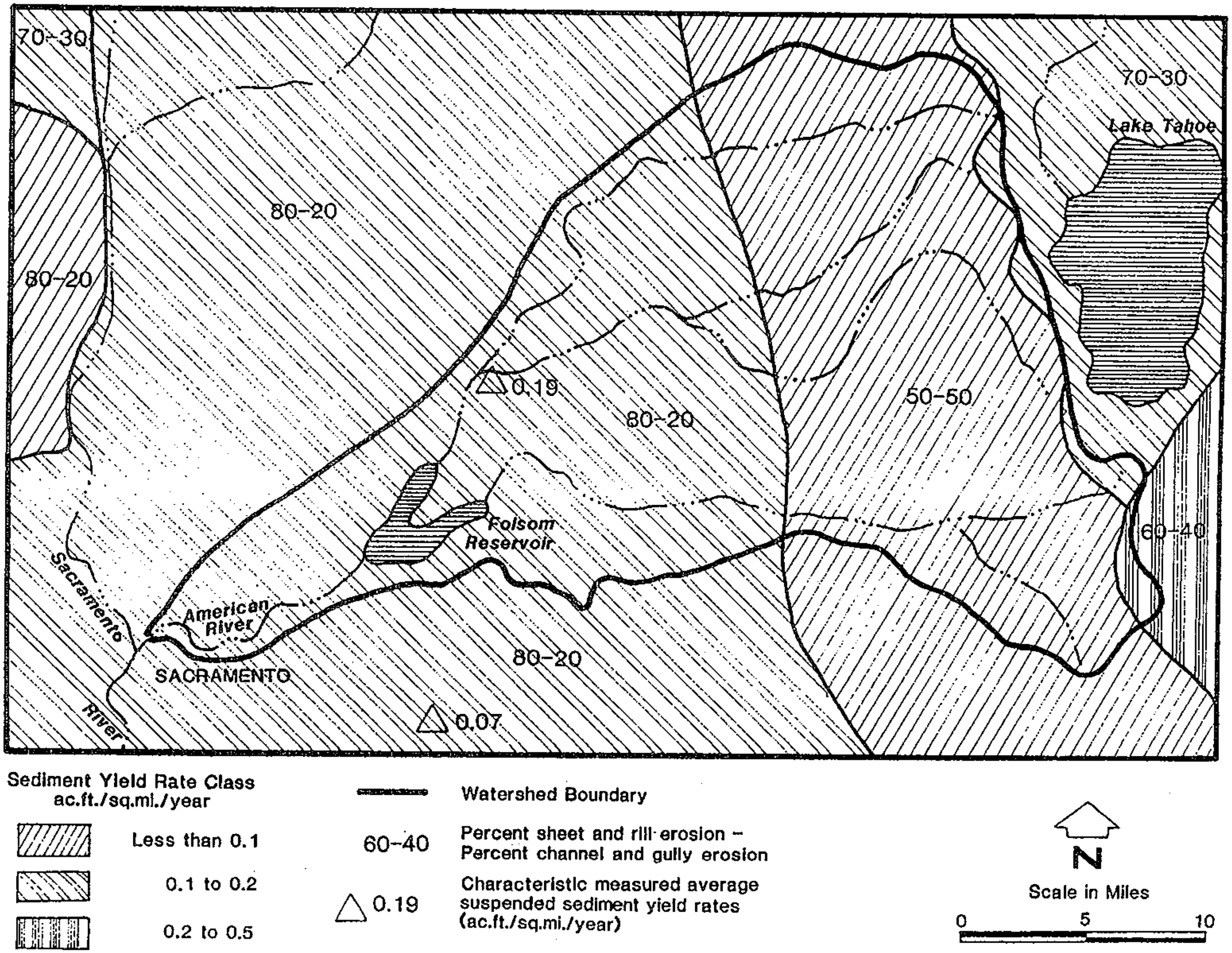


Figure 3.12. Portion of the SCS Sediment Yield Map for the Western United States showing the American River Watershed.

Table 3.2. Summary of PSIAC Watershed Sediment Yield Results for Three Locations in the Main Dam Element Study Reach.

Location	Gross Watershed Area (mi^2)	Contributing Watershed Area (mi^2)	PSICA Watershed Rating	Annual Sediment Yield (ac-ft/mi^2)	Total Annual Sediment Yield (ac-ft)
Middle Fork at Ralston Afterbay	429	255	23	0.18	47.0
North Fork at Lake Clementine	343	333	28	0.22	73.0
North Fork at Auburn Dam site Incremental	198	198	39	0.32	64.0
North Fork at Auburn Dam site Total	970	786	N/A	0.16	123.0

The Churchill procedure was also applied, yielding trap efficiencies of 88 and 65 percent. To provide conservatism in the sediment yield estimates to the dry dam, the lower numbers from the Brune procedure were used. The resulting unit sediment yield for the 796 mi^2 net watershed area upstream of the Auburn Dam site using this procedure is 0.16 ac-ft/mi^2.

The amount of available reservoir sedimentation data is limited. Reservoir sedimentation data compiled by the COE for the Auburn Dam study (COE 1990) indicates annual sediment yields of 0.2 ac-ft/mi^2 at Lake Oroville and 0.28 ac-ft/mi^2 at Bullards Bar Reservoir. These are the closest data points to the study reach. As discussed in Chapter 2, the estimated sediment yield to the North Fork Dam from its construction in 1935 to 1993 is approximately 0.22 ac-ft/mi^2.

Wash Load Sediment Yield Based on Measured Suspended Sediment Data

Suspended sediment data were collected at the Middle Fork American River at Auburn Gage between 1957 and 1967 and during 1979. A total of 101 data points are available from this data set with which to estimate a suspended sediment rating curve. Review of the published gradations for these data indicate that essentially all of the sediment collected in the samples was less than 1 mm in size, thus the data reflects only the wash load component of the total sediment load. These data were plotted as a suspended sediment rating curve, and regression was performed to estimate a best fit curve through the data. (**Figure 3.13**) Relations representing one standard error on either side of the best fit line were also developed to evaluate the effect of scatter in the data on the predicted sediment yields.

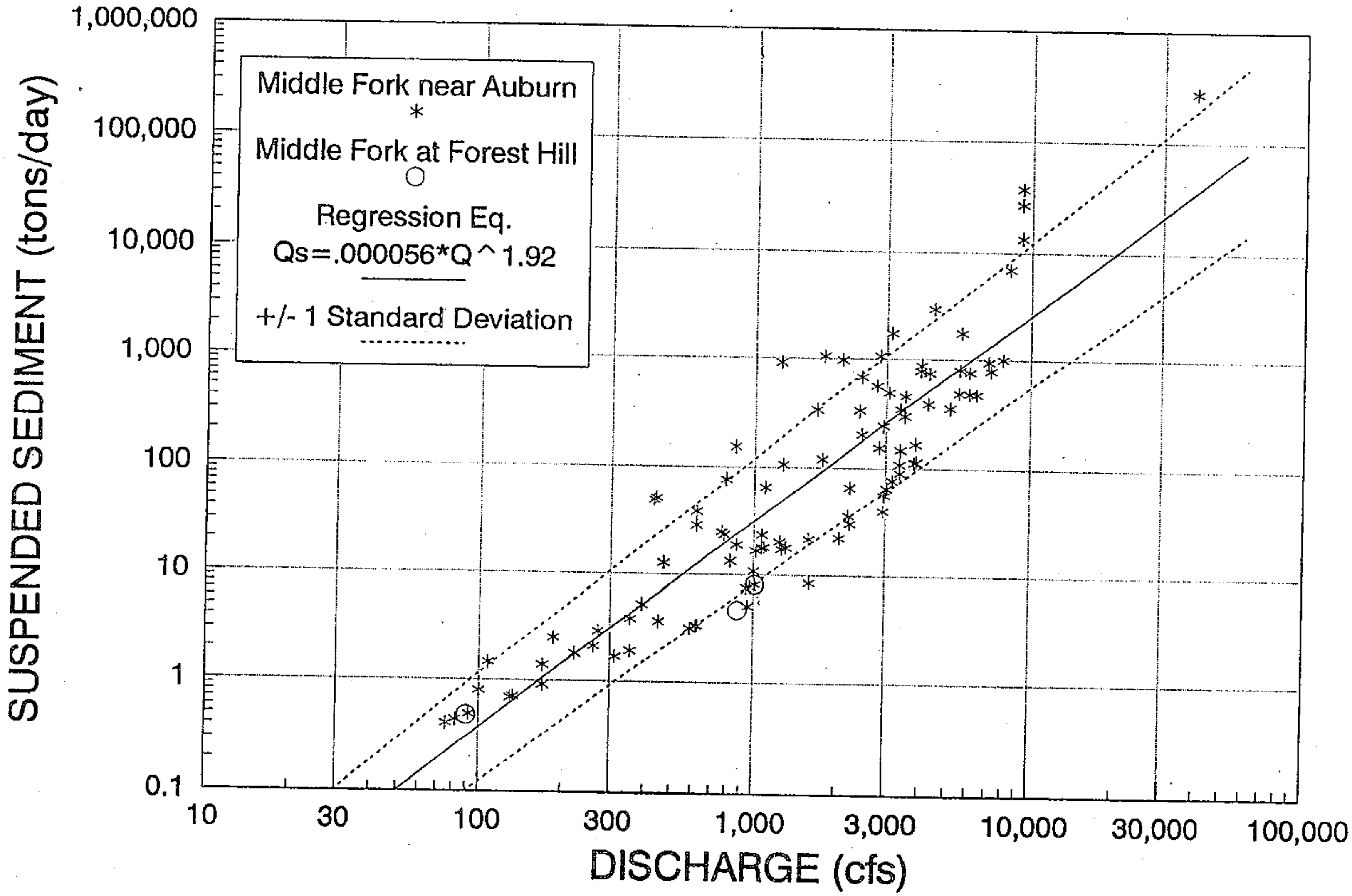


Figure 3.13. Measured suspended sediment data for Middle Fork American River at Auburn and suspended sediment rating curves.

The relations shown in Figure 3.13 were also used to represent the suspended sediment (wash load) yield for the Middle Fork at Forest Hill, North Fork at the North Fork Dam, and at the Auburn Dam site. The average annual wash load supply to the study reach on the Middle Fork was estimated by integrating the flow duration curve for the Forest Hill Gage. The same procedure was used for the North Fork using the flow duration cover from the North Fork Dam Gage. It was further assumed that the North Fork Dam would trap approximately 70 percent of the wash load sized sediment. The annual wash load yield at the Auburn Dam site was estimated as the sum of the yields from the Middle Fork and North Fork. The wash load yields for the design storm were estimated by integrating the design storm hydrographs for the Middle and North Forks and computing the Auburn Dam site yield as before. The results of these computations are summarized in **Table 3.3**. In general, the best estimate results indicate annual wash load yields of less than 0.1 ac-ft/mi^2 (0.3 tn/ac) at all three locations ranging from a lower limit estimate of less than 0.01 ac-ft/mi^2 (0.03 tn/ac) to an upper limit estimate of approximately 0.2 ac-ft/mi^2 (0.7 tn/ac). For the design storm, the wash load yields for the best estimate relations were about 1 ac-ft/mi^2 (3.4 tn/ac), ranging from a lower bound estimate of less than 0.1 ac-ft/mi^2 (0.3 tn/ac) to an upper bound estimate of about 8 ac-ft/mi^2 (27.2 tn/ac).

Table 3.3. Summary of Wash Load Sediment Yields for Three Locations in the Main Dam Element Study.

Location	Contributing Drainage Area (mi^2)	Annual Wash Load Sediment Yield (ac-ft/mi^2)		
		Best Estimate	Upper Limit*	Lower Limit*
Middle Fork at Forest Hill	255	0.43	0.20	0.009
North Fork at Lake Clementine	333	0.047	0.21	0.01
North Fork at Auburn Dam site	786	.043	0.20	0.009
		Design Storm Wash Load Sediment Yield (ac-ft/mi^2)		
Middle Fork at Forest Hill	255	1.64	9.66	0.28
North Fork at Lake Clementine	333	0.48	2.71	0.09
North Fork at Auburn Dam site	796	1.11	7.97	0.19

Total Sediment Yield based on Bed Material Load Computations

The bed material sediment yield was estimated at the upstream end of the Middle Fork study reach, the North Fork at the confluence with the Middle Fork and at the Auburn Dam site for base and dry dam conditions. Estimates were made for the average annual bed material yield at each point (taken as the average of the combined 4 yr of simulation for antecedent and succedent conditions) and the design storm and the accumulated yield was over a 50-year project life. **Table 3.4** summarizes the results of these computations. The best estimate values are based on results of the HEC-6 modeling presented in the next section. Based on the scatter in the bed material rating curves predicted by the model and typical scatter in bed material load data, it was assumed that the upper and lower limits could vary from the best estimate by a factor of two.

Table 3.4. Summary of Bed Material Sediment Yields for Three Locations in the Main Dam Element Study Reach.			
Location	Annual Bed Material Sediment Yield (tons)		
	Best Estimate	Upper Limit*	Lower Limit*
Middle Fork at Forest Hill	14,500	29,000	7,250
North Fork at confluence with Middle Fork (Base Conditions)	1,700	3,400	850
North Fork at confluence with Middle Fork (Dry Dam Conditions)	110	220	55
North Fork at Auburn Dam site (Base Conditions)	16,900	33,800	8,450
North Fork at Auburn Dam site (Dry Dam Conditions)	13,500	27,000	6,750
Design Storm Bed Material Sediment Yield (tons)			
Middle Fork at Forest Hill	560,000	1,120,000	280,000
North Fork at confluence with Middle Fork (Base Conditions)	270	540	135
North Fork at confluence with Middle Fork (Dry Dam Conditions)	40	80	20
North Fork at Auburn Dam site (Base Conditions)	265,100	530,200	132,550
North Fork at Auburn Dam site (Dry Dam Conditions)	70	140	35
Project Life (50-years) Bed Material Sediment Yield (million tons)			
Middle Fork at Forest Hill	0.725	1.450	0.363
North Fork at confluence with Middle Fork (Base Conditions)	0.085	0.170	0.043
North Fork at confluence with Middle Fork (Dry Dam Conditions)	0.006	0.011	0.003
North Fork at Auburn Dam site (Base Conditions)	0.845	1.690	0.423
North Fork at Auburn Dam site (Dry Dam Conditions)	0.675	1.350	0.338

The results shown in Table 3.4 indicate that about 14,500 tn of sand, gravel, and cobble sized sediment will be delivered to the upstream end of the study reach on an average annual basis. For base conditions, the North Fork will deliver an additional 1,700 tn and the main stem would carry approximately 16,900 tn past the Auburn Dam site, which makes the overall study reach slightly degradational (removal of about 700 tn/yr over the approximately 22 mi long study reach) under these conditions. For dry dam conditions, the annual yield from the North Fork is reduced to approximately 110 tn, and the amount passing through the dry dam sluices will be about 13,500 tn (indicating that about 1,00 tn of material will accumulate in the reach on an average annual basis).

During the design storm, the relative sediment balance changes significantly. Approximately 560,000 tn of sediment will be delivered to the upstream end of the Middle Fork study reach. Only about 270 tn would be delivered from the North Fork under base conditions while about 265,100 tn would be carried past the dam site. During the design storm event for base conditions, the overall study reach is aggradational (accumulation of approximately 295,200 tn). For dry dam conditions, the yield from the North Fork reduces to about 40 tn and the amount passing through the sluices is about 70 tn, which increases the amount of aggradation in the reach to approximately 560,000 tn.

The relative sediment balance over the 50-year project life is the same as for average annual conditions, if large storm events are neglected in the analysis. Sufficient information is not available from the current study to quantify the effects of large storms other than the 200-year design storm. Given the tendency for sediment to accumulate upstream of constrictions (e.g., Mammoth Bar), channel blockages (e.g., Landslide Rapid), and the in the pool created by the dry dam, it can be expected that a significant quantity of sediment may accumulate in the study reach during this period. The amount and spatial distribution of the accumulated sediment is dependent on the number and magnitude of large events that occur during the life of the project.

3.4. Sediment Transport Routings

Sediment transport routings were performed to (1) evaluate the amount and size distribution of sediment reaching the sluice gates in the dry dam and (2) the distribution of sediment deposits in the pool area for design event conditions and normal flows that will occur subsequent to the design event. The effects of the reservoir pool on the sediment deposits were evaluated by comparing model results for two runs: the first based on normal depth conditions at the dry dam location and the second based on reservoir pool conditions with the dry dam in place. Normal depth was used as the downstream control for the first run to represent run-of-the-river conditions. This run was treated as a base condition, differences between the two runs. Therefore, it represents the effects of the pool caused by the dry dam.

3.4.1. Model Setup

Model Selection and Input Data

The HEC-6 sediment routing model was selected for use in this study because of its acceptance among the engineering community for such purposes and for consistency with previous sediment transport studies within the Sacramento River system. The library version of HEC-6 (HEC 1991) could not be used for this analysis because it does not accommodate grain sizes greater than 64 mm. It does not include sediment transport functions suitable for estimating the transport capacity of coarse bed material. Grain sizes in the study reach include cobbles and boulders. Field observations of the Mammoth Bar and other reaches indicate that these sizes are transported.

The analysis was performed using an experimental version of HEC-6 supplied by the Corps of Engineers Waterways Experiment Station (CEWES) and modified subsequently by RCE/Ayres (RCE/Ayres 1993). The experimental version of HEC-6 allows the use of particle sizes up to 2,000 mm. A detailed description of modifications made by RCE/Ayres is included in RCE/Ayres (1993). Significant modifications relevant to this study include addition of the Parker bed-load equation (Parker, et.al. 1982) linked with the Toffaleti suspended-load equation to provide a more suitable method of computing bed material transport capacity for large bed material sizes than those available in the original model. Other modifications include the ability to specify different transport functions for different branches of the stream network (a feature not used in this study), creation of a new summary output file, and modifications to port the program from mainframe computer to PC.

The study reach considered in the HEC-6 modeling considered the Middle Fork between the outlet of the tunnel near RM 73 and the confluence with the North Fork and the North Fork from the North Fork Dam and the Auburn Dam site (Figure 3.7). The North Fork Dam and Lake Clementine will be inundated by the dry dam pool during part of the design storm hydrograph. The reach of the North Fork upstream of the North Fork dam was not, however, considered in the modeling because all of the bed material delivered to Lake Clementine during both the design storm and under normal flow conditions will be trapped and, thus will not contribute to bed changes between the dams or the amount of sediment delivered to the dry dam sluices. The North Fork branch of the HEC-6 model was, therefore, truncated at the North Fork Dam and bed material inflow at that point was assumed to be zero.

The geometric portion of the HEC-6 model was developed from the HEC-2 model discussed in the previous section. The detailed Mammoth Bar HEC-2 deck was included in the overall deck. To avoid numerical instability, several of the original Mammoth Bar cross sections were removed to produce a cross section spacing similar to the rest of the model. The HEC-6 model computed super critical flow at the cross sections at the entrance to Murderer's Gorge (the downstream control for hydraulics at Mammoth Bar) whereas HEC-2 used critical depth. The HEC-2 results were deemed more appropriate because critical flow at this location is more physically reasonable and because the HEC-2 results were calibrated to observed high-water marks. For this reason, an internal water surface boundary condition was used at this location to replicate the HEC-2 water surface.

As described in the previous section downstream control (starting water surface elevations) for the base conditions run was estimated based on normal depth at the dam site, with the energy slope assumed equal to the bed slope in the downstream mi of the study reach. The only change in discharge along the study reach in the base conditions model occurs at the confluence of the North and Middle Forks.

Starting water surface elevations for the project conditions run, used to simulate the effects of the dam during antecedent and succedent conditions were based on a stage-discharge rating curve provided by the COE (Figure 3.5) for the proposed sluice configuration. For the design flood, an outflow discharge and reservoir stage hydrographs supplied by the COE (Figure 3.1) were used. Equation 3.3 was used to estimate the variation in discharge along the reach within the pool area during operation of the dry dam.

Bed material size gradations used in the model were based on sediment samples obtained during the field reconnaissance. As discussed in Chapter 2, a total of nine Wolman counts of the surface material were collected in the study reach during the field reconnaissance. Four of the samples were taken at various locations within the Mammoth Bar reach and the other five were taken at key locations between the upstream end of the Middle Fork study reach and the Auburn Dam site. Five subsurface samples were also collected, one at each of the Mammoth Bar Wolman count locations and one at the sample location approximately 1.5 mi upstream of the Auburn Dam site. It was not possible to obtain subsurface samples at the four upstream sites due to the coarseness and imbrication of the channel bed materials in those locations.

Based on the size characteristics of the surface material, the sample taken near the Forest Hill gage was assumed to be representative of the upstream end of the study reach (RM 69.5 to RM 72.9). A representative surface gradation for the reach between Murderer's Gorge (RM 52.3) and RM 69.5 was developed by compositing the four samples between the head of Mammoth bar and RM 69.5. The reach downstream of the confluence was represented by the sample taken at RM 48.8. These gradations are shown in **Figure 3.14**. Based on field observations and to avoid discontinuity in the model created by an abrupt reduction in the sediment size at Murderer's Gorge, a transitional gradation was applied between Murderer's Gorge and the confluence with the North Fork by averaging the latter two gradations.

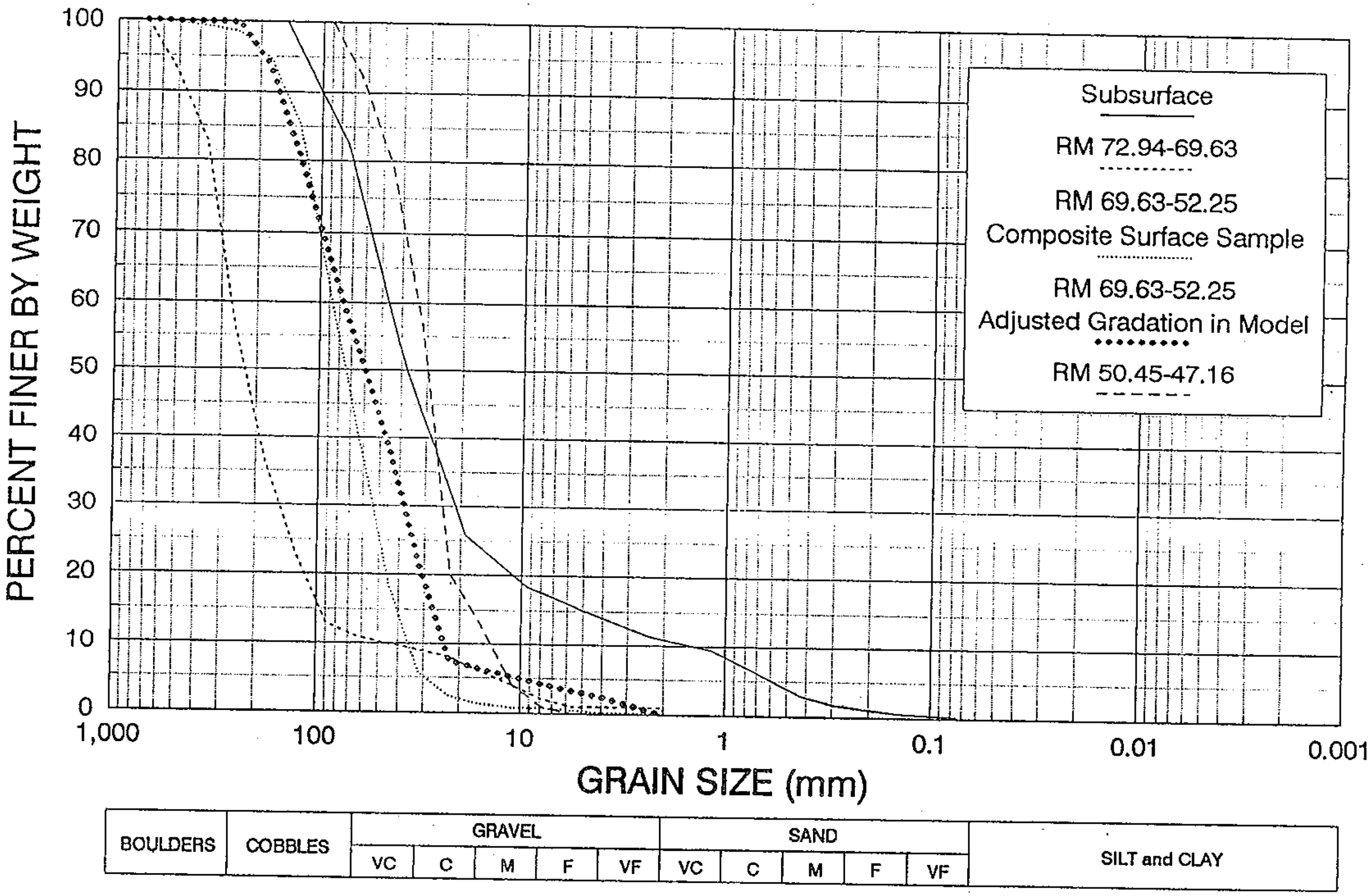


Figure 3.14. Representative bed material size gradations used in the HEC-6 modeling.

Since sand sized material is generally under-represented in the Wolman count data, it was necessary to adjust the measured gradations to supplement the amount of material in the sand size range. This was accomplished by comparing the gradation of the bed material load from the model results during the calibration process with the gradation of the subsurface samples taken during the field reconnaissance. The sand material was increased until the computed gradations were similar to the subsurface material at the higher flows. This adjustment is reasonable because the subsurface material is the parent material from which the bed material load is derived during conditions when the surface pavement is mobile and has been shown to be representative of the gradation of the transported material under those conditions. The resulting adjusted size distribution curve used in the model for the alluvial reach between Murderer's Gorge and the downstream end of Ruck-a-chucky Rapid (RM 59.3) is also shown in Figure 3.14.

Depths of alluvium were specified in the model for each reach based on available data (i.e. USBR trenching) and field observations. Where field observations indicated that little or no alluvium occurred in the channel bed in bedrock or boulder bed reaches, a model sediment depth of 0.5 ft was specified. This was done because of uncertainty regarding the ability of HEC-6 to simulate sediment deposition where zero sediment depth is initially specified in the model input. This would be a major drawback for simulating the impacts of the proposed dam. In addition, some sediment storage occurs along the bank lines for all reaches.

Using the above guidelines, a sediment depth of 0.5 ft was specified between the upstream end of the study reach (RM 72.9) and RM 65.7, between Landslide Rapid (RM 61.2) and the downstream end of Ruck-a-chucky Rapid (RM 59.3), the North Fork from the confluence to the North Fork Dam, and at all riffle sections identified during the field reconnaissance. Sediment depths varying between 1 and 3 ft were used in the backwater area upstream of Landslide Rapid (RM 65.7) to RM 61.2 and 1 and 2 ft between Murderer's Gorge (RM 52.4) and the Dry Dam (RM 47.6). A sediment depth of 10 ft was used in the alluvial reach between Greenwood Bridge (RM 59.3) and Murderer's Gorge (RM 52.4).

As previously discussed, the input step hydrographs were developed for the design (200-year) flood, 2 yr of antecedent flows and 2 yr of succedent flows. Time step lengths were selected to prevent numerical instability in the model using the procedures in "Guidelines for the Calibration and Application of Computer Program HEC-6" (COE 1981) and checked using the general guideline of no more than 1 ft of change in the bed elevation during one time step in the shortest reach between cross sections using the transport capacity at the highest discharge in the simulation. In some cases it was necessary to shorten the time step lengths derived from the above procedure to insure that the final step hydrograph adequately represented the shape of the hydrograph. For the design flood hydrograph, time steps ranged from 1 to 7 hr. For antecedent and succedent flows, time steps ranged from 2 hr for the highest flows to 35 days for low flow conditions.

The SPI value, which represents the number of sorting iterations per time step, was selected by making test runs with different values and selecting the smallest value for which no significant change in transport capacity occurred with increasing SPI. These tests showed that an SPI value of 10 produced the desired results; this value was used in all subsequent model runs.

Inflowing Sediment Load

There are no measured bed material transport data with which to calibrate the bed material sediment inflow for the Middle Fork of the American River. Additionally, with the exception of deposits in the backwater created by Landslide Rapid (RM 61.2), reaches of the channel upstream of Greenwood Bridge (RM 59.8) are characterized primarily by large cobbles, boulders, and bedrock that are not transported in significant quantities by the range of flows considered in this study (Plate 1). These reaches are supply limited in that the transport capacity for the cobbles, gravel, and sand that are transported is significantly greater than the supply. Development of an inflowing sediment load curve based on the characteristics of the upstream reaches of the model was therefore not possible.

From field observations and other available data, the channel bed in the reach between Mammoth Bar (~ RM 52.5) and just downstream of Greenwood Bridge (RM 59.3) is composed of alluvial material in sizes that can be transported under the range of flows being considered in this study. The channel in this reach is characterized by alternate bars, with depths of alluvium of 30 to 50 ft based on trenching performed by the U.S. Bureau of Reclamation (BOR) and appears to be relatively stable, with no obvious evidence of significant aggradation or degradation. Based on these observations, it was assumed that this reach is adjusted to the sediment load delivered from upstream and probably transports sediment at approximately its capacity, which must also represent the inflowing sediment load to the reach. The inflowing sediment load curve for the Middle Fork study reach was, therefore, estimated based on the transport capacity of the alluvial reach. This was accomplished by computing the sediment transport capacity for seven cross sections within the alluvial reach (RM 52.35 to 59.25) for flows ranging from 4,000 to 200,000 cfs and averaging the results to obtain the inflowing sediment load curve. The model was then run for the period of record and the volume of material transport through the alluvial section was compared to the inflowing sediment load. The inflowing sediment load curve was then adjusted and the model re-run until the two values were in reasonable agreement. The final inflowing bed material load rating curve developed using this procedure is presented in **Figure 3.15**. Rating curves for each of the size ranges considered in the simulation are also shown in the figure. This curve indicates bed material sediment loads ranging from zero at approximately 3,700 cfs (critical conditions for disruption of the surface pavement) to approximately 280,000 tn/day at 200,000 cfs.

It should be noted that, with the exception of the backwater area caused by Landslide Rapid, the transport capacity of the reaches between the upstream study limit and the alluvial reach is significantly higher than is indicated by this rating curve. As previously discussed, the sediment depth in these high transport rate regions was set to 0.5 ft based on field observations (Plate 2). The model thus transports essentially all of the sediment supply through these reaches without significant changes in bed elevation.

As previously discussed, the North Fork Dam will trap all of the bed material sized sediment brought in from upstream; thus, the inflowing sediment load curve for the North Fork Branch of the model was assumed to be zero. Additionally, as discussed in the hydrology section (Section 3.1), little additional runoff occurs along the Middle Fork study reach between the upstream study limit and the confluence with the North Fork. Tributary sediment contributions in this reach were therefore neglected in the modeling.

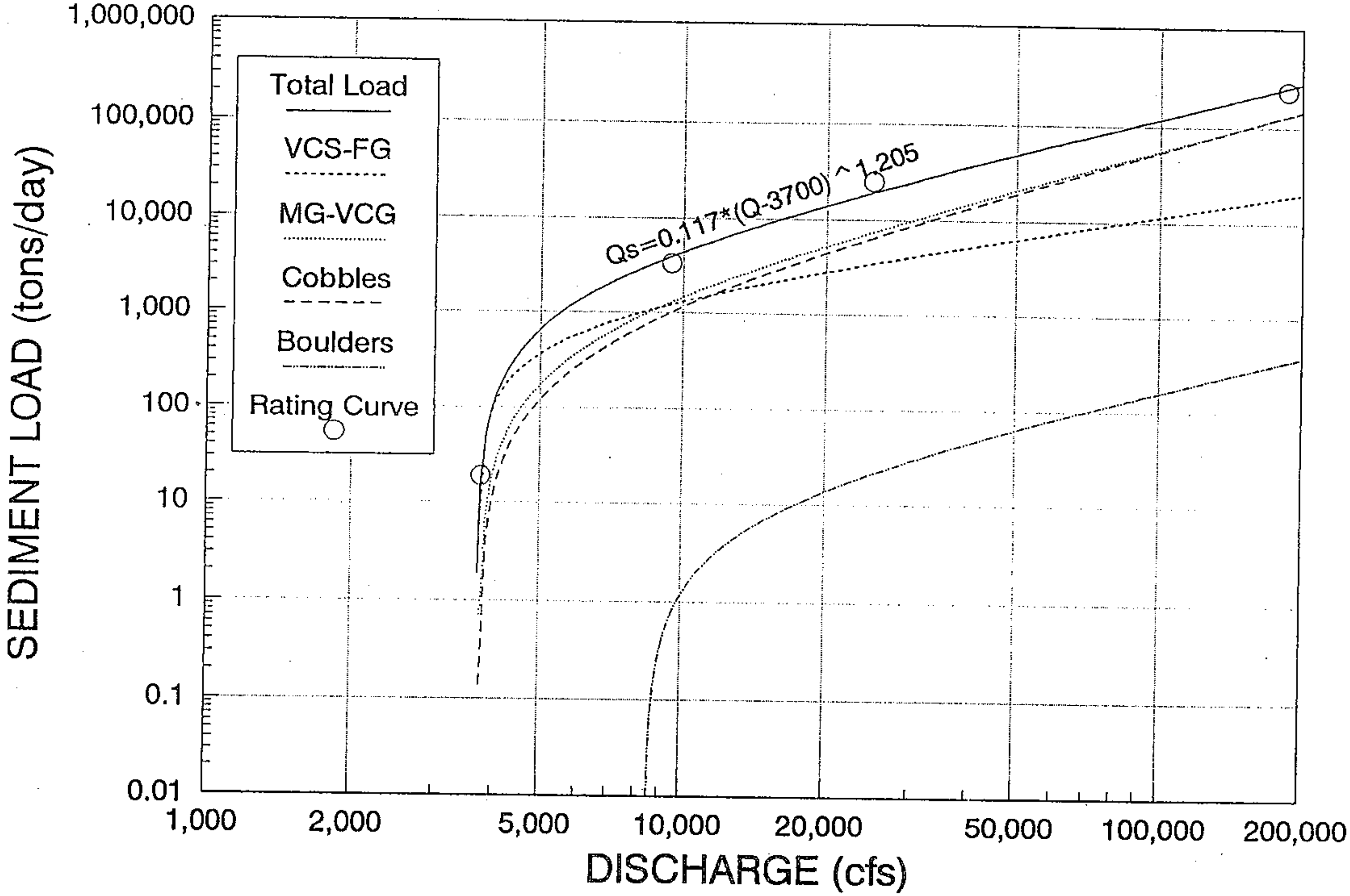


Figure 3.15. Inflowing bed material load rating curve for the Middle Fork study reach.

3.4.2. Model Results

Reach Descriptions

To facilitate analysis of the model results, the study reach was divided into eight reaches between the dam site and the upstream limit of the reach along the Middle Fork. The North Fork between the confluence with the Middle Fork and the North Fork Dam was treated as one reach. These reaches are shown in Figure 3.7 and are generally delineated by significant hydraulic controls.

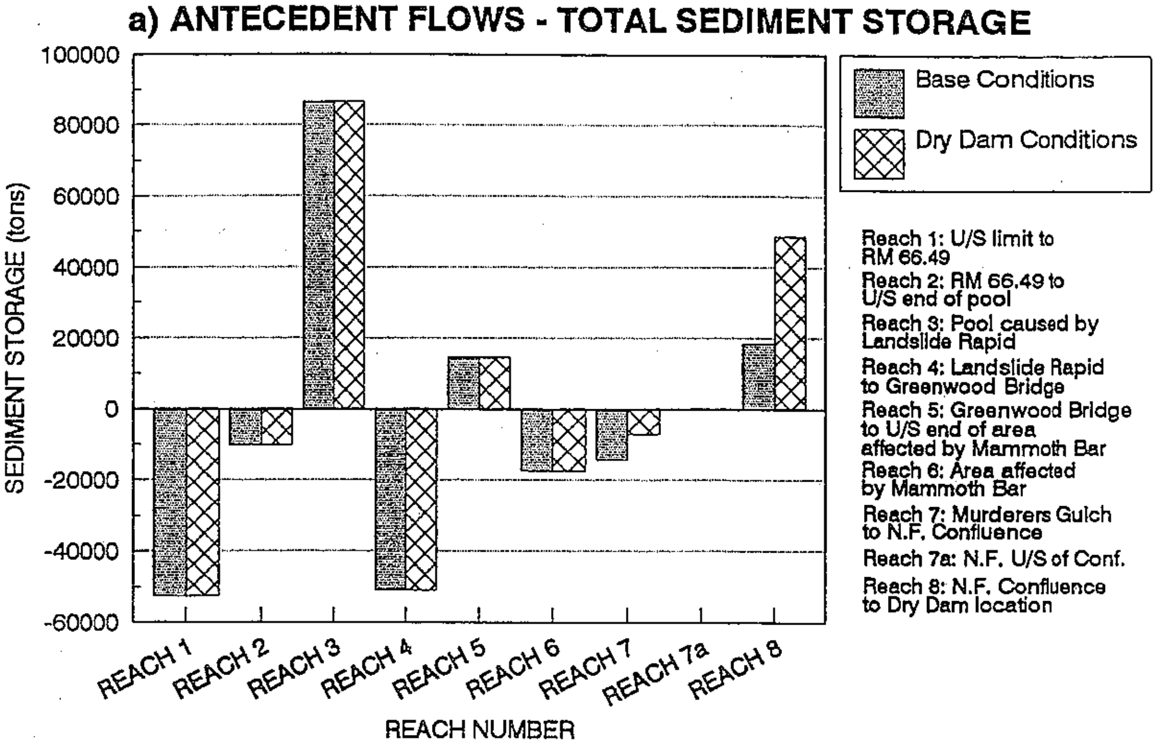
Comparison of Results

Figure 3.16 shows the sediment storage by reach for antecedent flows for both base conditions and project conditions. Prior to the design event, the upstream impact of the dam is limited to Reaches 7 and 8; storage in all other reaches is identical. The majority of material in transport for this range of flows is medium to coarse gravels and cobbles. The figure also shows that Reaches 1 and 4 (upstream study limit to RM 66.5 and between Landslide Rapid and Greenwood Bridge) are strongly degradational. Bed changes in these reaches will be minimal because of the very small sediment depth specified in the model. During the antecedent flows, the majority of the scour occurs in Reaches 1 and 4. Significant sediment storage is indicated in the pool caused by Landslide Rapid (Reach 3) (Plate 4). Nearly all of the upstream supply to this reach is trapped during antecedent flows.

Figure 3.17 shows sediment storage by reach during the design event and succedent flows for both base conditions and Dry Dam conditions. For base conditions, the design event causes scour in Reaches 1 and 2, Reach 3 (the pool caused by Landslide Rapid) traps approximately 20 percent of the upstream supply and Reaches 4 and 5 are relatively stable. Reach 6 is the area affected by the constriction at Murderers Gorge (Plate 5). This reach extends from the downstream end of Mammoth Bar to approximately one mi upstream of the bar (Plate 6). Approximately 90 percent of the inflowing bed material load is deposited in this reach. Reaches 7 and 7a are relatively stable and Reach 8 shows significant scour.

For base conditions, the succedent flows cause very little response in Reaches 1 through 5. Nearly 20 percent of the material stored during the design event in the reach containing Mammoth Bar is removed during the 2 yr of succedent flows. Most of this material (90 percent) is deposited in Reach 7, between Murderer's Gorge and the North Fork confluence (Plate 8). Upstream of the dam site, Reach 8 traps nearly 80 percent of the upstream supply. The primary difference between low flow and high flow is the response along and downstream of Mammoth Bar where a significant quantity of material is deposited upstream of the contraction during the design flood, and it is subsequently scoured during the lower succedent flows (Plate 6).

For the dry dam conditions for both the design event and succedent flows, adjustments in Reach 1 are nearly identical to base conditions. **Figure 3.18** is a plot showing the inflow hydrograph for the Middle Fork, the pool elevation for the dry dam, and the upstream extent of the backwater for dry dam conditions. This figure shows that the maximum upstream extent of the backwater is approximately RM 67, which occurs approximately 80 hr into the hydrograph. Because of the effects of the backwater, Reach 2 traps a significant amount of material (nearly 40 percent of the upstream supply) during the design event. Reach 3 traps a similar volume of material during both conditions. This amount,



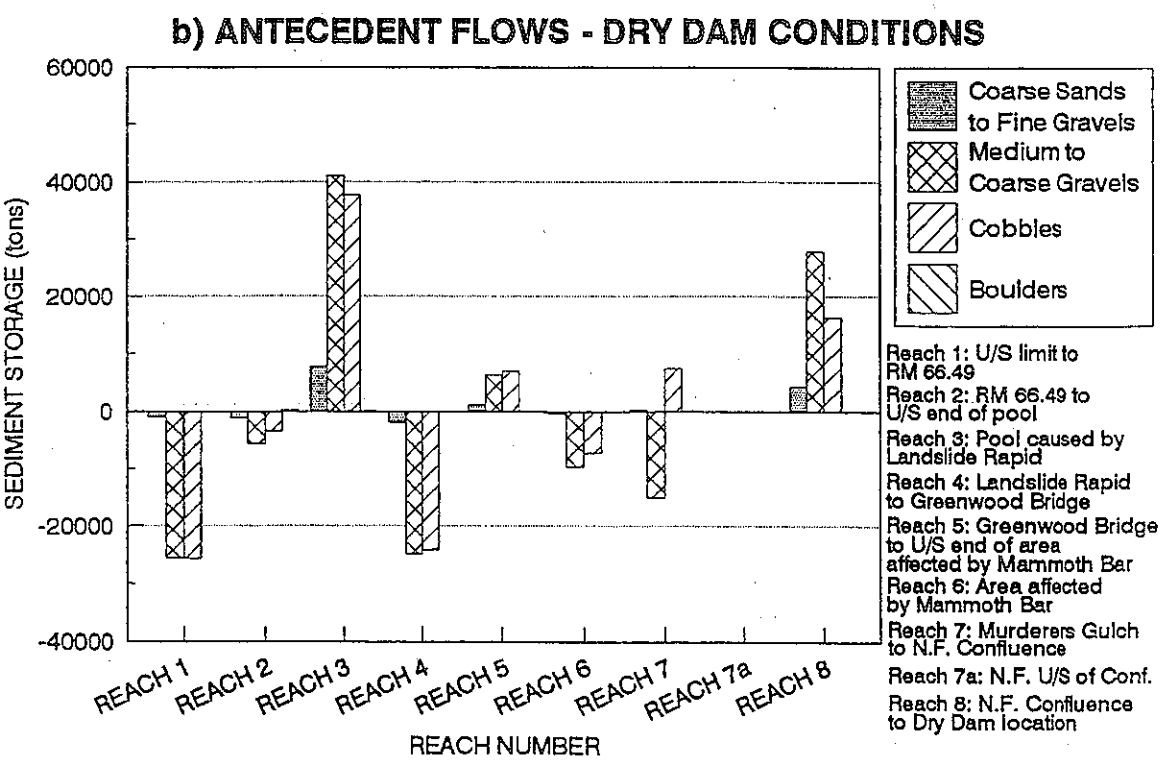


Figure 3.16. Sediment storage by reach for antecedent flows for both base conditions and dry dam conditions

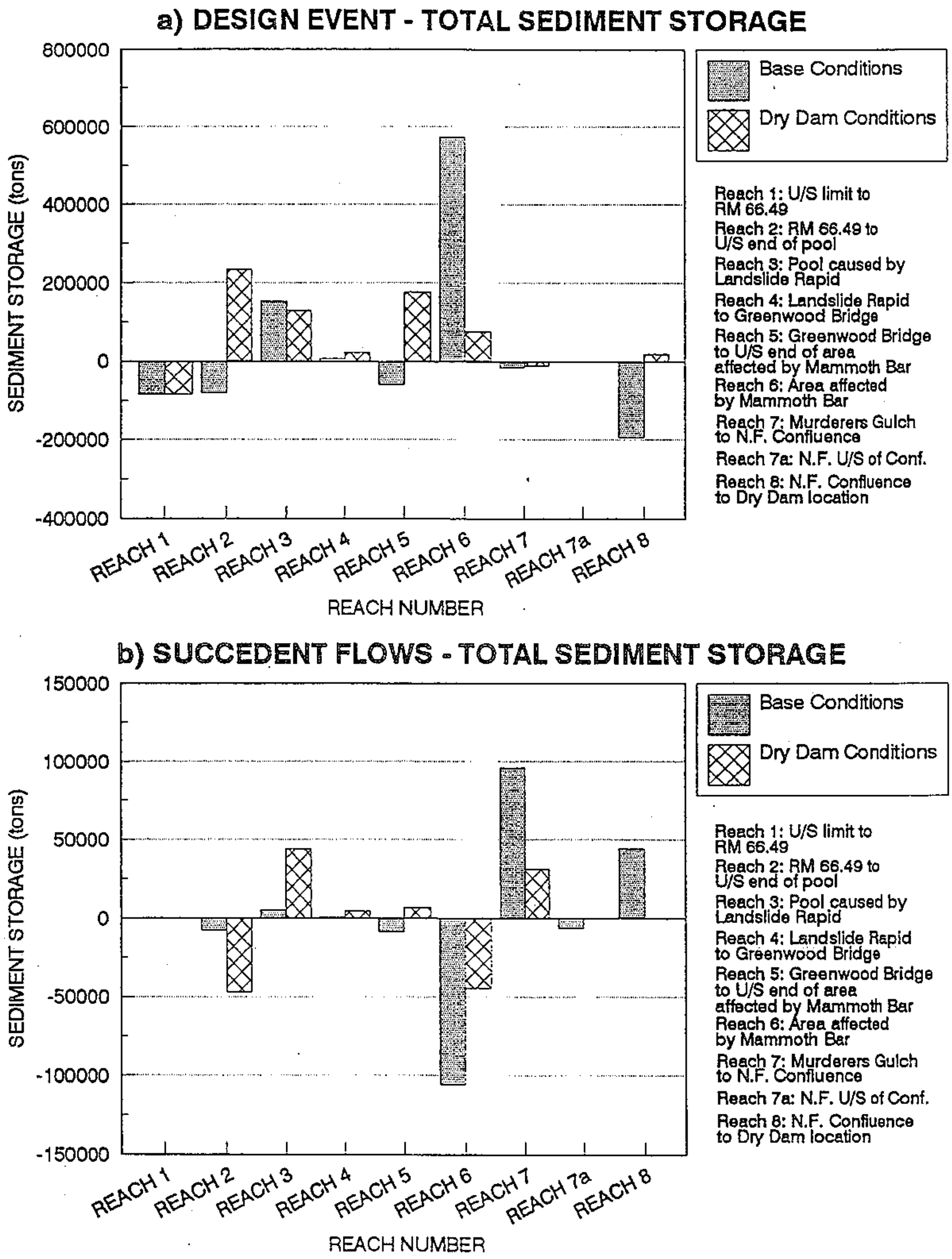


Figure 3.17. Sediment storage by reach for the design storm and succedent flows for both base conditions and dry dam conditions.

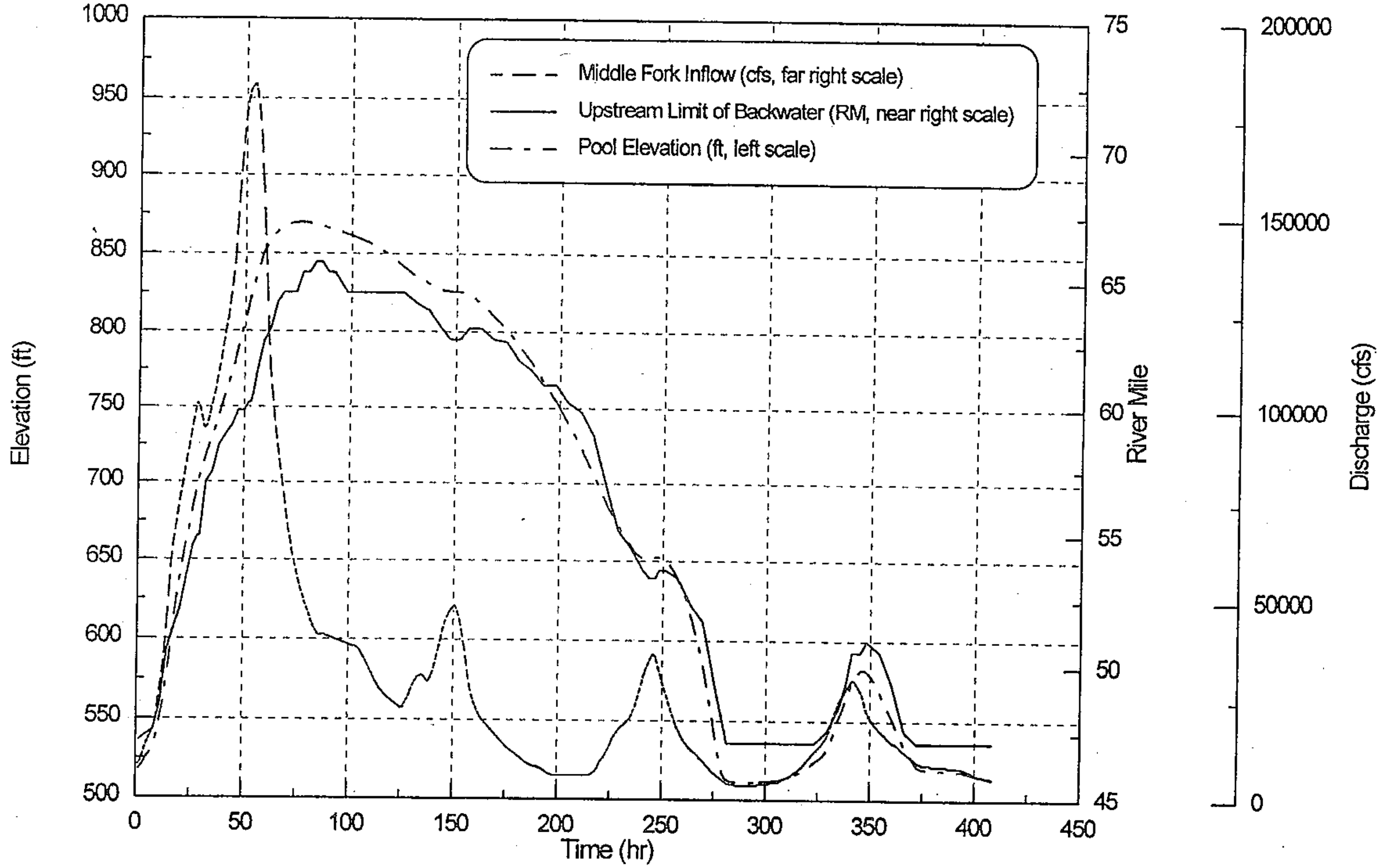


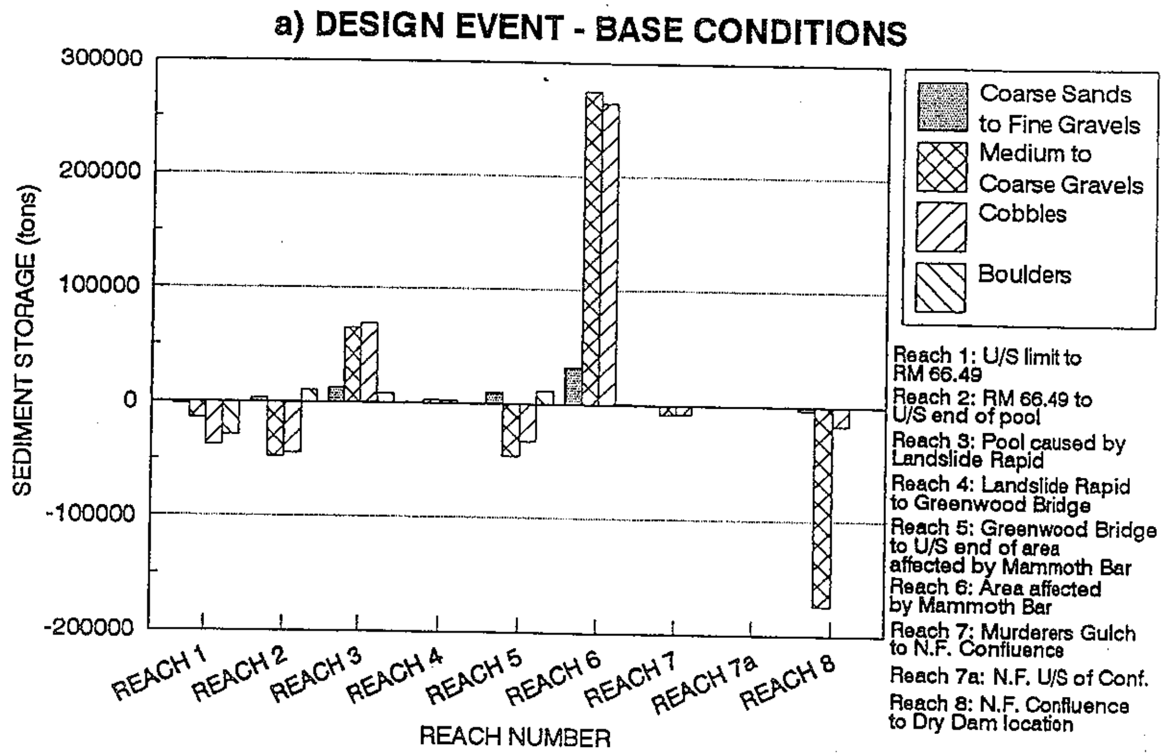
Figure 3.18. Middle Fork inflow hydrograph, Dry Dam pool elevation, and upstream limit of backwater effects for the design event.

however, is a greater proportion of the upstream supply for dry dam conditions (30 percent) for dry dam conditions versus 20 percent for base conditions) Reach 4 remains relatively stable; while Reach 5 becomes depositional under dry dam conditions for the design event. This reach traps the majority (70 percent) of the remaining bed material in transport. This leaves a relatively small quantity of material in transport and available for deposition in Reach 6. Approximately 90 percent of the remaining material in transport is trapped in this reach. Because the upstream supply is virtually cut off for these conditions, the reaches between Murderer's Gorge and the dam site (Reaches 7 and 8) show little or no deposition during the design flows.

During the succedent flow period for the dry dam condition, Reach 1 is unaffected as under the normal depth runs. Similar amounts of material are entrained along Reach 2 and deposited in Reach 3 compared with the base conditions run. Reaches 4 and 5 trap minimal amounts of material for these conditions. As with the normal depth run, Reach 6 scours during succedent flows though at less than half the rate. Most of the material scoured from Reach 6 is trapped in Reach 7 upstream of the North Fork confluence. The reach between the confluence and the dam site (Reach 8) shows little change under dry dam conditions during the succedent flow period.

Figures 3.19 and 3.20 show sediment storage by size class for the base and dry dam conditions runs. These figures show that the majority of the material in transport (being deposited or entrained) ranges from medium gravels to cobbles. Little sand is transported because it is not well represented in the bed material, and the boulders that are present are generally too large to be entrained, except at the highest flows. The distribution of deposited material among the size ranges is relatively consistent in all reaches except Reach 8 just upstream of the dam site. In this reach, most of the material deposited under base conditions is in the medium to coarse gravel size range. This is most likely because little of the coarser cobble sized material is transported through the constriction at Murderer's Gorge. It is also important to note that nearly 10,000 tn of coarse gravel to cobble sized material is scoured from this reach during succedent flows under dry dam conditions. This is a significant result considering the potential for damage to the sluice gates that could result from the passage of this material. The fact that this material is removed under relatively low flow conditions when the velocities through the sluice gates will also be relatively low, however, tends to reduce the potential for damage.

The amount of material transported past the dam site for both base and dry dam conditions during antecedent, design event, and succedent flows is shown **Figure 3.21**. The amount of material passing through the sluice gates during antecedent flows, is approximately 30 percent of what would be carried past that point under run-of-the-river (base) conditions (approximately 17,400 tn versus 54,800 tn for the 2-year period). Approximately 4 percent of this material (approximately 760 tn) is cobbles, and the remainder is predominantly medium to coarse gravels. During the design event, a small amount of sediment (approximately 72 tn) passes through the dam of which nearly 80 percent is coarse sand to fine gravel size range. The remaining 20 percent (approximately 14 tn) is made up of medium to coarse gravels. Because nearly all the upstream supply is trapped upstream of the dam during the design flood, the amount of bed material passing the dam during succedent flows is nearly 3 time greater for dry dam conditions than for base conditions (approximately 36,400 tn versus 12,800 tn for the 2-year period). Approximately 65 percent of this material (approximately 23,600 tn) is in the medium to coarse gravel size range, 31 percent (approximately 11,200 tn) is in the coarse sand to fine gravel size range, and the remainder (approximately 1,700 tn) is cobbles.



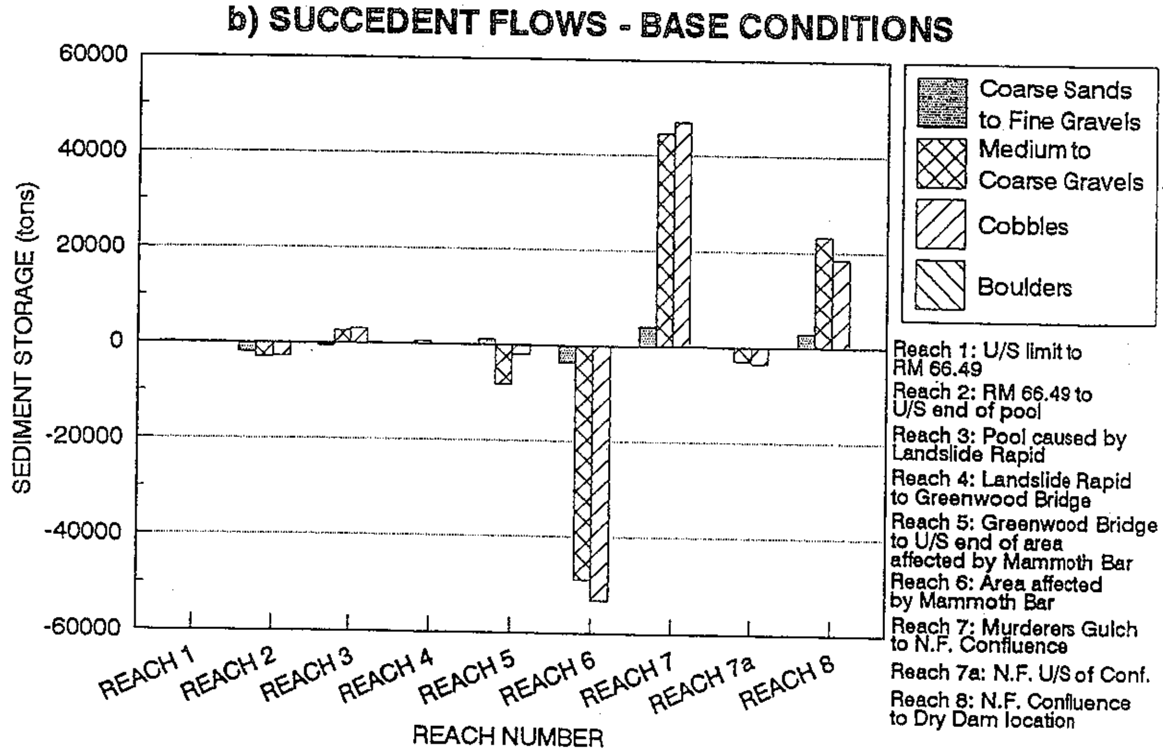


Figure 3.19. Distribution of sediment in storage by reach for the design event and succedent flows for base conditions.

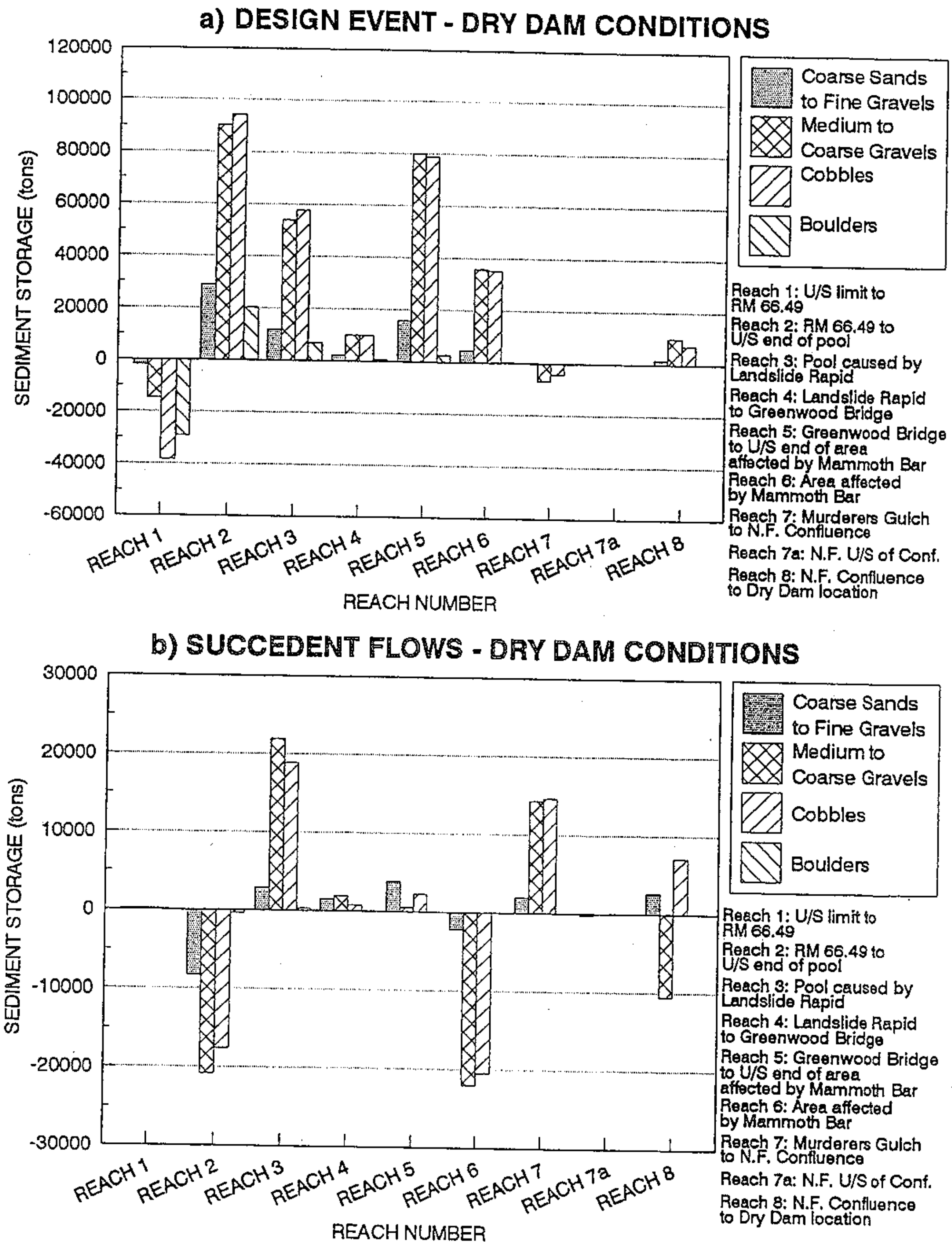


Figure 3.20. Distribution of sediment in storage by reach for the design event and succedent flows for dry dam conditions.

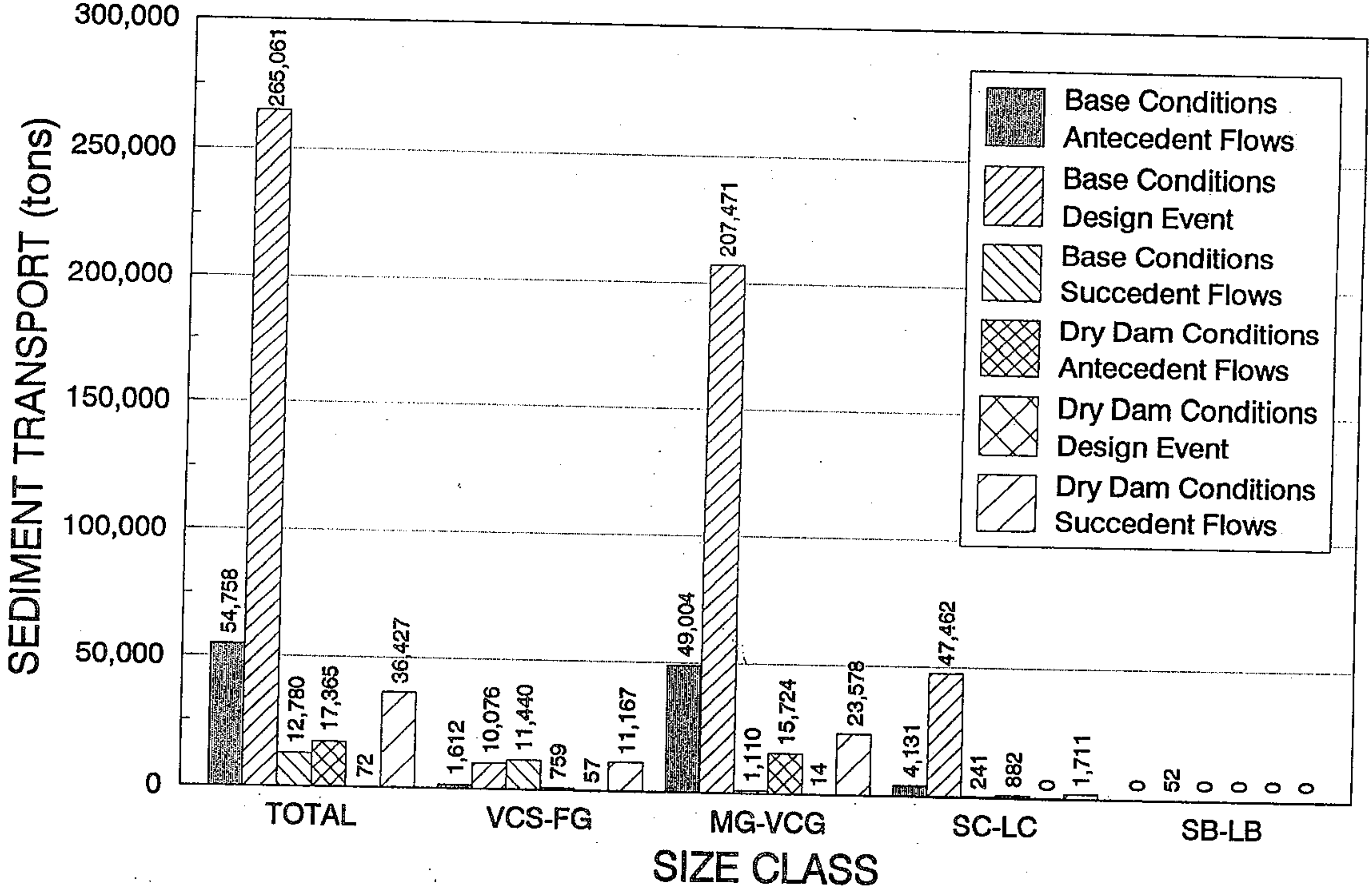


Figure 3.21. Quantity of sediment, by size fraction, transported past (or through) the dry dam for base conditions and dry dam conditions for antecedent, design event and succedent flows.

Figure 3.22 shows the bed material transport rate through the sluices during the design flood hydrograph. The total inflow hydrograph to the study reach (combined North and Middle Fork flows) and the outflow hydrographs are also shown in the figure. The figure indicates that material is only transported through the sluices during outlet discharges less than approximately 30,000 cfs. During higher flows, upstream backwater causes deposition of sediment and eliminates transport to the sluices. Because the HEC-6 model does not adequately simulate conditions near the inlet to the sluices, it is possible that local scour due to flow acceleration at the entrance of the sluices could entrain additional material deposited there during low flows prior to the flood. Depending on the inlet design, a concrete apron in the channel bed upstream of the sluice entrance may eliminate this source of material. Because material is passed though the sluices during the lowest flows when velocities through the sluices are also relatively low, abrasion caused by this material may not be significant. This must be carefully considered in selecting the materials for lining the sluices.

Scour in the downstream channel resulting from essentially clear water releases from the dry dam during the design event is not expected to be a problem since this reach of the river is flowing primarily on bedrock.

3.4.3. Sediment Impacts on Flood Control Sluices

Literature Search

This section summarizes the results of a literature related search the potential for damage associated with sediment transport through the outlet works and includes material from a report prepared for the COE by Henry T. Falvey & Associates, Inc. (HFA). The HFA report provides background information on problems experienced with high head, gated, and ungated outlets. The HFA study encompassed problems of cavitation, abrasion, downstream scour, vibration, maintenance, and debris. For this section, problems associated with sediment only will be discussed. Of the numerous dams included in the review, only four dams listed sediment related problems. These dams were (1) Punchiná Dam, Guatape River, Columbia, South America; (2) Ramganga Dam, tributary to the Ganga River in Uttar Pradesh, India; (3) Auburn Dam site coffer dam failure, North Fork of the American River, California; and (4) Oroville Dam, Feather River, California.

The Punchiná Dam has concrete lined outlet works designed for flow velocities of 115 ft/sec under heads of 218 ft. Gravel and sand abraded a 3 ft wide strip along the invert to a depth 2 in. below the upper reinforcement bars. A carefully finished, highly resistant concrete layer was cast on top of the original concrete.

The Ramganga Dam has concrete lined outlet works designed for flow velocities of 75 ft/sec under heads reaching 140 ft. Abrasion damage due to cobbles was eliminated by constructing a boulder masonry wall upstream of the inlet to divert the cobbles. The bottom 45° of the invert was coated with abrasion resistant paint.

The Auburn Dam site experienced the flood of record in February 1986. The diversion tunnel was complete and a coffer dam was in place. The coffer dam failed, which resulted in sediment deposition in the diversion tunnel and accumulation of as much as 30 ft of sediment in the river channel. The source of the sediment was not identified in the HFA report, however, numerous landslides occurred during the rapid drawdown resulting from the coffer dam failure. This material may account for a significant proportion of the sediment accumulated upstream of the coffer dam in addition to the bed material load trapped by the pool of the coffer dam.

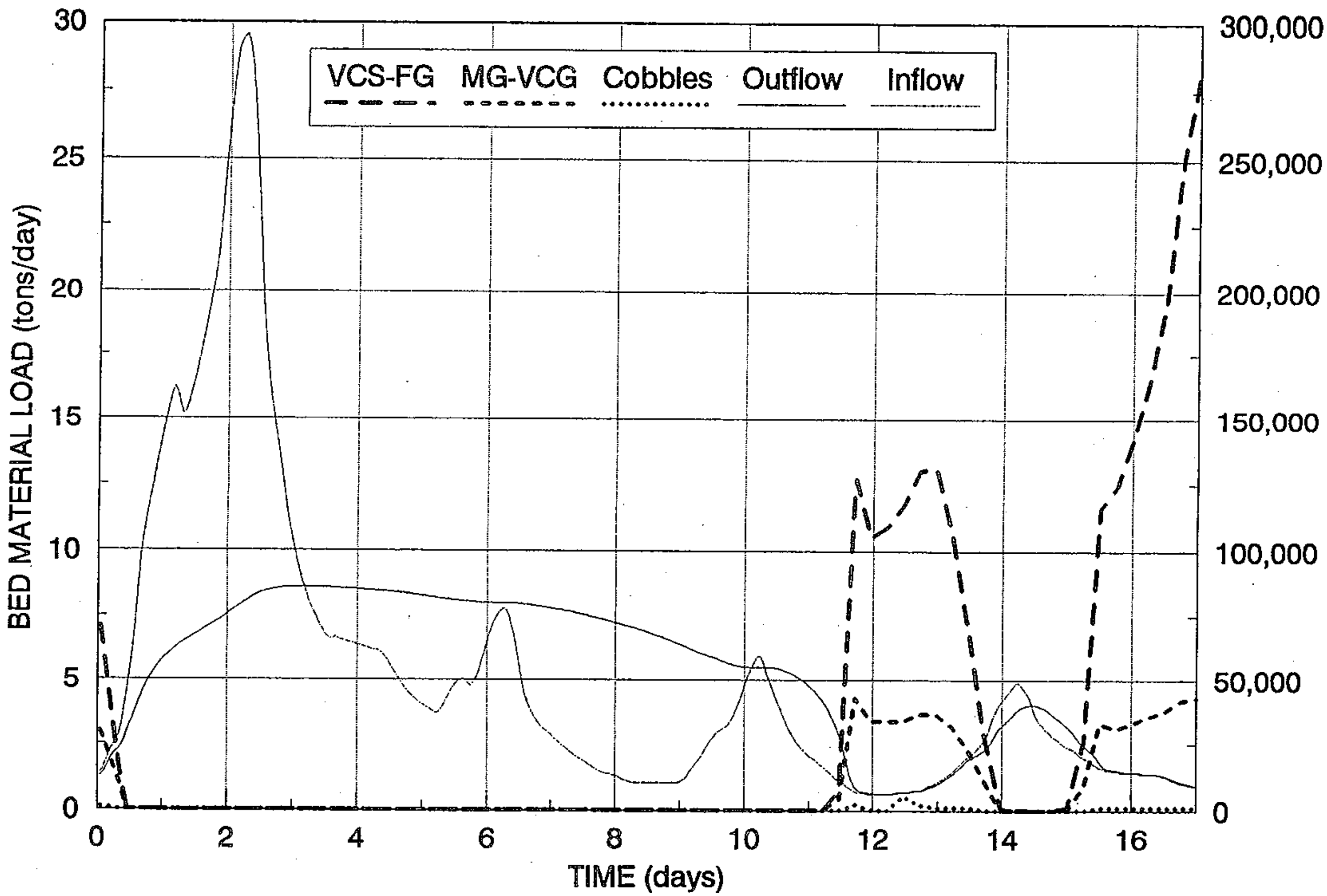


Figure 3.22. Water and bed material sediment hydrographs at the dry dam for dry dam conditions.

The Oroville Dam has concrete lined outlet works designed for flow velocities of 100 ft/sec under heads reaching 380 ft. The 1964 flood resulted in superficial abrasive wear.

Expected Impacts

It is clear that should sediment reach the outlet works when velocities are high, abrasion may become a significant problem. **Figure 3.23** shows the relationship between sediment load through the sluices and outlet discharge during the design event. The peak sediment load occurs at approximately 9,000 cfs, and it drops to zero for discharges greater than approximately 35,000 cfs. Apparently, backwater limits sediment transport for flows greater than approximately 9,000 cfs. The peak rate is quite small: approximately 1 lb per second of sediment.

Figure 3.24 shows the relationship between sediment load through the sluices and discharge during antecedent and succedent flows. This figure shows the same general pattern as for the design event with the peak sediment transport through the sluices occurring at discharges ranging from 8,000 cfs to 10,000 cfs, and it drops to zero for discharges greater than 35,000 cfs. The peak rate shown in Figure 3.24 is approximately 22 lb per second of bed material load, which is significantly higher than that predicted for similar discharges during the design event. The figure shows that the transport rates for cobble and medium to coarse gravel are consistent for both antecedent and succedent flows; while the rate of coarse sand to fine gravel transport increases by an order of magnitude after the design event. During the design event, this occurs because the finest material would be transported the farthest downstream into the backwater created by the dam. Thus, this material is more available for subsequent entrainment during succedent flows. This phenomenon may cause an increase in the rate of fine sediment transport through the sluices; therefore, it probably will increase with time as the material is progressively moved downstream toward the dam during succeeding events. It is also possible that sediment transport rates of the coarser materials will increase through the life of the project for the same reason. The data available from the sediment routing results is inadequate to assess the time required for this to become significant in relation to potential damages to the outlet works.

For comparison, **Figure 3.25** shows the sediment transport through the sluices for the normal depth runs under antecedent and succedent flows. For antecedent conditions, the with and without dam sediment transport are similar for flows less than 10,000 cfs. For flows greater than 10,000 cfs, the normal depth run indicates that sediment transport rates continue to increase, as would be expected.

The sluices, therefore, have minimal impact for low flows. For base conditions, the rates of medium gravel to cobble transport decrease by approximately two orders of magnitude after passage of the design event. This is probably due to armoring or coarsening of the bed material gradation. Rates of coarse sand to fine gravel transport increase moderately from antecedent to succedent flows.

Figures 3.23 and 3.24 indicate that for similar low flow conditions, sediment transport rates are much greater for antecedent and succedent conditions than for the design flood. The rates of sediment transport differ by an order of magnitude or greater. Investigation of the cause of this apparent discrepancy revealed that for discharges less than 20,000 cfs, water surfaces (used as the downstream control) were approximately 2 to 3 ft higher for the design flood than for antecedent or succedent conditions. This is illustrated in

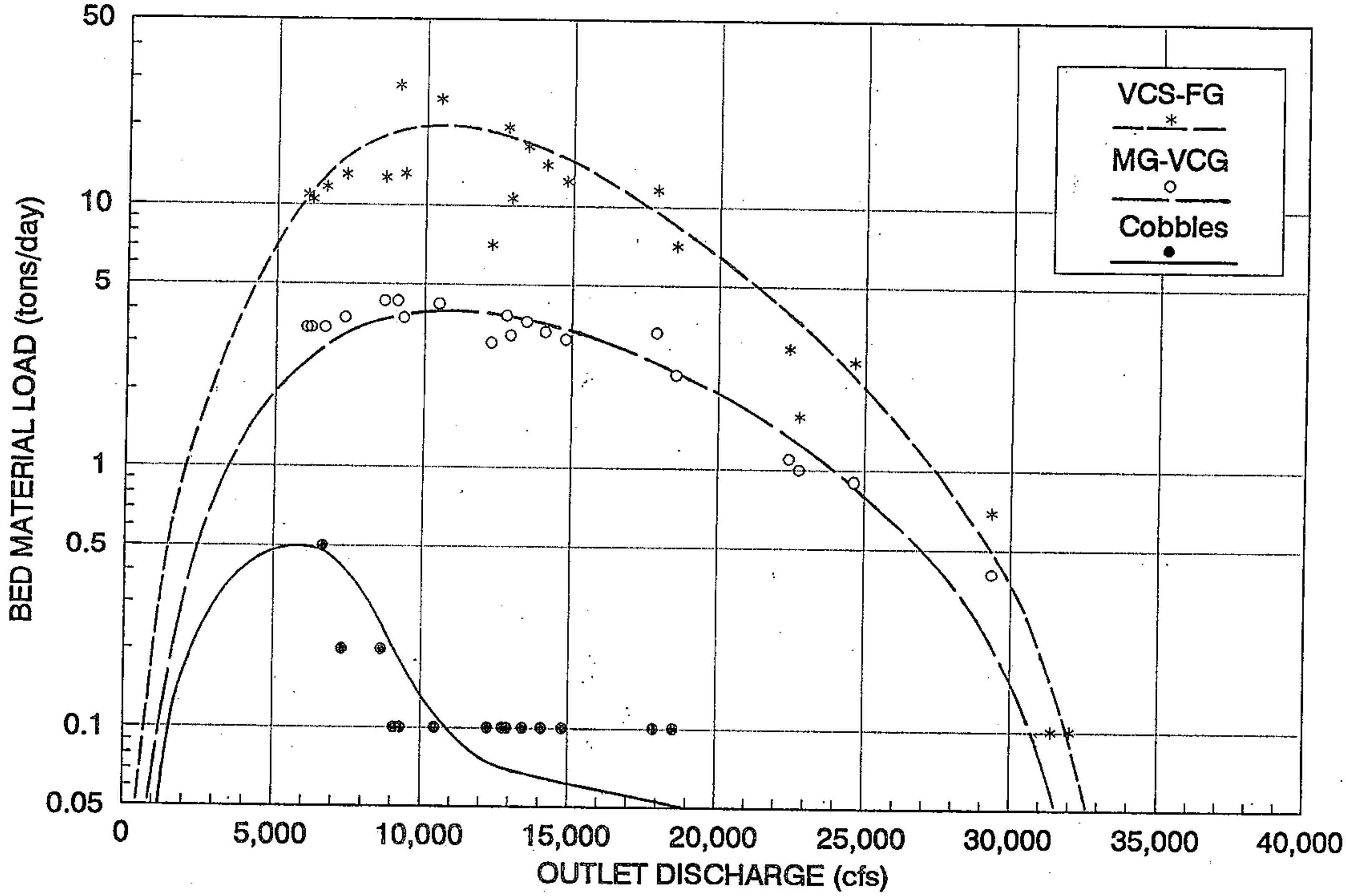


Figure 3.23. Relationship between sediment transport through the sluices and outlet discharge for the design event.

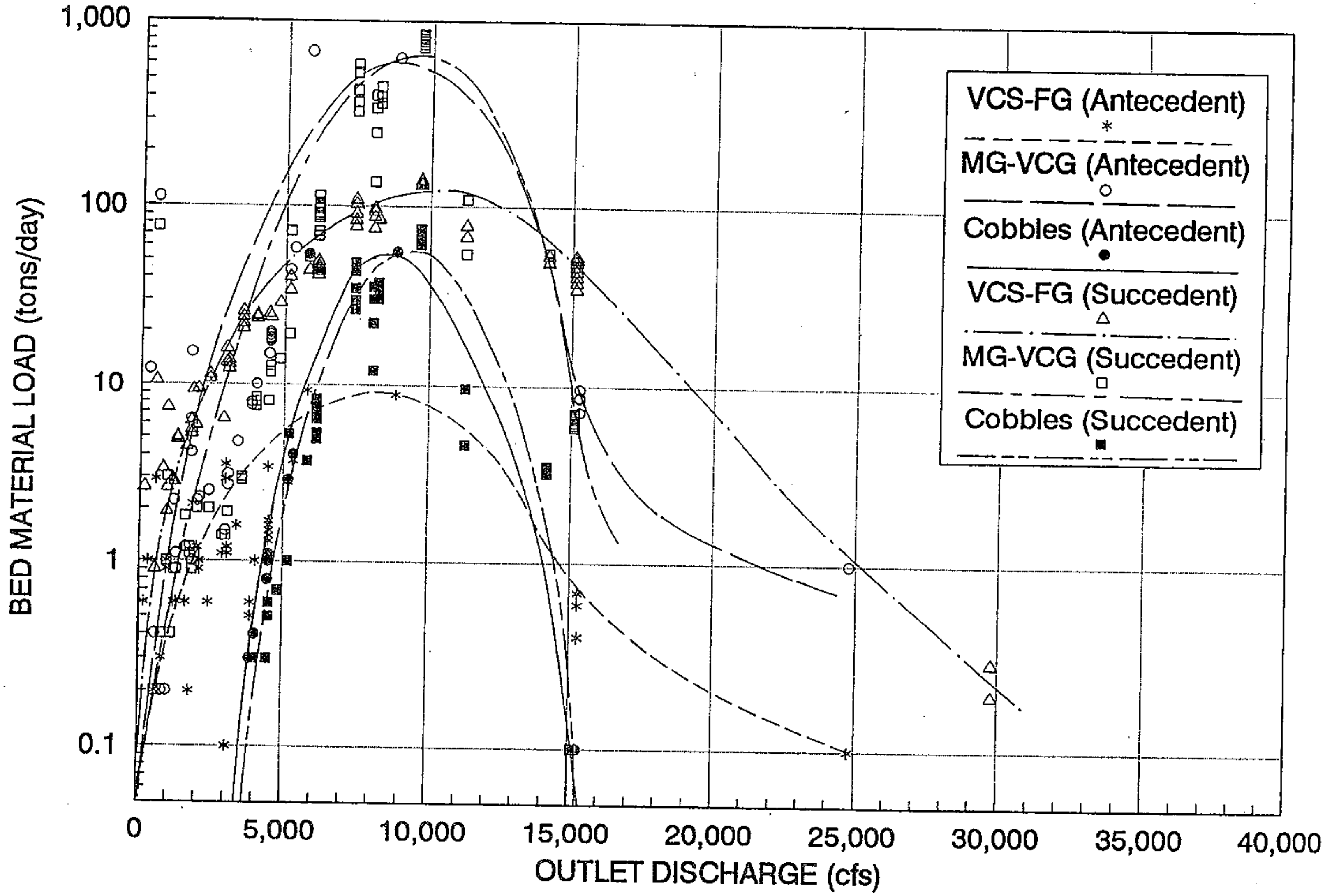


Figure 3.24. Relationship between sediment transport through the sluices and outlet discharge for the antecedent and succedent conditions.

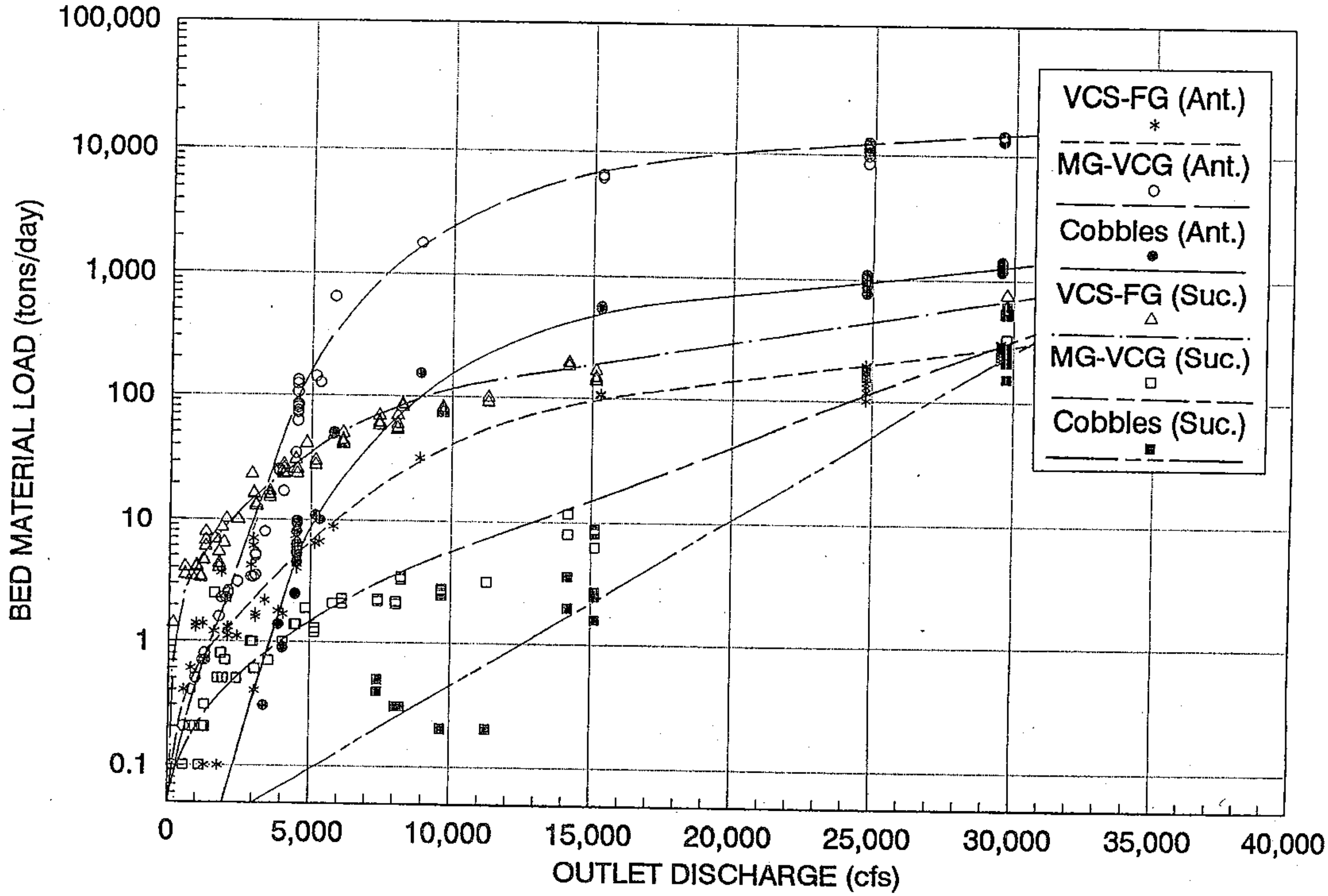


Figure 3.25. Relationship between sediment transport past the dry dam and river discharge for the antecedent and succedent flows for run-of-the-river (base) conditions.

Figure 3.26, which shows the stage plotted outlet discharge for the dam runs. As previously discussed, the pool elevations for the design event were taken from a routed hydrograph; while the antecedent and succedent flow elevations were based on a rating curve for the proposed sluice configuration, which were both provided by the COE. Apparently, the sluice configuration assumed for the flood routing and the single valued rating curve for the proposed sluices (Figure 3.5) were different. The differences in water surface elevations result in significant differences in the hydraulic characteristics immediately upstream of the sluices. Therefore, from a sediment transport and abrasion damage perspective, the sluice configuration used in the design flood routing is superior at least until bed elevation raises to the level of the outlet invert through deposition during low flows.

It should be noted that the sediment modeling performed for this study assumes that all of the bed material sediment supply will be derived from upstream reaches of the river. It does not consider the potential effects of landslides that may occur during rapid drawdown of the dry dam pool as the flood recedes. If landslides were to occur in the vicinity of the dam, the local supply could significantly increase, which may increase the potential for damages to the outlets.

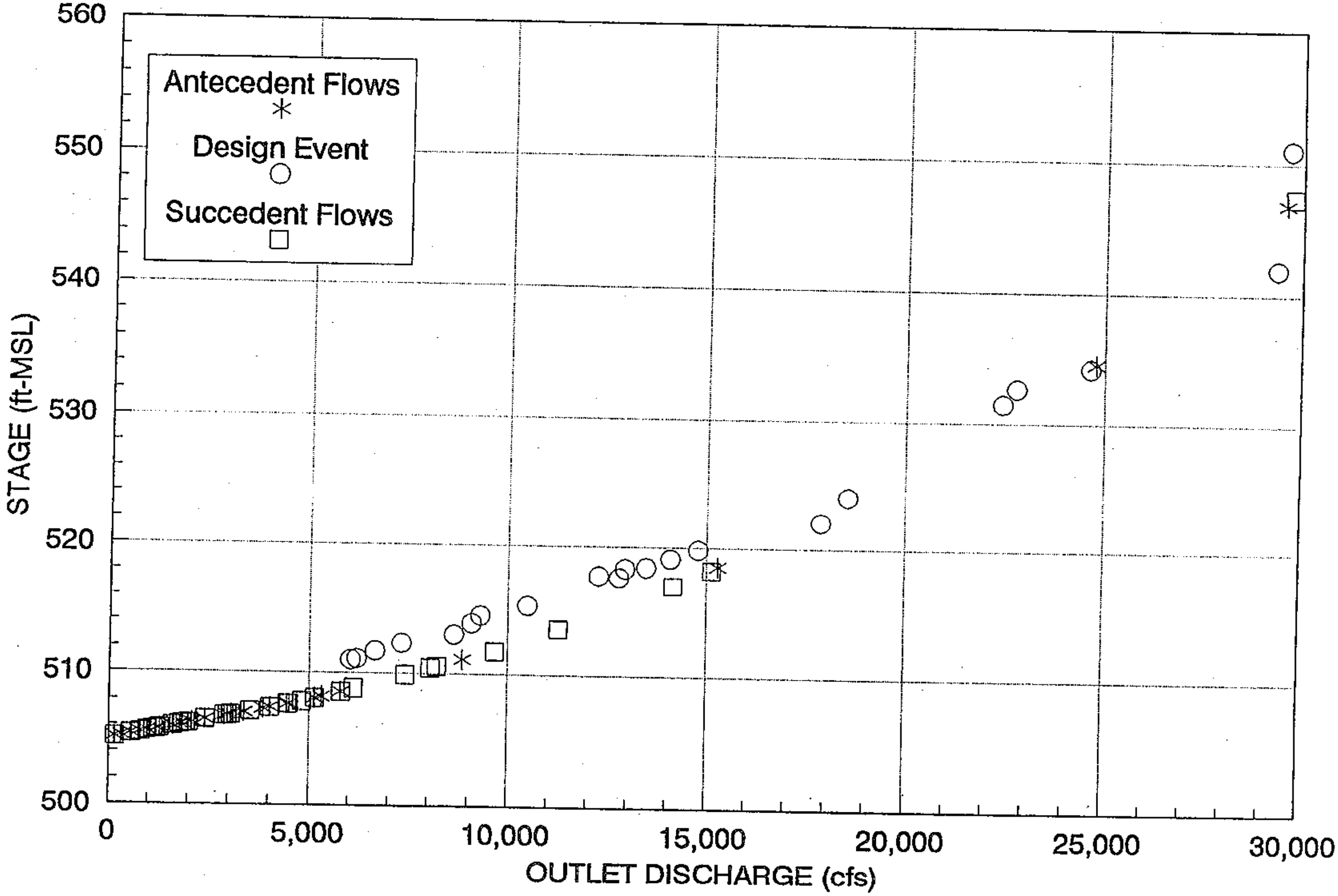


Figure 3.26. Comparison of pool stage versus discharge based on the routed stage hydrograph for the design event and the single valued rating curve for the proposed sluices.

4. DOWNSTREAM LEVEES ELEMENT

4.1. Hydrology

To perform the sediment transport and channel stability analyses presented in later chapters of this report, hydrologic data for the study reach were required. Necessary data included flow releases from upstream reservoirs, flows from the Notomas East Main Drainage Canal (NEMDC) and stages at the confluence with the Sacramento River. Much of the required data were provided by the COE. Additional analyses were required by RCE/Ayres to extend the data supplied by the COE to all the conditions necessary to perform the sediment transport and channel stability calculations. In addition, gage data were used to facilitate the analyses. The hydrologic data and additional analyses performed by RCE/Ayres are presented in this chapter.

4.1.1. Design Hydrographs

Hydrographs for the lower American River for five design scenarios were developed by the COE and provided to RCE/Ayres for use in this study. The five design scenarios, which represent different operating rules for Folsom Dam with one scenario including the Auburn Dry Dam, were developed with the goal of bracketing potential impacts on the river's geomorphic, sediment transport, and channel stability characteristics. **Table 4.1** summarizes the objective release and upstream storage conditions for the different scenarios. The hydrographs were developed by the Sacramento District by routing flows through Folsom Dam (and through the Auburn Dry Dam in Scenario 5). This represents flows at the upstream end of the project reach. In developing the flows for these scenarios, the COE assumed that Nimbus Dam would be in flood operation and would pass flows from Folsom Dam to the project reach downstream of Nimbus Dam without modification.

Table 4.1. Design Scenarios Analyzed in this Study.

Scenario	Folsom Dam Storage (ac-ft)	Objective Release (cfs)	Upstream Storage
1. Existing Conditions	400,000	115,000	None
2. FEMA 100-year	400,000	145,000	None
3. FEMA 100-year	590,000	115,000	None
4. 125-year	650,000	180,000	None
5. "Recommended Project", dry dam	400,000	115,000	Auburn Dry Dam

Table 4.2 summarizes the various return period floods and associated peak flows for the American River design hydrographs provided to RCE/Ayres. Plots of the hydrographs for each design scenario are presented in **Figures 4.1 through 4.5**. Lower return period hydrographs not provided for Scenarios 1 through 5 are the same as Scenario 1 (existing conditions). **Figure 4.6** shows the peaks of the hydrographs plotted on a flood frequency diagram.

Table 4.2. Flood Peaks for Design Hydrographs Provided by the Sacramento District.

Design Scenario	2-Year	3-Year	5-Year	10-Year	25-Year	50-Year	100-Year	200-Year	500-Year
1	16,000	35,000	67,000	86,700	115,000	115,000	234,000	430,000	**
2	*	*	*	*	142,900	145,000	190,000	420,000	**
3	*	*	35,000	46,200	113,500	115,000	160,000	400,000	**
4	*	*	35,000	35,000	97,200	160,200	180,000	320,000	**
5	*	*	59,000	72,600	99,600	115,000	115,000	115,000	350,000

*Same as Scenario 1
**Event not analyzed

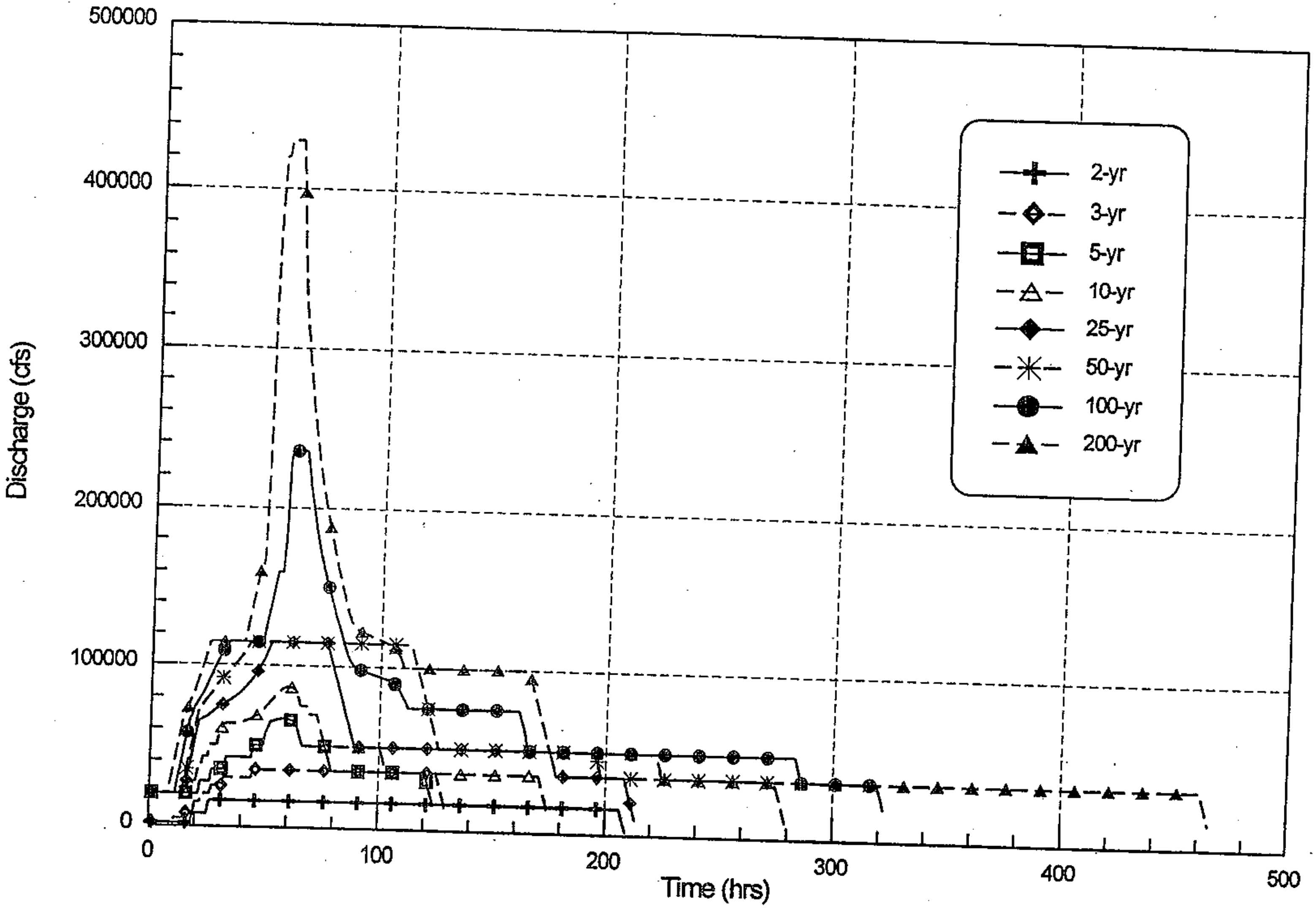


Figure 4.1. Lower American River upstream hydrographs, Design Scenario.

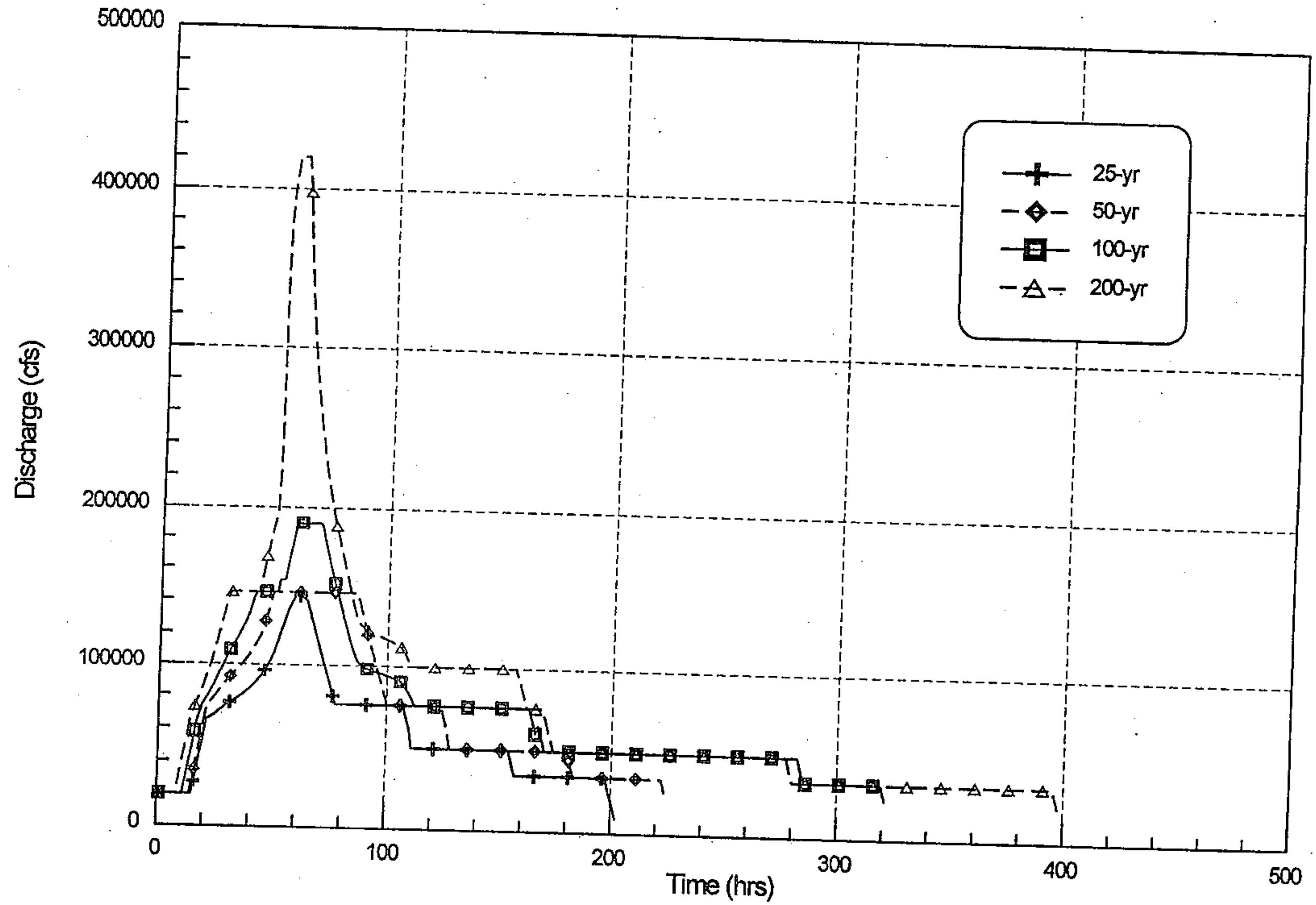


Figure 4.2. Lower American River upstream hydrographs, Design Scenario 2.

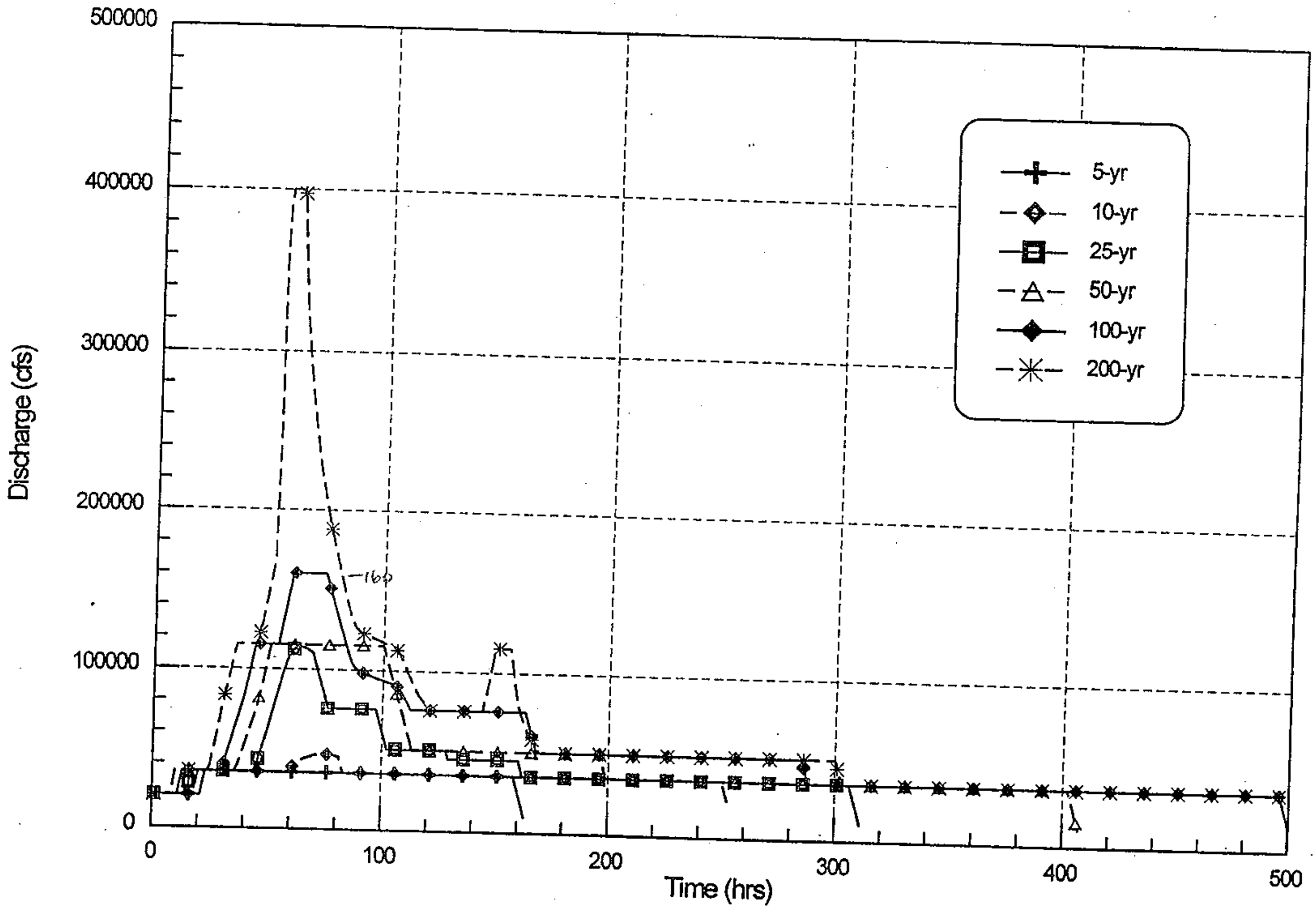


Figure 4.3. Lower American River upstream hydrographs, Design Scenario 3.

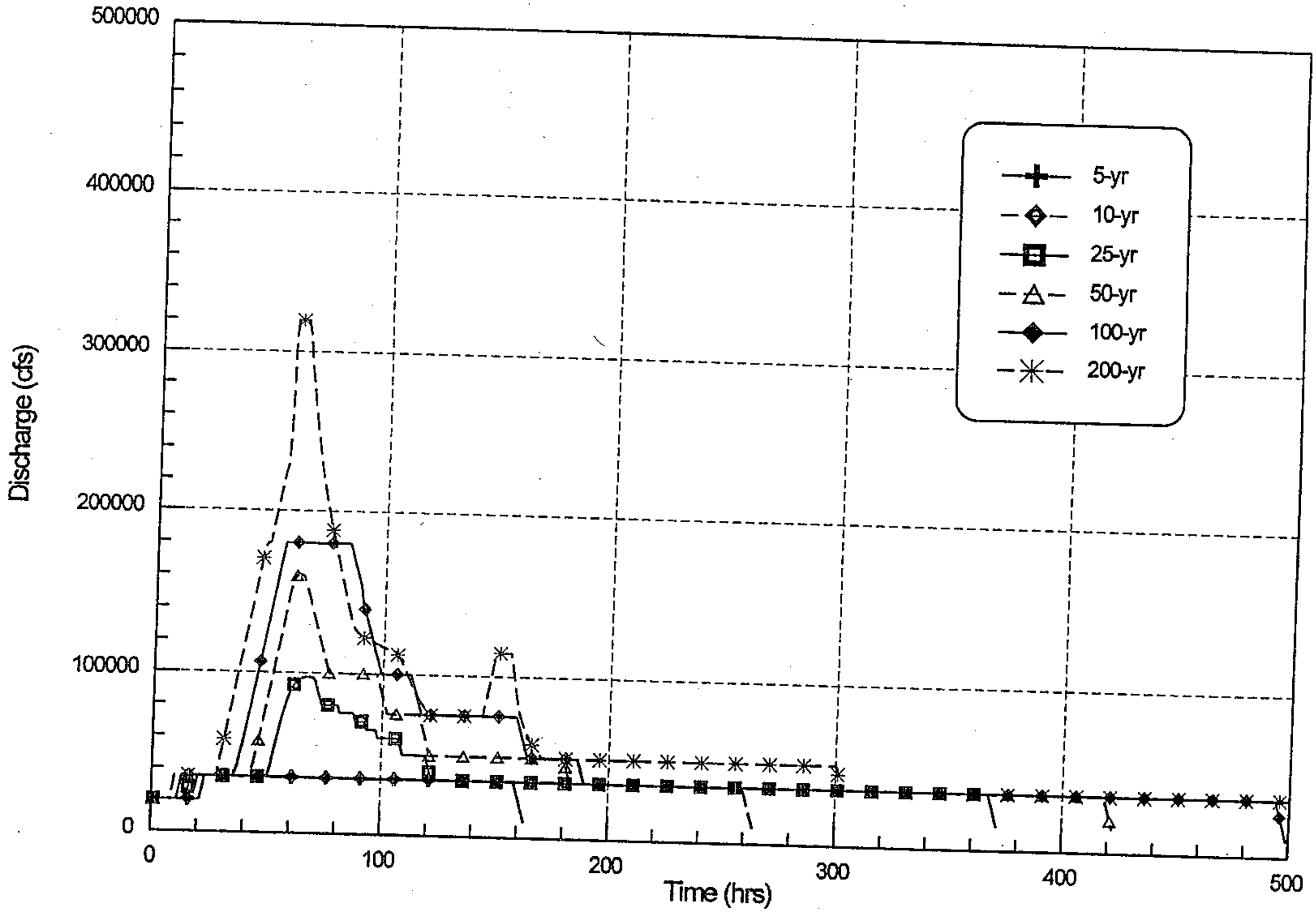


Figure 4.4. Lower American River upstream hydrographs, Design Scenario 4.

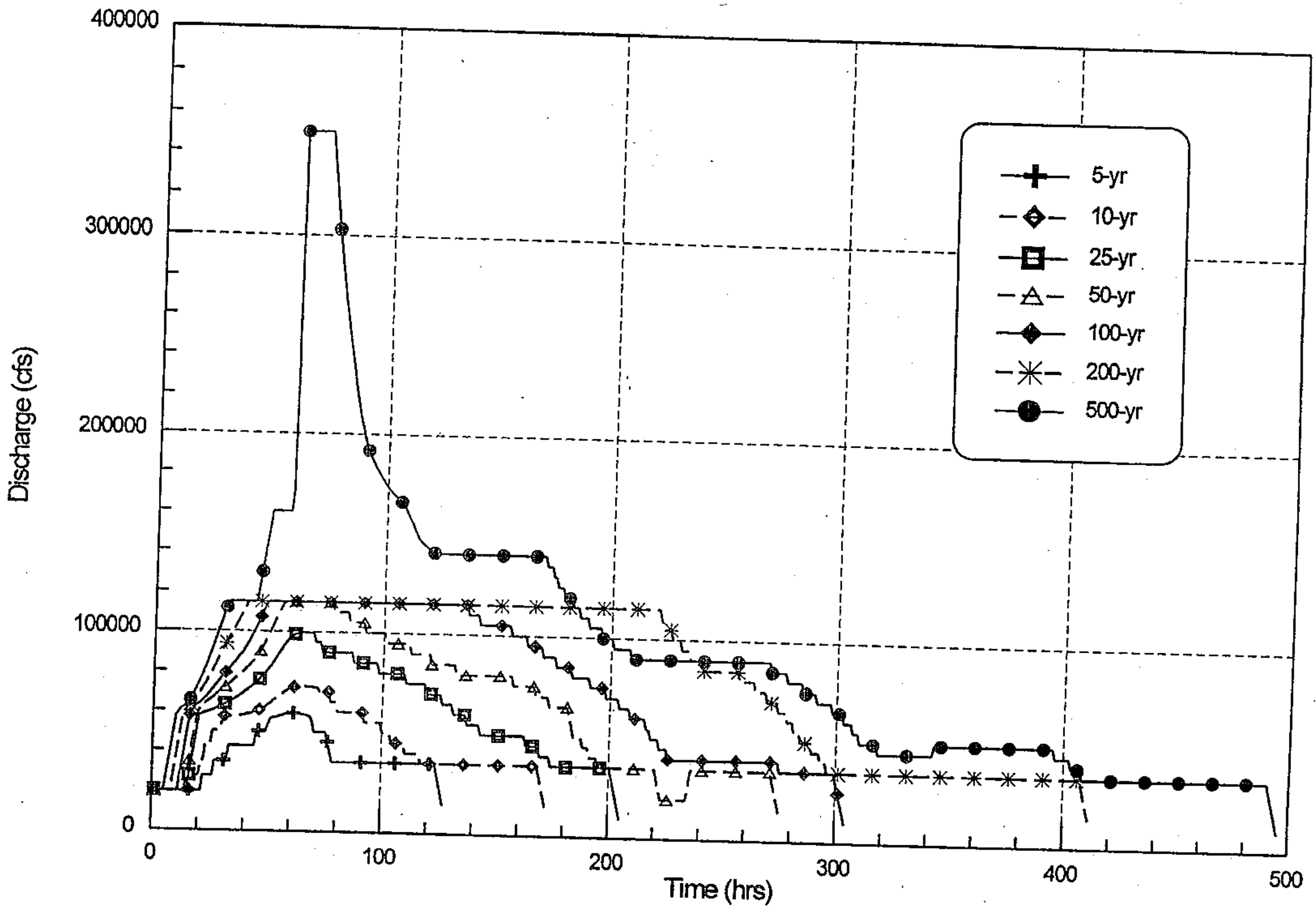


Figure 4.5. Lower American River upstream hydrographs, Design Scenario 5.

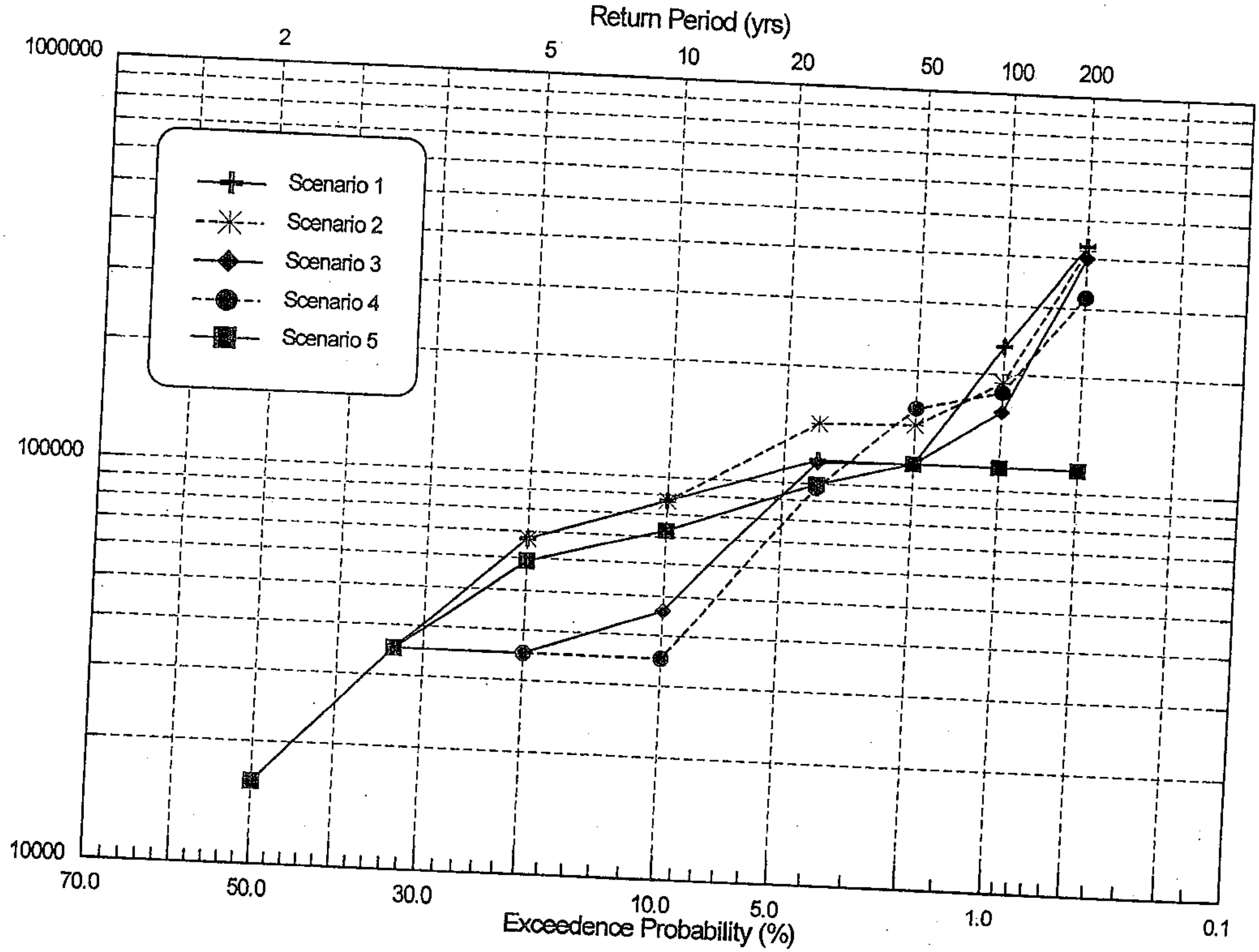


Figure 4.6. Flood frequency plot of Lower American River upstream flood peaks.

Design event (100-year) hydrographs for the NEMDC were developed by the COE for each design scenario and provided to RCE/Ayres. A plot of the hydrographs is given in **Figure 4.7**. Small differences in the hydrographs for each scenario are due to different routing effects caused by flows in the American River. Since the hydrographs provided by the COE did not extend to the end of the Folsom release hydrographs on the American River, they were extended by RCE/Ayres for use in the sediment transport and channel stability analyses by assuming a constant base flow at the end of each hydrograph. NEMDC hydrographs for the 2-, 3-, 5-, 10-, 25-, and 50-year events were developed by the COE assuming a 115,000 cfs release from Folsom Dam (**Figure 4.8**). These hydrographs were used for all the design scenarios on the American River. NEMDC flood hydrographs for the 200- and 500-year events were not provided. The 100-year NEMDC hydrograph developed for a Folsom objective release of 180,000 cfs (Scenario 4) was used with the 200- and 500-year events.

4.1.2. Downstream Stages

One hundred-year stage hydrographs for the mouth of the American River were developed by the Sacramento District for each design scenario. Plots of these stage hydrographs are given in **Figures 4.9 through 4.13**. As with the flood hydrographs, the COE routings used to develop the downstream stages did not extend to the end of the Folsom release hydrographs. The recession limb stages were extended by RCE/Ayres for use in the sediment transport and bank stability analyses by extrapolating the recession limbs using a gradually decreasing slope. The stage extensions assumed by RCE/Ayres are shown in Figures 4.9 through 4.13.

Routings used to produce the stage hydrographs at the mouth of the American River were performed by the COE only for the 100-year event using a specific scenario for concurrent flows on the Sacramento River, which covers only one of an infinite combination of possibilities. Routings to determine downstream stages for all the design scenarios and flood events (including a range of assumptions for concurrent flows on the Sacramento River) would require a prohibitive amount of effort. As an alternative, a simplified procedure was developed by RCE/Ayres in consultation with the COE with the goal of bracketing a realistic range of possibilities for downstream stages that would allow the potential impacts of the various design scenarios on sediment transport and channel stability to be addressed. First, a single valued rating curve was developed assuming concurrent flows on the two rivers with equal exceedence frequencies. This was accomplished using the flow duration and stage-discharge relationships for the I-Street Gage on the Sacramento River (just downstream of the American River mouth) and the flow duration curve for the American River at Fair Oaks Gage. The I-Street flow duration and stage-discharge curves were provided by the Sacramento District and are shown in **Figures 4.14 and 4.15**. Pre- and post-Folsom Dam flow duration curves developed by RCE/Ayres for the Fair Oaks Gage are given in **Figure 4.16**. For a given flow and exceedence frequency on the American River (post Folsom Dam), the corresponding flow on the Sacramento River for the same frequency was determined. The corresponding stage was then determined from the stage-discharge relationship, and a stage-discharge relationship for the mouth was constructed using the I-Street stages and the corresponding American River flows. Due to the limited resolution of the flow duration curves at low exceedence frequencies, the largest abscissa on the stage-discharge relationship developed using this method was 50,000 cfs. The relationship was extended to higher discharges using stage-discharge data provided by the Sacramento District. The resulting stage-discharge curve is given in **Figure 4.17**.

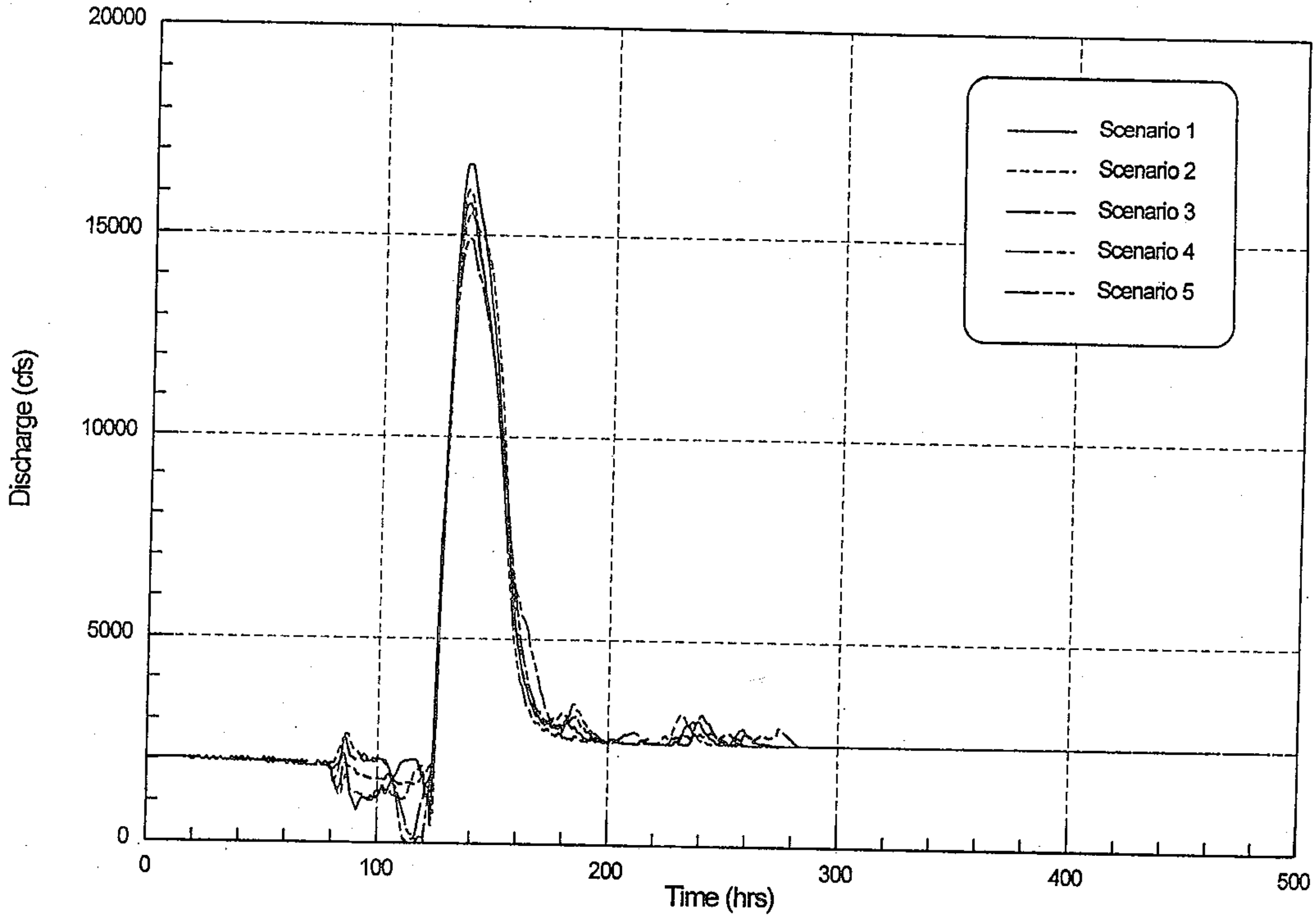


Figure 4.7. NEMDC 100-yr flood hydrographs from COE Design Scenario simulations.

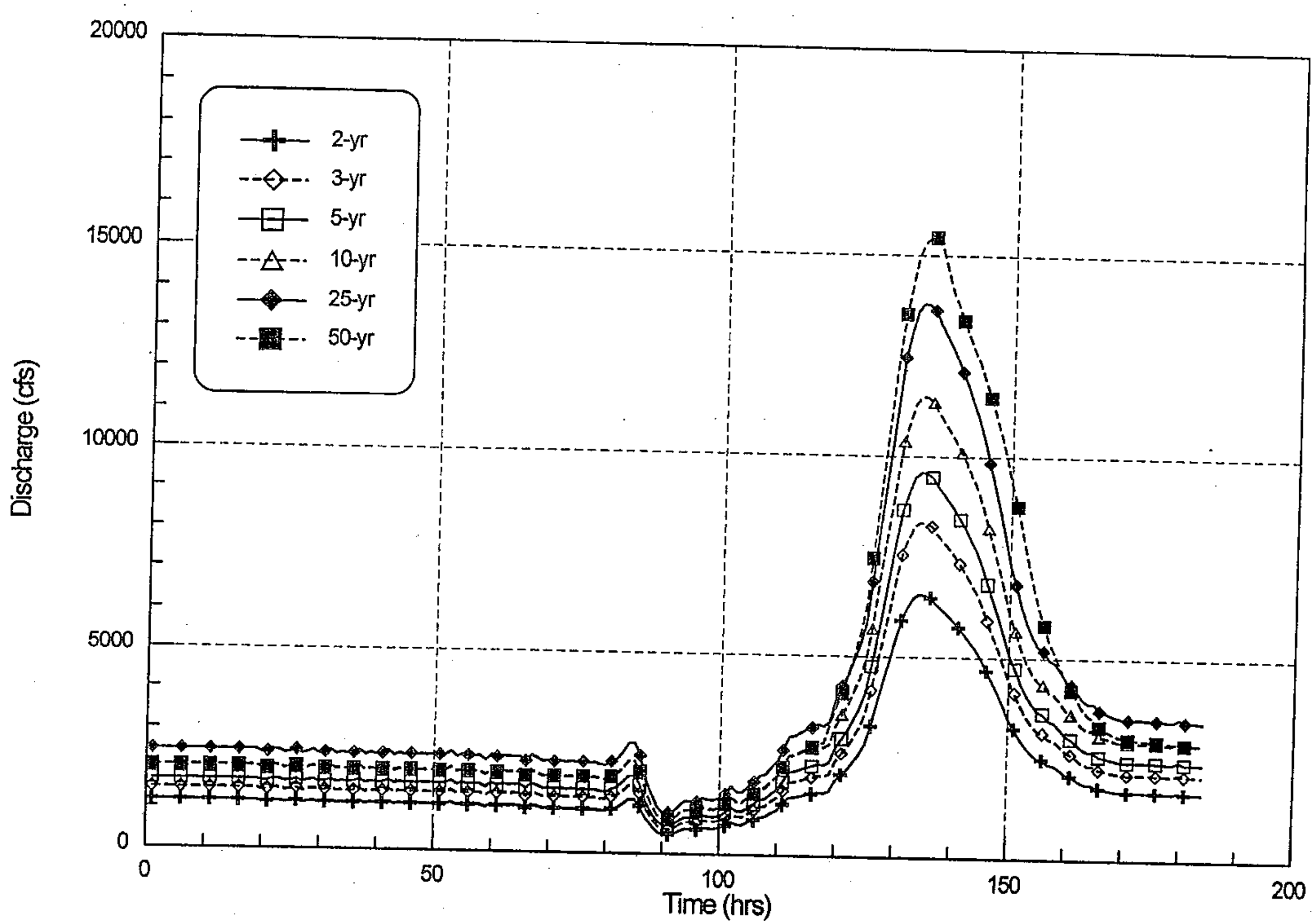


Figure 4.8. NEMDC flood hydrographs, 115,000 cfs Folsom Dam objective release.

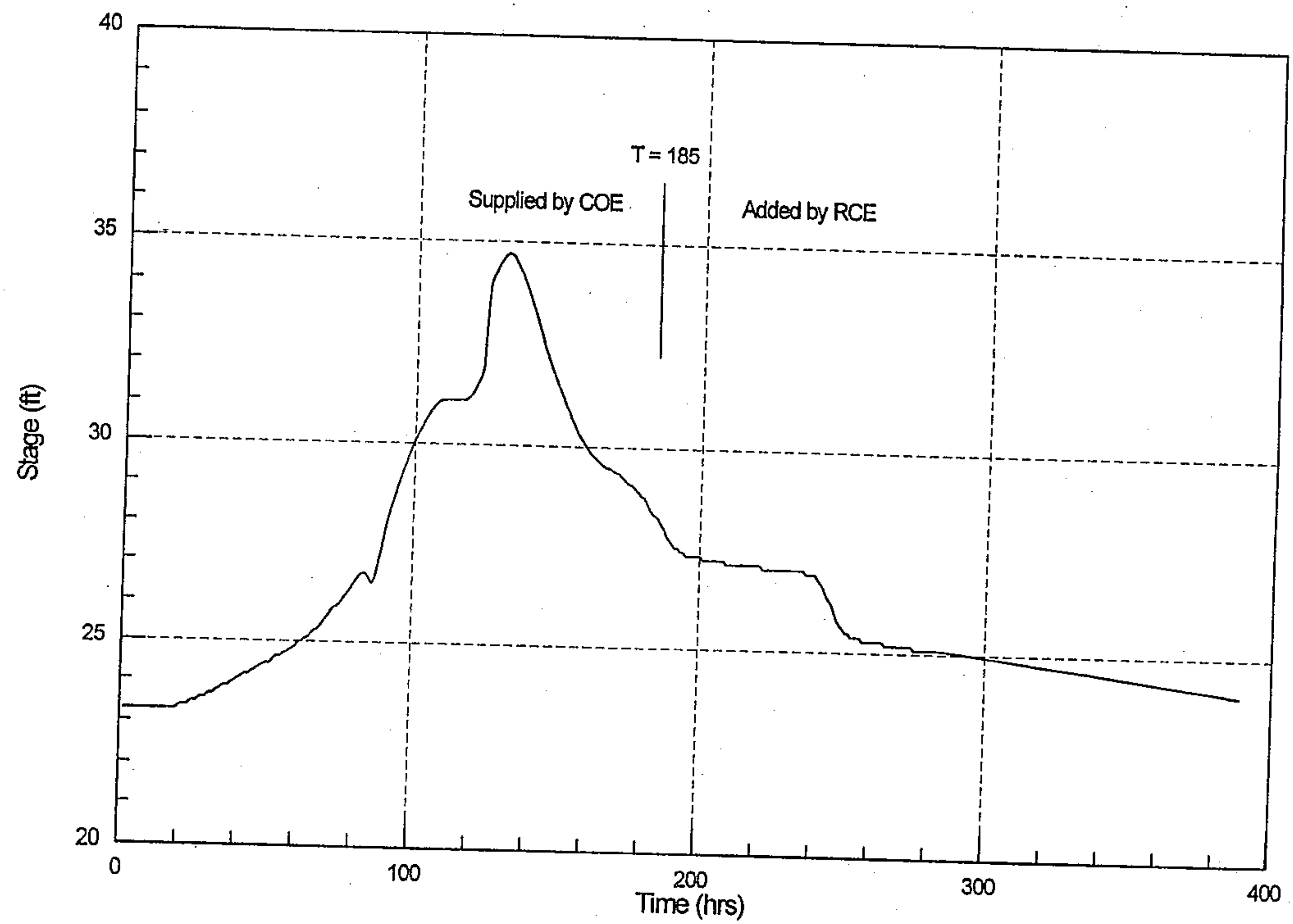


Figure 4.9. Stage hydrograph at the mouth of the American River, 100-yr event Design Scenario 1 (from COE simulations).

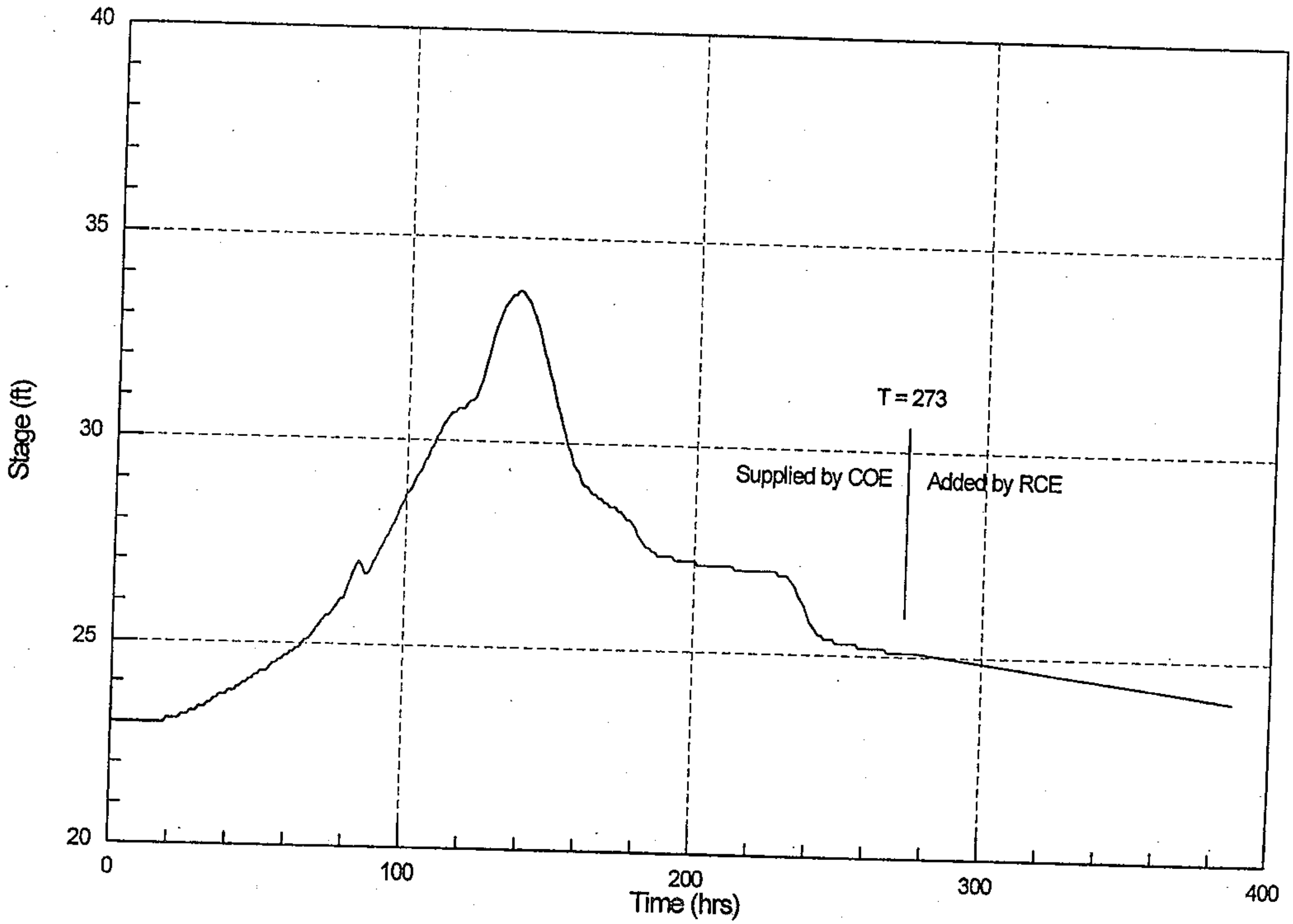


Figure 4.10. Stage hydrograph at the mouth of the American River, 100-yr event Design Scenario 2 (from COE simulations).

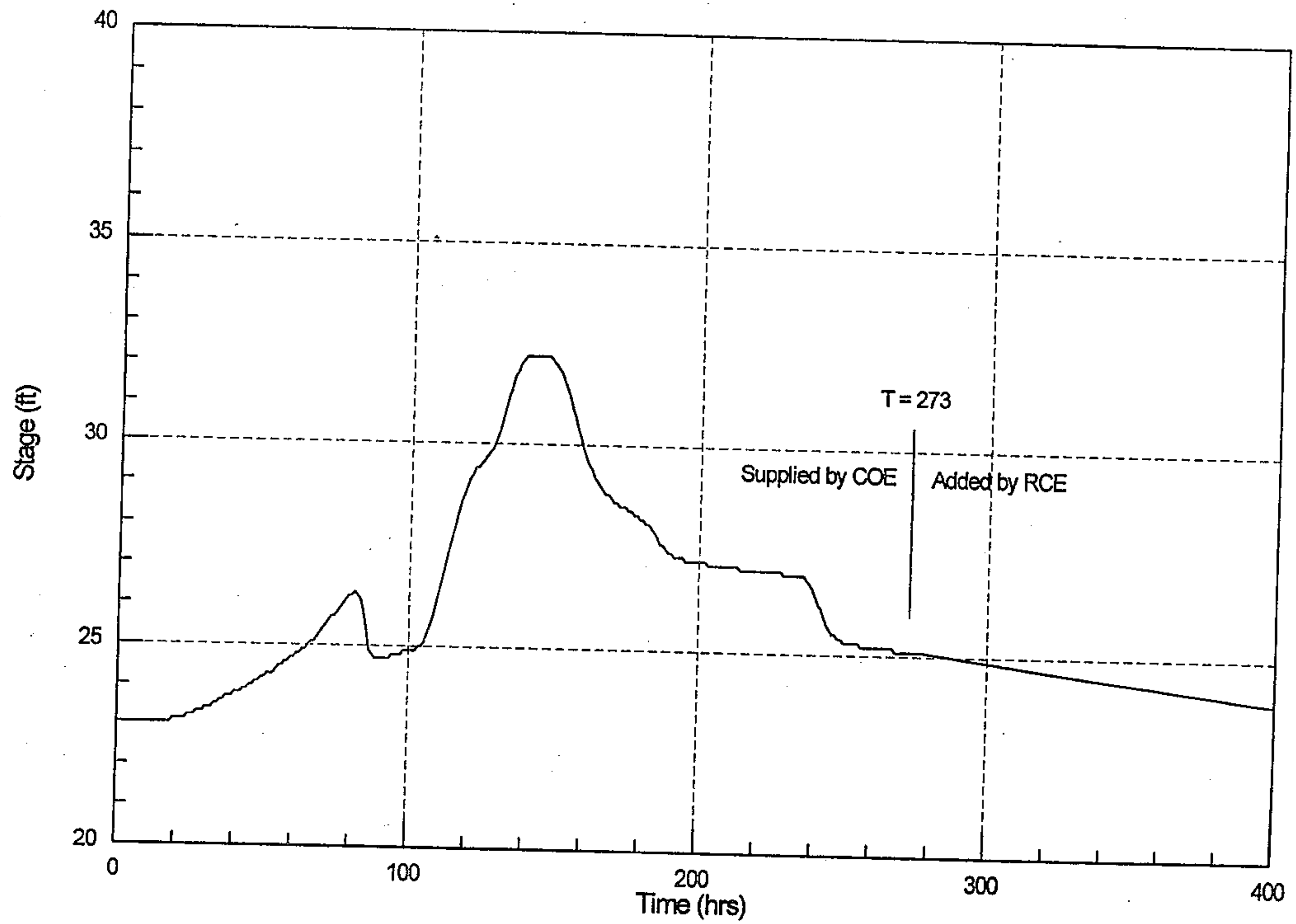


Figure 4.11. Stage hydrograph at the mouth of the American River, 100-yr event Design Scenario 3 (from COE simulations).

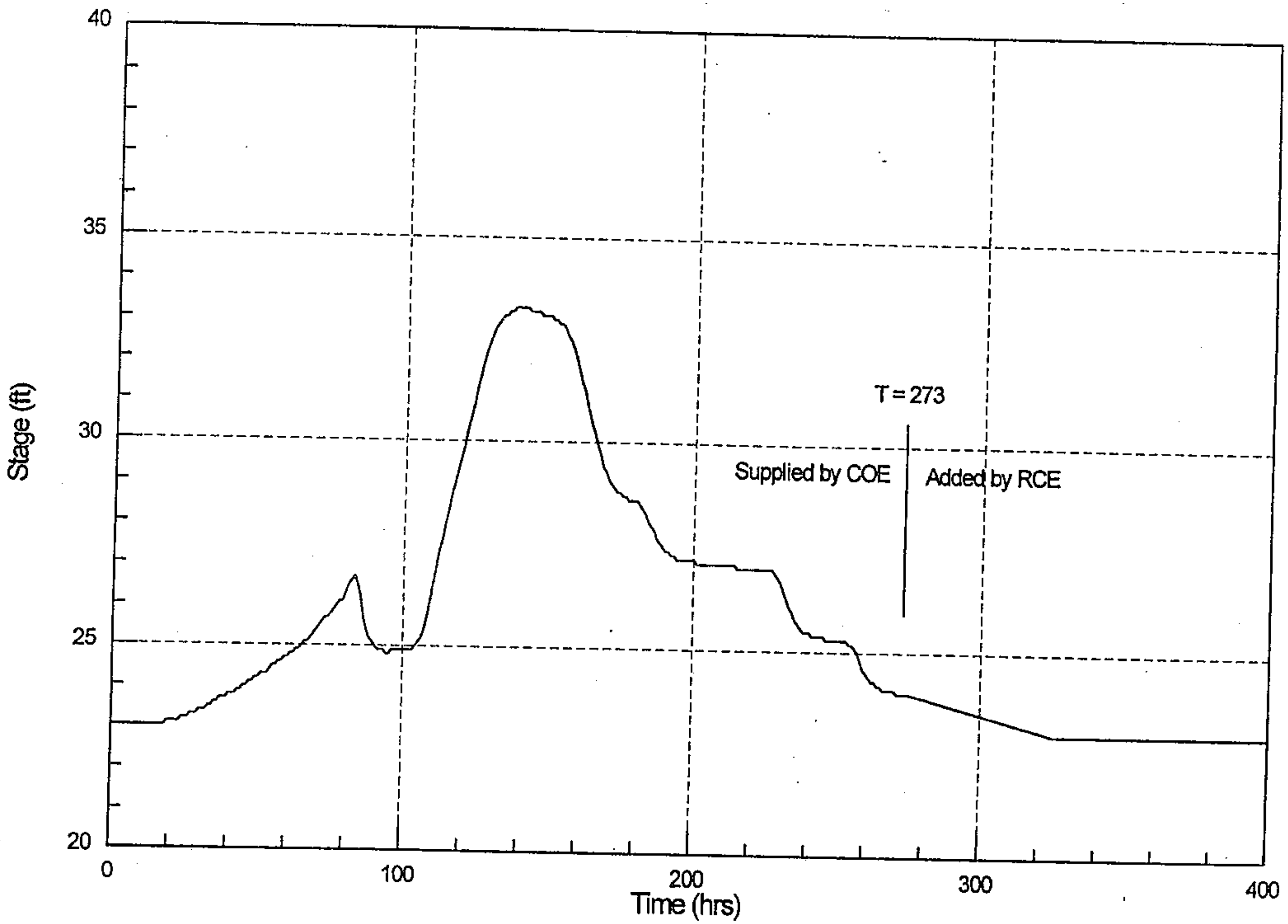


Figure 4.12. Stage hydrograph at the mouth of the American River, 100-yr event Design Scenario 4 (from COE simulations).

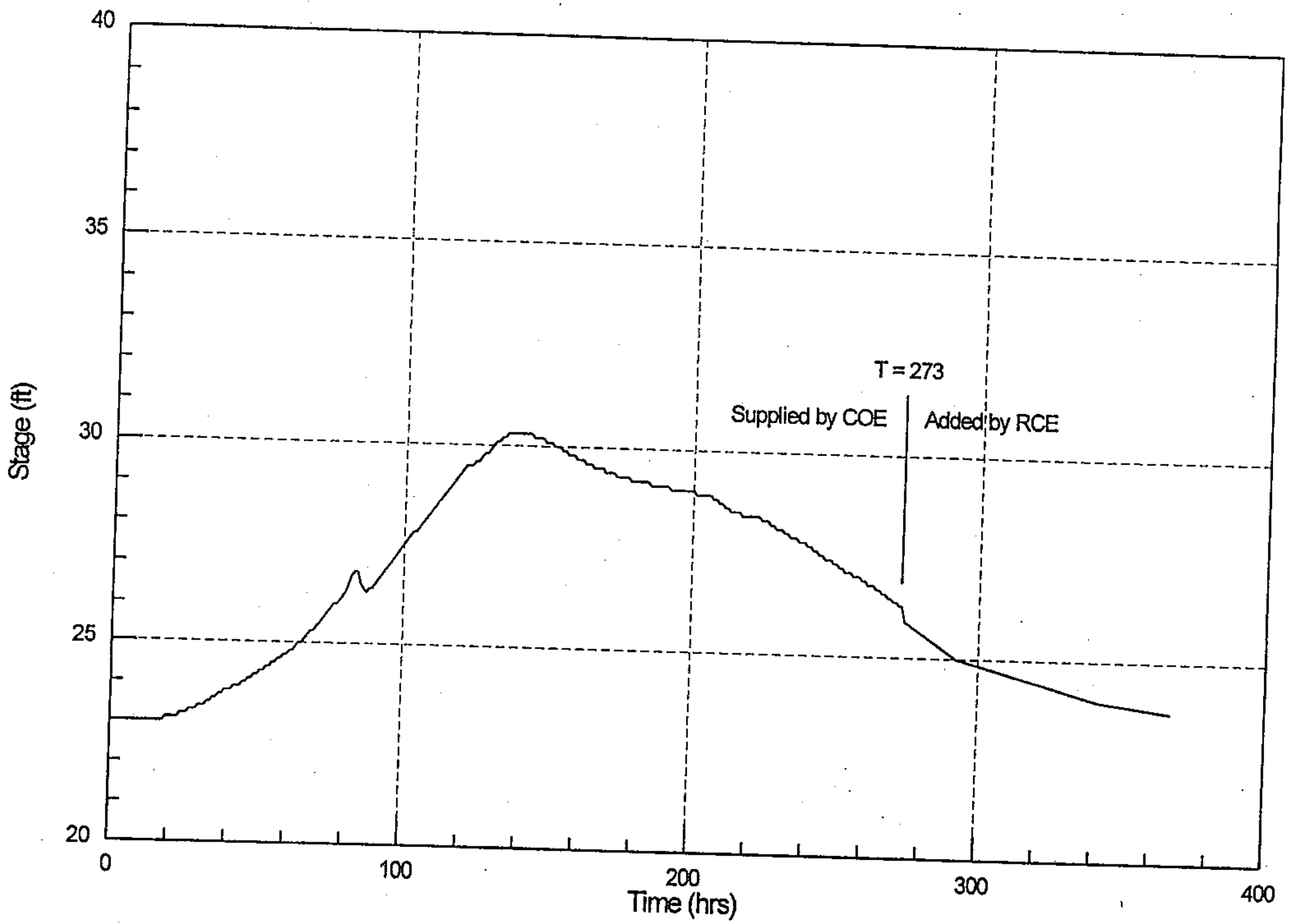


Figure 4.13. Stage hydrograph at the mouth of the American River, 100-yr event Design Scenario 5 (from COE simulations).

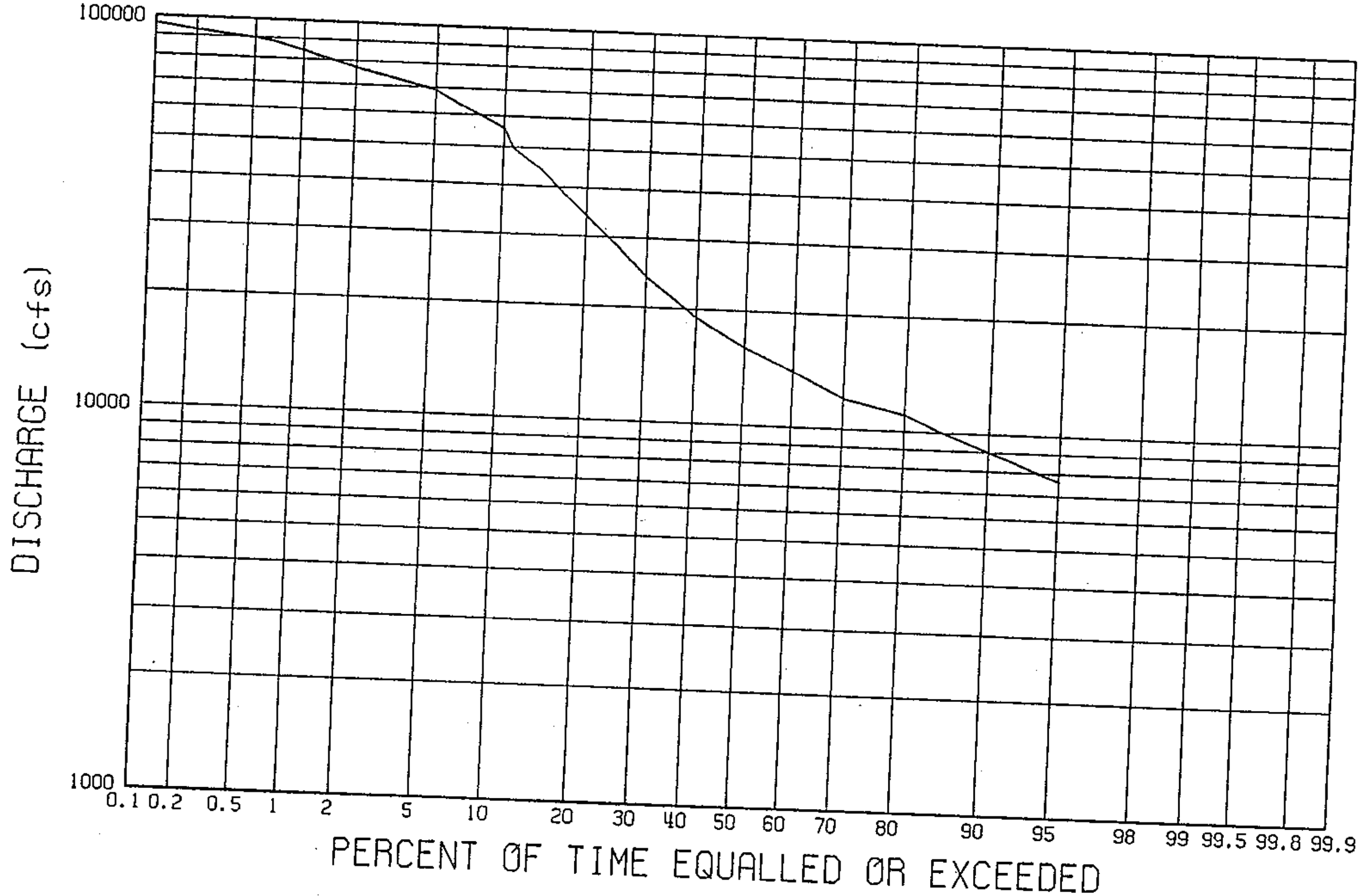


Figure 4.14. Sacramento River at I-Street flow duration curve (data supplied by the Sacramento District).

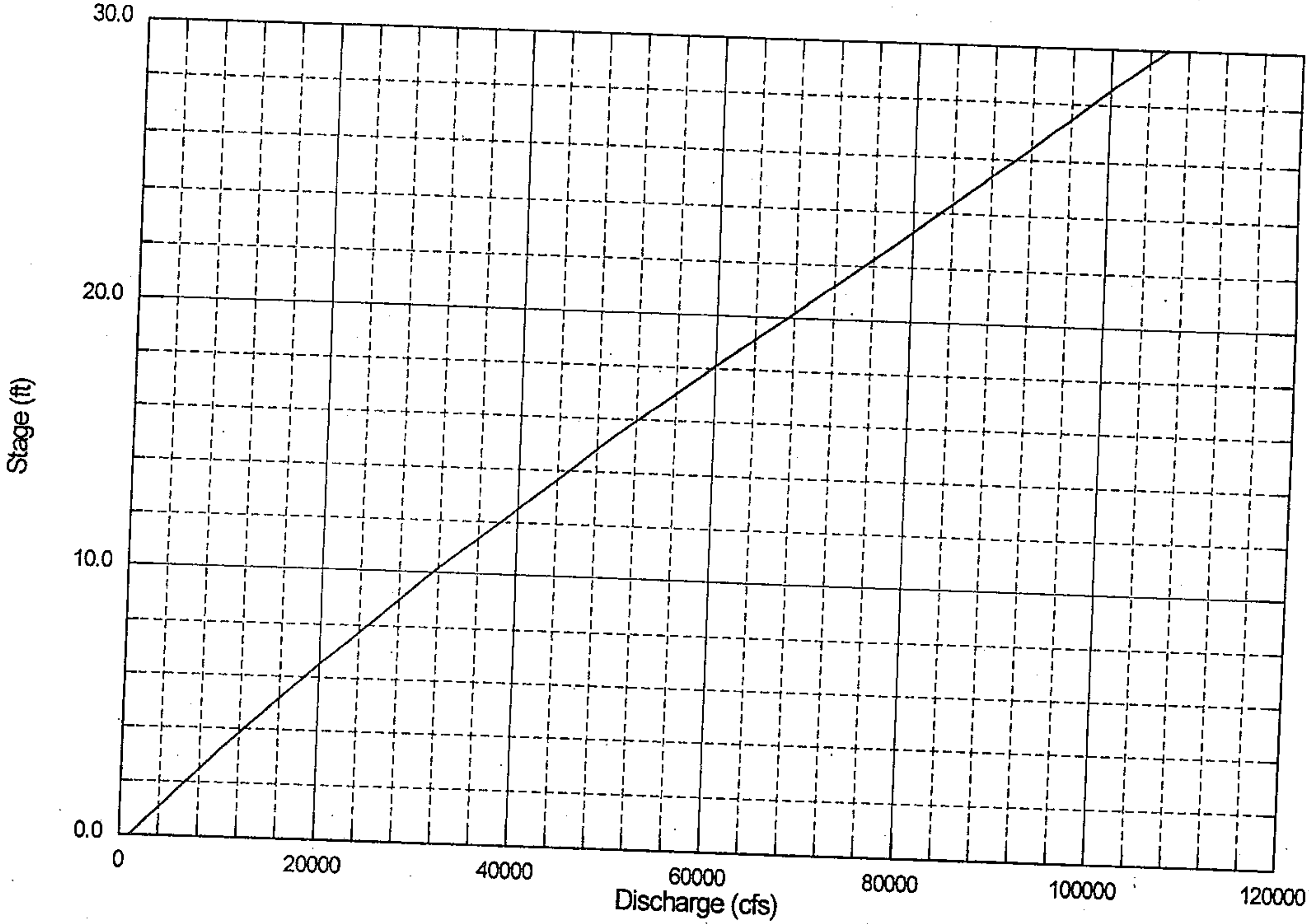


Figure 4.15. Sacramento River at I-Street stage-discharge relationship (data supplied by the Sacramento District).

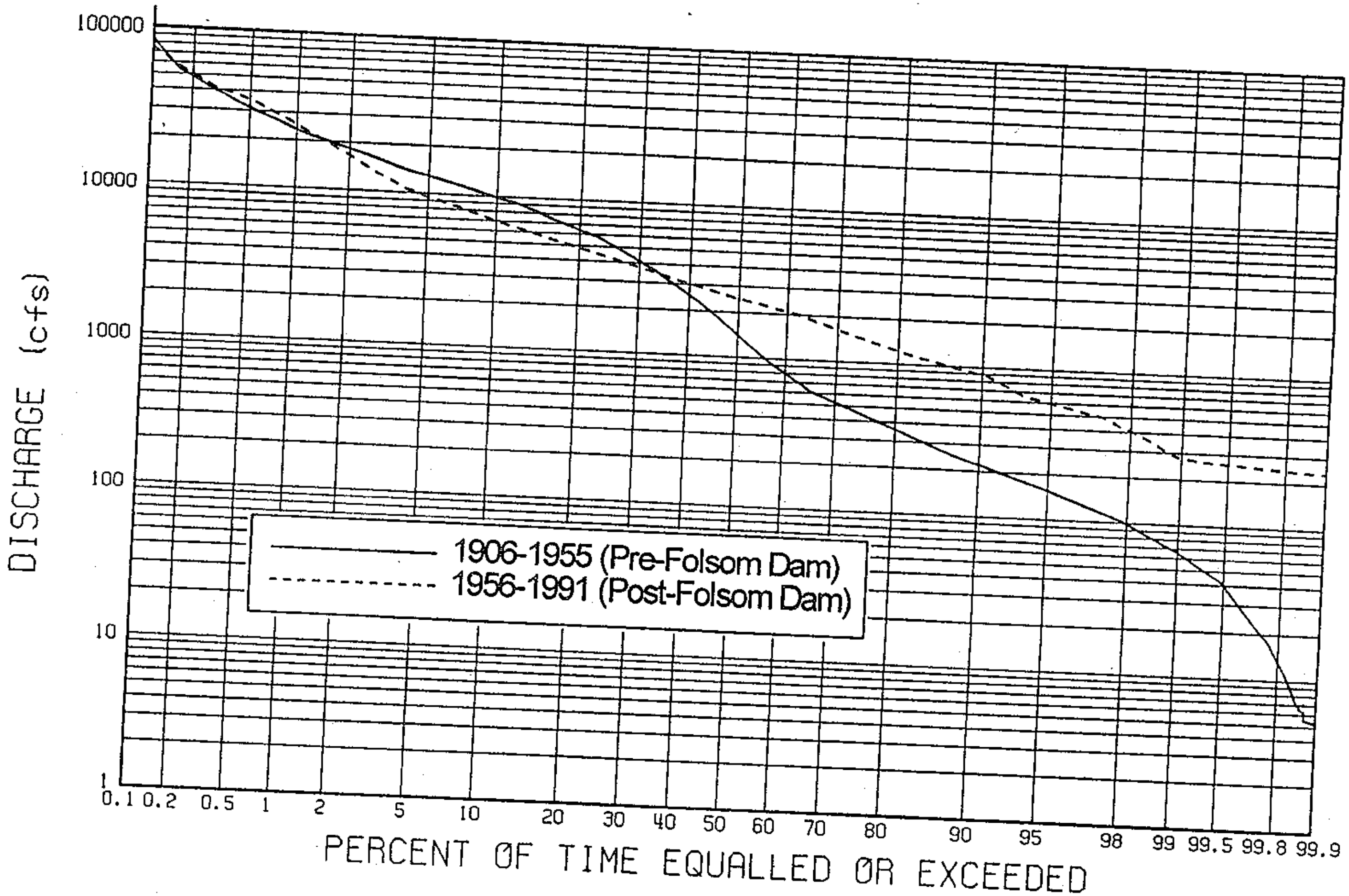


Figure 4.16. American River at Fair Oaks flow duration curves.

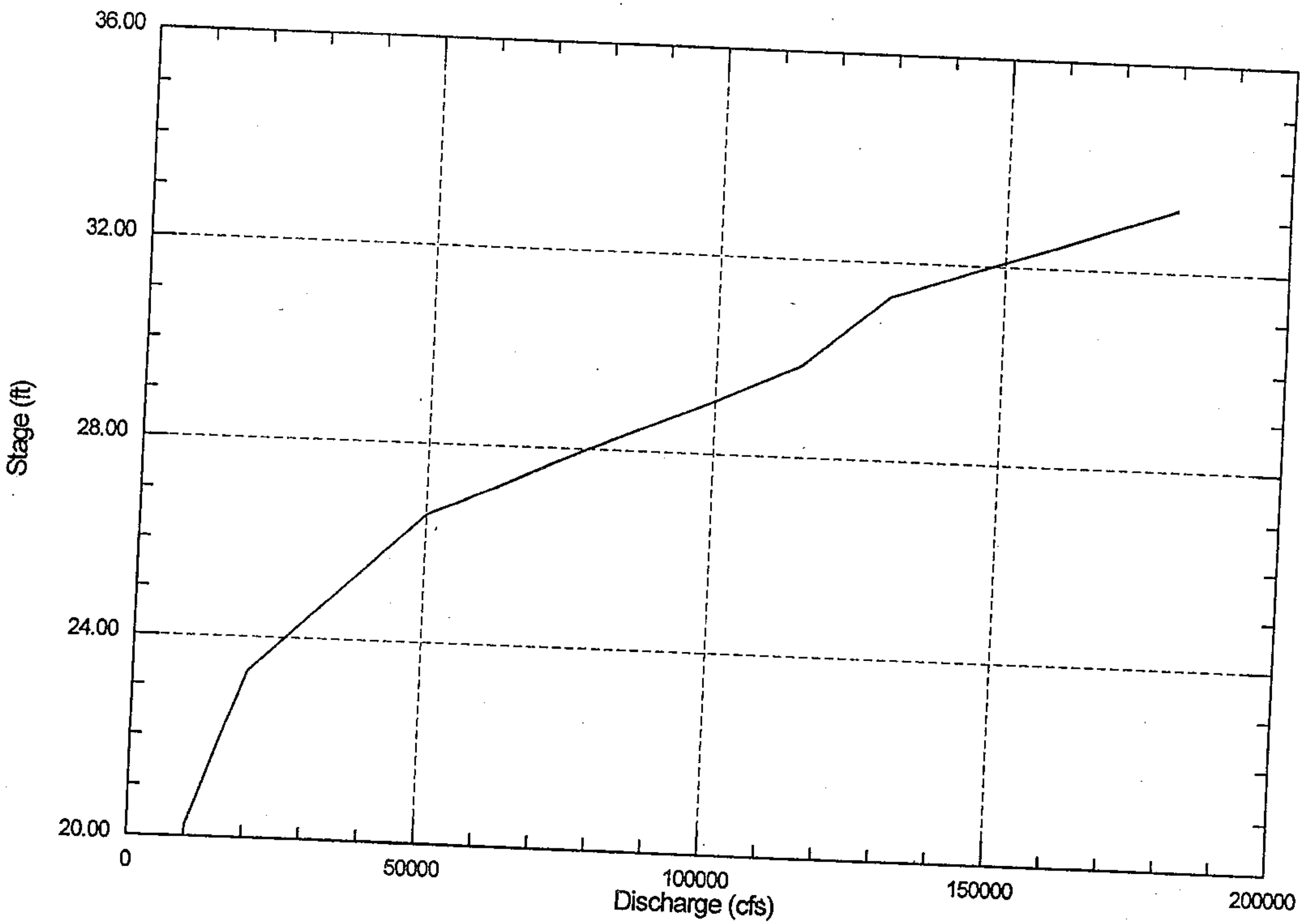


Figure 4.17. Single-valued stage-discharge relationship for the mouth of the American River, developed assuming equal flow probabilities on the American and Sacramento Rivers.

The next step in bracketing the range of stage-discharge conditions at the mouth of the American River was to examine stage-discharge traces developed from the Sacramento District routings for the 100-year event. **Figure 4.18** shows the stage-discharge trace plots developed for the five design scenarios. The plot also shows two single-valued, stage-discharge relationships that bracket most of the individual stage traces. The two stage-discharge relationships form a single relationship below a discharge of 28,500 cfs. This portion of the relationship is the same as the single valued relationship developed using concurrent flows with equal exceedence frequencies. The two single valued relationships were used in the subsequent analyses to bracket the range of likely conditions at the mouth.

4.2. Hydraulics

A detailed hydraulic analysis of the study reach was carried out to provide hydraulic information for use in the sediment transport and channel stability analyses. Hydraulic conditions were modeled using the HEC-2 water surface profile program. Output from the HEC-2 model was used to evaluate channel capacity, bed shear stress, work done on the channel banks, and local scour at bridge crossings. Geometric and hydraulic roughness data for the HEC-2 model were also used to form the basic input for the hydraulic portion of the HEC-6 sediment transport model (COE 1991) discussed in a later section of this chapter.

4.2.1. Data Development

A detailed HEC-2 input file covering the study reach from the confluence with the Sacramento River (RM 0.0) to Nimbus Dam (RM 23.1) was provided by the COE. The original HEC-2 input file was obtained from the 1992 Flood Insurance Study (FIS) for the American River below Nimbus Dam performed by the COE. This file was based on cross sections surveyed in 1987. The COE modified the FIS input file to make it compatible with the current PC version of the HEC-2 program and to account for new bridge piers at Sunrise Boulevard. The FIS input file used NH cards with high roughness values to account for ineffective flow areas. Since the current PC version of HEC-2 treats the channel roughness subdivided by NH records differently than the version used in the FIS study, the original input file produced unsatisfactory result. This problem was corrected by the COE by blocking out ineffective flow areas defined by high roughness with coordinate data entered on X4 records. Further changes were made to the FIS input file by the COE to make the results more useful in the channel stability analyses. These changes included moving the bank stations from the top of levees to the topographic top of banks and replacing roughness values specified on NH records with equivalent roughness values specified on NC records. The COE adjusted roughness values to calibrate the model using water surface elevations from the original FIS model for the peak flow from the 1986 event (upstream peak flow set equal to 130,000 cfs).

Additional changes were made to the model by RCE/Ayres to facilitate use of the results in the channel stability analyses. Changes included replacement of 7 cross sections with sections surveyed in the spring of 1993, which use constant roughness values over longer reaches and refine the location of some of the bank stations, and adding encroachments to certain overbank areas to account for ineffective flow areas and to produce reasonable changes in overbank flow among cross sections. **Table 4.3** summarizes the locations of

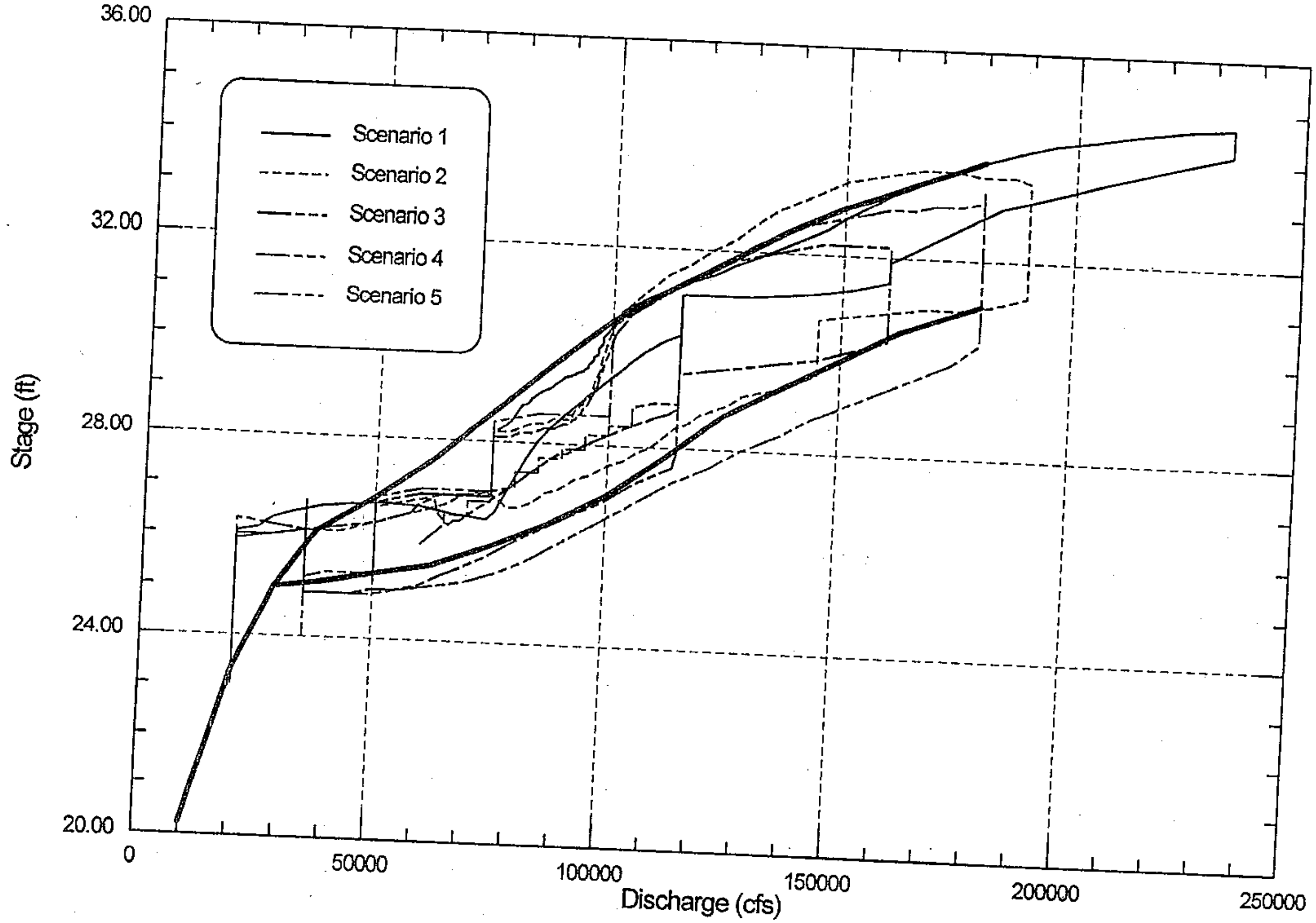


Figure 4.18. Stage-discharge traces at the mouth of the American River from COE 100-yr event Design Scenario simulations showing lower and upper bound single-valued stage-discharge relationships.

Table 4.3. Surveyed Cross Sections and Method Incorporated into HEC-2 Model.		
Location of 1993 Surveyed Cross Section (RM)	Nearest Cross Section in COE HEC-2 Model	Method of Incorporating Into HEC-2 Model
3.9	3.847	not used
5.4	5.497	not used
5.670	5.770	not used
6.012	6.012	direct replacement
6.528	6.528	direct replacement
9.205	13.465	merged with overbank geometry
13.465	13.592	merged with overbank geometry
15.200	15.200	merged with overbank geometry
19.729	19.729	merged with overbank geometry

the new cross sections used in the model. Cross sections in the levee reach (downstream of approximately RM 13) were completely replaced by the new cross sections since the new sections were surveyed from levee to levee. Surveyed cross sections upstream of RM 13 extended over only a portion of the floodplain. These cross sections were merged with the overbank portion of the existing cross sections for use in the model.

The HEC-2 model received from the COE contained frequent changes in main channel and overbank roughness values. These roughness values were selected by the COE to calibrate the model to the water surface profile predicted by the original FIS model for the peak flow during the 1986 event (flow at upstream end of the reach of 130,00 cfs). For purposes of the channel stability and sediment transport calculations, RCE/Ayres modified the roughness values to reduce the variability along the reach with changes made to reflect observable changes in the general character of the main channel and overbanks. Constant roughness values were maintained over reach lengths of approximately 2 miles or greater in the RCE/Ayres model. Initial estimates for the roughness values were based on bed material characteristics, bank vegetation, and overbank characteristics determined from aerial photographs. Changes to the specified roughness values were made to improve calibration with the original FIS results and recorded high water marks (see discussion in next section.)

4.2.2. Calibration

Included with the HEC-2 input data received from the COE were target water surface elevations based on the original FIS results used to calibrate the COE version of the model. These target water surface elevations were used to verify the modified RCE/Ayres model and to make changes to roughness values that were considered physically justifiable. **Figure 4.19** shows computed differences among the RCE/Ayres model results, the COE model results, and the target water surface elevations. Also

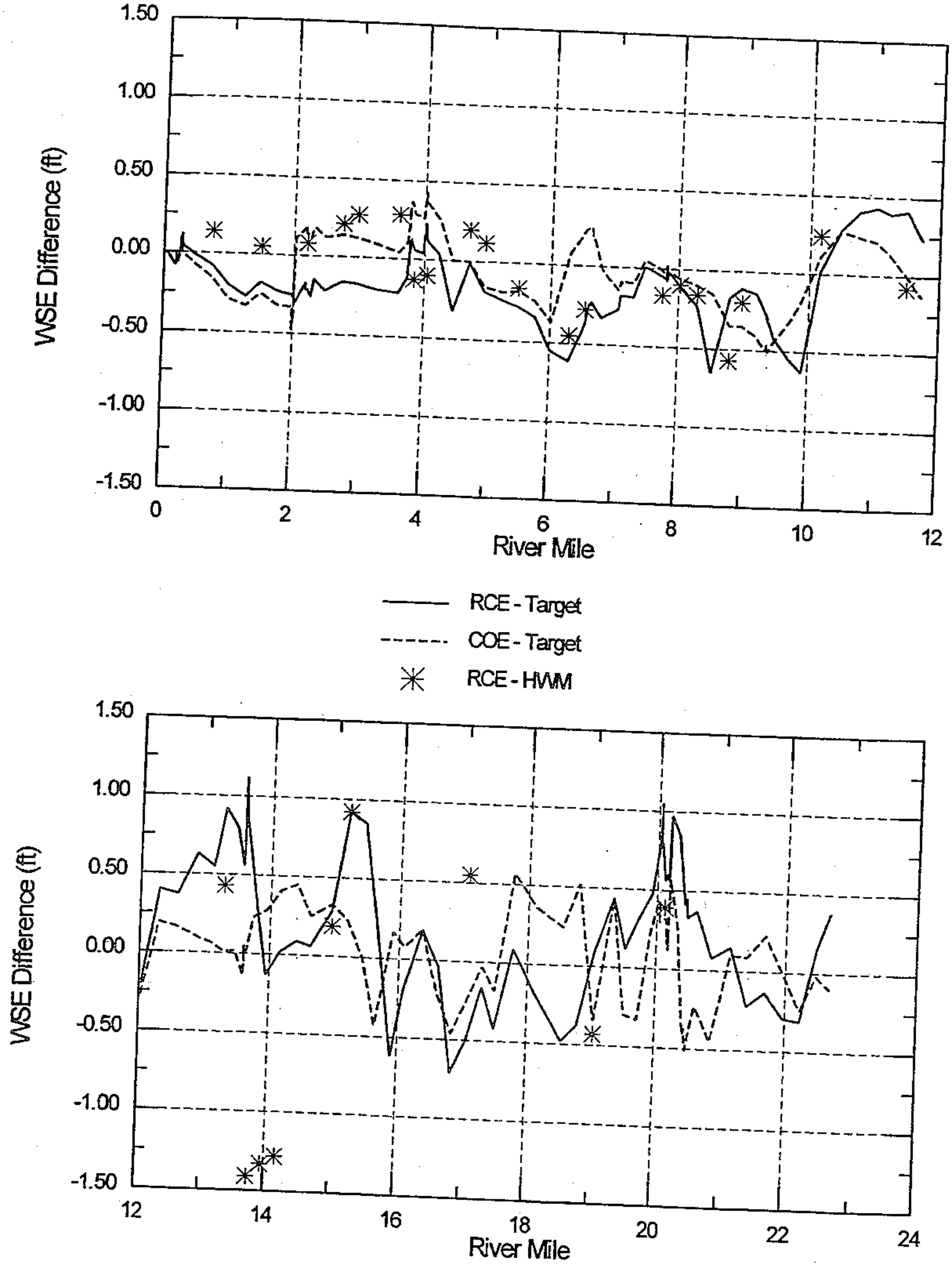


Figure 4.19. RCE/Ayres and COE water surface profile differences.

included are differences between the RCE/Ayres model and recorded high water marks from the 1986 event. The results show that the RCE/Ayres model is generally within 0.5 ft of both the target water surface elevations and recorded high water marks. The largest deviations with the target water surface elevations occur near RM 13, 15, and 20. The largest deviations with the recorded high water marks occur near RM 14 where computed water surface elevations are up to 1.4 ft low. Attempts to correct this difference by raising roughness values resulted in unreasonable overprediction of water surface elevations upstream. Given that the results are generally within 0.5 ft and reasonable roughness values were used, the results were judged to be sufficiently accurate for the purposes of this project. **Table 4.4** lists the final main channel and overbank roughness values used in the model.

Table 4.4. Summary of Main Channel and Overbend Manning's n Values used in the HEC-2 Model of the Lower American River.

River Mile	Manning's n		
	Left Overbank	Main Channel	Right Overbank
0.044	0.050	0.035	0.050
5.497	0.05	0.040	0.070
6.012	0.050	0.035	0.050
8.028	0.070	0.035	0.050
9.344	0.050	0.035	0.050
9.396	0.050	0.035	0.060
10.165	0.050	0.040	0.070
11.499	0.050	0.040	0.060
13.573	0.070	0.040	0.060
13.616	0.070	0.030	0.060
14.963	0.050	0.040	0.050

To utilize the model over a range of discharges, assumptions concerning starting water surface elevations, modeling of flow split between the main channel and the NEMDC, and flow breakouts between Nimbus Dam and the levee reach were necessary. Starting water surface elevations were taken from the single-valued, stage-discharge relationships developed for the mouth of the American River. The NEMDC enters the American River floodplain at RM 1.9 and parallels the main channel along the north levee to the confluence with the Sacramento River. For flows less than bankfull, the two channels are completely separate. During significant inundation of the floodplain by the American River, the flows join at RM 1.9. The Sacramento District HEC-2 model was developed for flows with significant floodplain inundation and contained flow input from the NEMDC at RM 1.9. It was therefore necessary to account for separation of the two channels at lower flows in the modeling. This was accomplished by first observing that the bankfull discharge is approximately 50,000 cfs in this reach. For flows less than 50,000 cfs, the two channels were assumed to be completely separate and modeling was restricted to the main American River channel through the use of a 10 in field 1 on X3 records. No tributary input from the NEMDC was included. For flows greater than 50,000 cfs, tributary flow from the NEMDC was added to the American River at RM 1.9 and flow across the entire floodplain including the NEMDC was considered.

Flow breakouts occur at several locations between the upstream limit of the levees (approximately RM 13) and Nimbus Dam. For flows less than approximately 180,000 cfs, all the flow that leaves the main channel re-enters upstream of the levees so that the entire flow is contained between the levees. For flows greater than approximately 180,000 cfs, some of the flow that leaves the main channel does not return. For modeling purposes, flows in the levee reach (downstream of RM 13.3) were limited to 180,000 cfs (190,000 cfs for Scenario 2); and flows upstream of the levees were limited to 200,000 cfs.

The COE HEC-2 input file contained 13 QT records representing flow breakouts and return flows upstream of the levees for an upstream discharge equal to 130,000 cfs. To extend the flow breakouts to other discharges, an earlier version of the HEC-2 input file provided by the Sacramento District was utilized, which contained upstream discharges equal to 115,000, 145,000, and 180,000 cfs. This file contained 12 QT records upstream of the levees with the locations not always the same as the file with the 130,000 cfs discharge. **Table 4.5** compares the QT records for the two files. To develop the appropriate QT records for other discharges, rating curves at points with equivalent QT records on both files were developed (RM 13.465, 16.594, and 18.532, **Figures 4.20 through 4.22**). These rating curves were extrapolated linearly downward to a zero flow breakout to determine the beginning discharge for flow breakout. Flows at other discharges were determined through interpolation below 180,000 cfs and extrapolation upward to 200,000 cfs. QT records at other locations were then developed using the locations specified on the 130,000 cfs profile and scaling the discharges specified for 130,000 cfs by the flows at the rating curve locations. **Table 4.6** gives the final discharges used at each location for a range of flows greater than or equal to 20,000 cfs (no breakouts occur less than 20,000 cfs).

4.2.3. Results

The HEC-2 model discussed in the previous section was run for discharges at the upstream end of the study reach ranging from 10,000 cfs to 180,000 cfs. Starting water surface elevations were set according to the stage-discharge relationship given in Figure 4.17. **Figure 4.23** shows bed and water surface elevation profiles for upstream discharges of 10,000, 20,000, 50,000, 100,000, and 180,000 cfs. Included on the plot are top of bank elevations set at the minimum of the left and right bank elevations for each cross section.

Figures 4.25 through 4.27 show main channel velocity, topwidth, and hydraulic depth (area/topwidth) profiles for discharges of 10,000, 20,000, 50,000, 100,000, and 180,000 cfs. The velocity and hydraulic depth profiles show a general trend of increasing velocities and decreasing hydraulic depths in the upstream direction which can be related to an increase in the channel slope. Sections of the river with large main channel top widths occur at locations where the river crosses wide gravel bars, often at bends. The variability in all the hydraulic parameters between cross sections reflects the highly variable nature of the flow along the study reach. As will be seen in the discussion of the sediment transport routings, this variability has a direct effect on the sediment transport characteristics of the reach.

Table 4.5. Discharges From QT Records Contained on HEC-2 Input Files Obtained From the Sacramento District.

River Mile	Discharge on QT Record (cfs)			
	$Q_{u/s}$ = 115,000 (cfs)*	$Q_{u/s}$ = 130,000 (cfs)**	$Q_{u/s}$ = 145,000 (cfs)*	$Q_{u/s}$ = 180,000 (cfs)*
0.044	129,400	144,500	161,000	196,300
1.950	115,000	130,000	145,000	180,000
13.465	109,300	121,000	132,600	158,000
14.963	115,000	126,000	145,000	180,000
15.200	----	130,000	----	----
15.440	110,200	----	134,60	160.000
15.649	112,100	----	138,800	168,000
16.079	113,100	----	140,800	172,000
16.345	115,000	----	145,000	177,000
16.594	110,200	123,000	134,600	160,000
16.832	105,400	----	129,500	150,000
17.054	115,000	118,000	145,000	180,000
17.290	113,100	120,000	139,800	165,000
17.498	----	126,000	----	----
18.127	----	130,000	----	----
18.532	113,100	127,000	139,800	172,000
18.515	----	125,000	----	----
19.729	----	127,000	----	----
19.920	115,000	128,000	145,000	180,000
20.169	----	130,000	----	----

* Early version of Sacramento District HEC-2 file.
** Latest version of Sacramento District HEC-2 file.

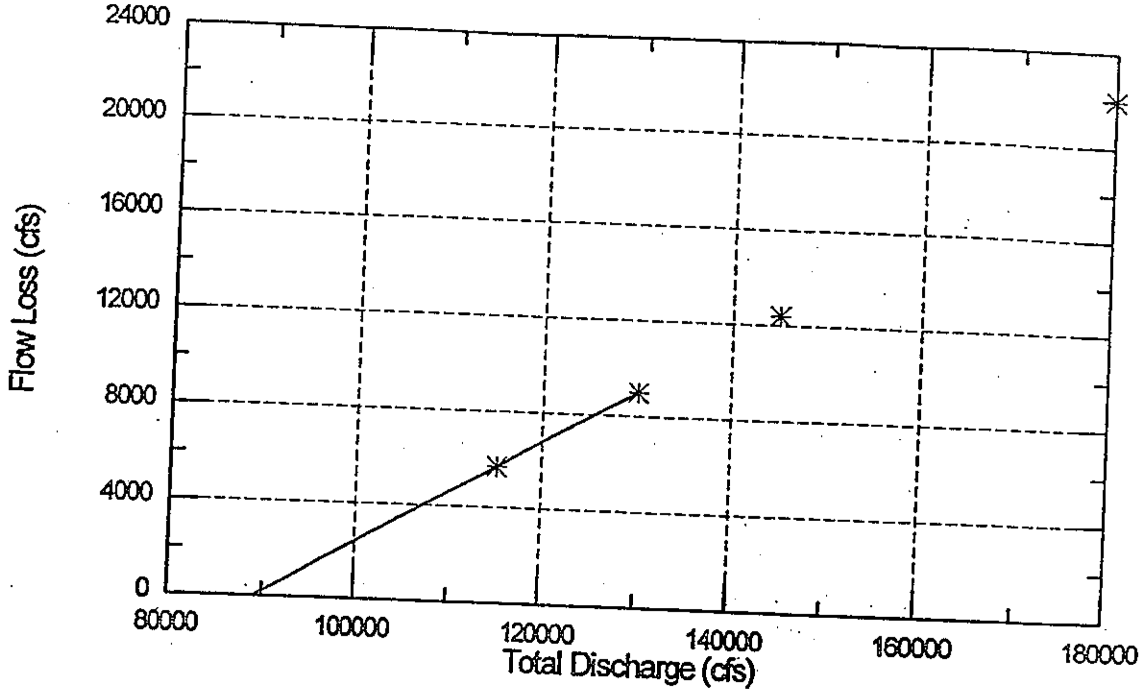


Figure 4.20. Split-flow relationship for RM 13.465.

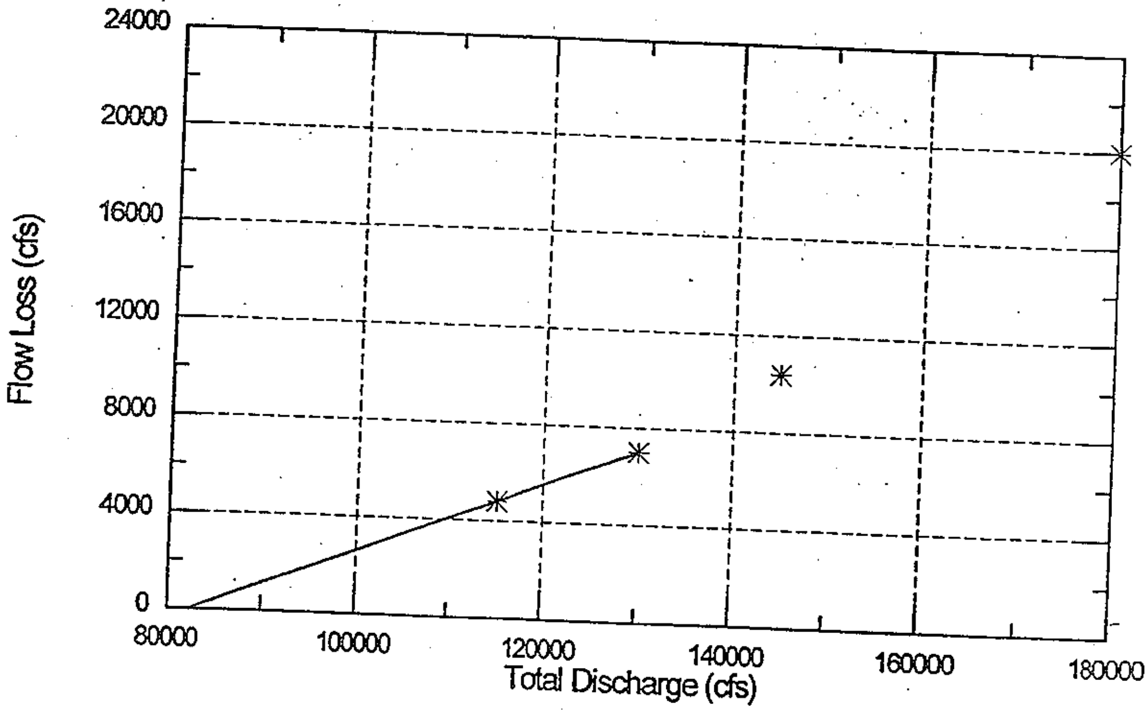


Figure 4.21. Split-flow relationship for RM 16.594.

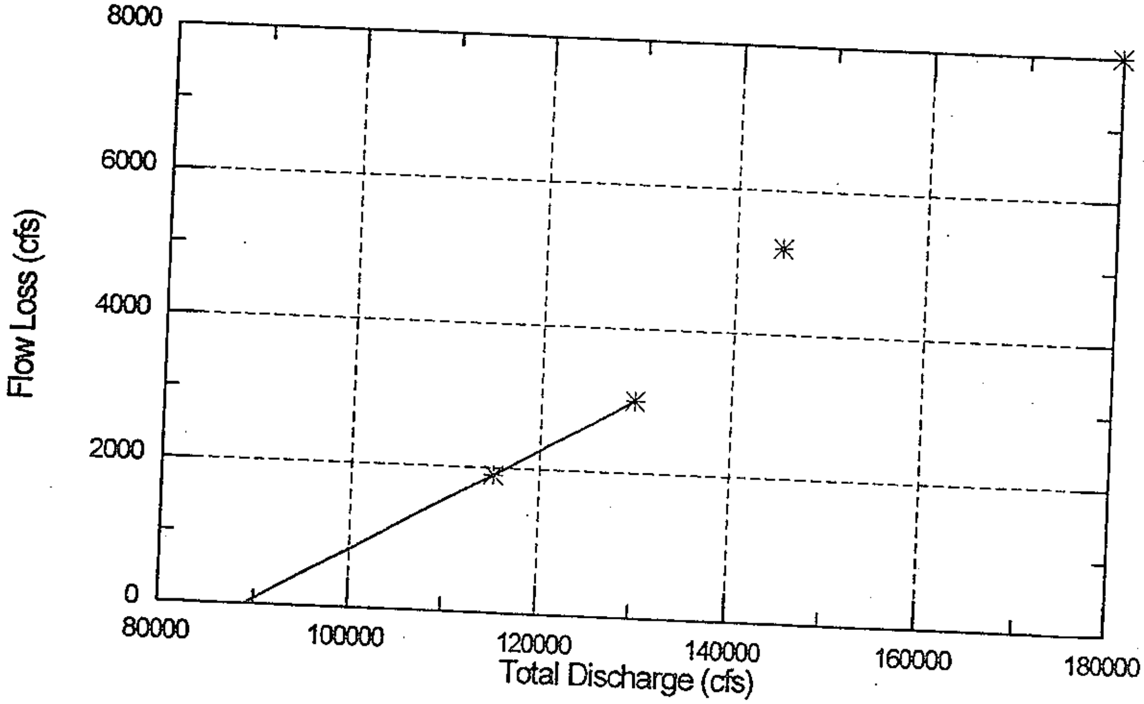


Figure 4.22. Split-flow relationship for RM 18.532.

Table 4.6.	Discharges Used on QT Records for Flows Greater Than or Equal to 50,000 cfs (No Flow Changes Occur for Smaller Discharges).					
River Mile	Discharge on QT Record (cfs)					
	$Q_{u/s}$ = 50,000 (cfs)	$Q_{u/s}$ = 100,000 (cfs)	$Q_{u/s}$ = 115,000 (cfs)	$Q_{u/s}$ = 130,000 (cfs)	$Q_{u/s}$ = 145,000 (cfs)	$Q_{u/s}$ = 180,000 (cfs)
0.044	50,000	110,000	129,400	144,500	161,000	196,300
1.950	50,000	100,000	115,000	130,000	145,000	180,000
13.465	50,000	97,600	109,300	121,000	132,600	158,000
14.963	50,000	100,000	114,300	126,000	137,600	163,000
15.200	50,000	100,000	115,000	130,000	145,000	180,000
16.594	50,000	97,400	110,200	123,000	134,600	160,000
17.054	49,700	92,400	105,200	118,000	129,600	155,000
17.290	50,000	94,400	107,200	120,000	131,600	157,000
17.498	50,000	100,000	113,200	126,000	137,600	163,000
18.127	50,000	100,000	115,000	130,000	145,000	180,000
18.532	50,000	99,200	113,100	127,000	139,800	172,000
19.515	50,000	97,200	111,100	125,000	137,800	170,000
19.729	50,000	99,200	113,100	127,000	139,800	172,000
19.920	50,000	100,000	114,100	128,000	140,800	173,000
20.169	50,000	100,000	115,000	130,000	145,000	180,000

The plot shows that the bankfull discharge averages approximately 50,000 cfs from Rm 0 to RM 13 (approximately 100,000 cfs from RM 13 to RM 18 and approximately 50,000 cfs from RM 18 to RM 23). Figure 4.24 shows top of levee profiles for the left and right levees along with the computed water surface profiles for a discharges of 100,000 cfs and 180,000 cfs. Nearing the top of the right levee near RM 7, the plot shows that 180,000 cfs is contained within the levees. The levee profiles shown were developed from the cross sections in the Sacramento District HEC-2 file and should be viewed with caution as low spots may not be reflected by the cross sections. While these results provide a general indication of the capacity of the levee reach, a more detailed HEC-2 model would be required to fully address the adequacy of the levees for a given level of protection.

The effects of the downstream starting water surface elevation on hydraulic conditions in the study reach was explored. Differences obtained from the model at 180,000 cfs, which use the upper and lower stage-discharge curves, are given in Figure 4.18. Differences in water surface elevations and main channel velocities are plotted in **Figures 4.28 and 4.29**. The figures show that the effects of the starting water surface elevation extend upstream to approximately RM 16, although the difference between the two conditions diminishes to less than 0.5 ft in the water surface elevation and approximately 0.3 fps by about RM 6.

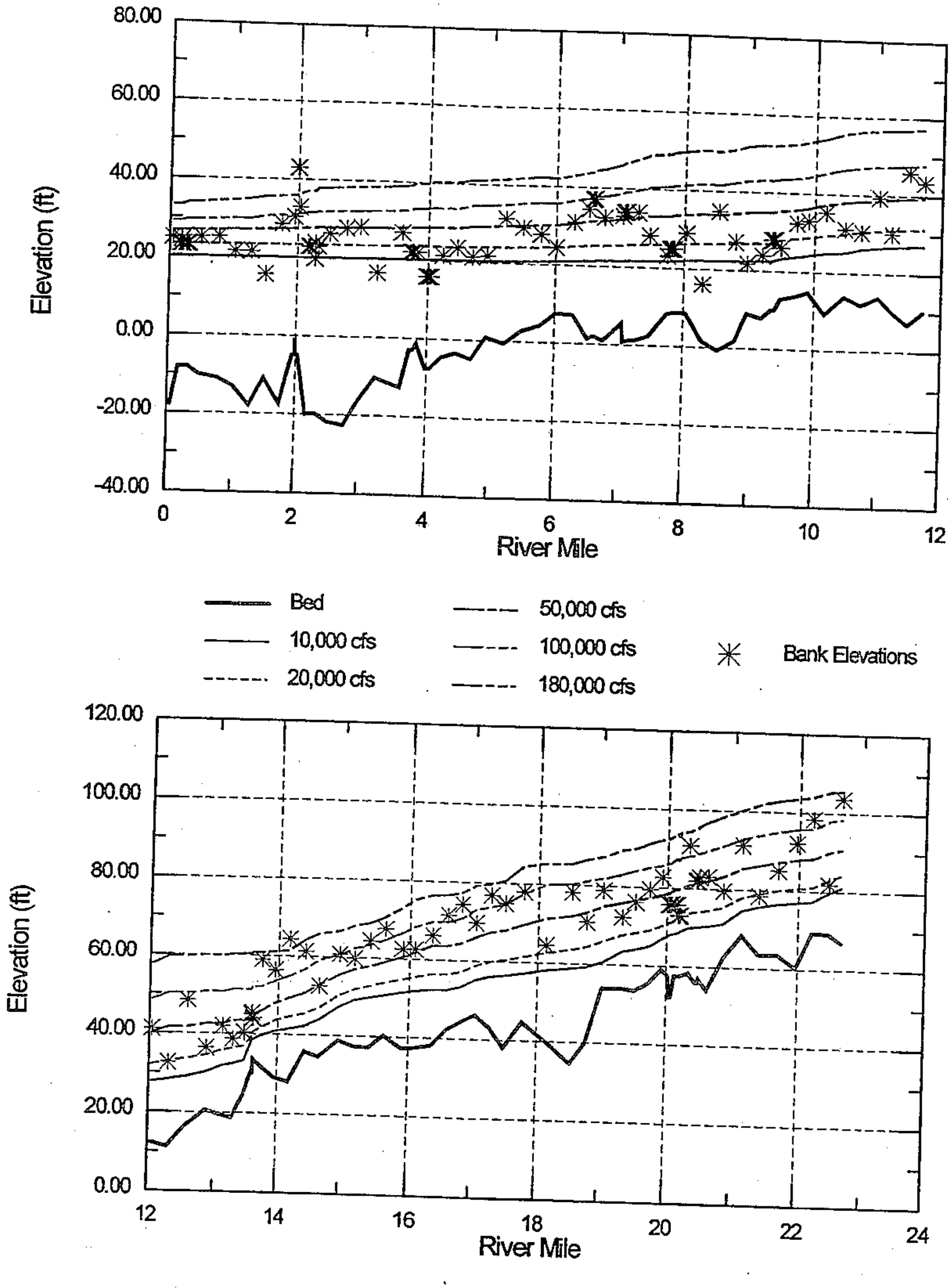


Figure 4.23. Water surface profiles computed using single-valued downstream stage-discharge relationship.

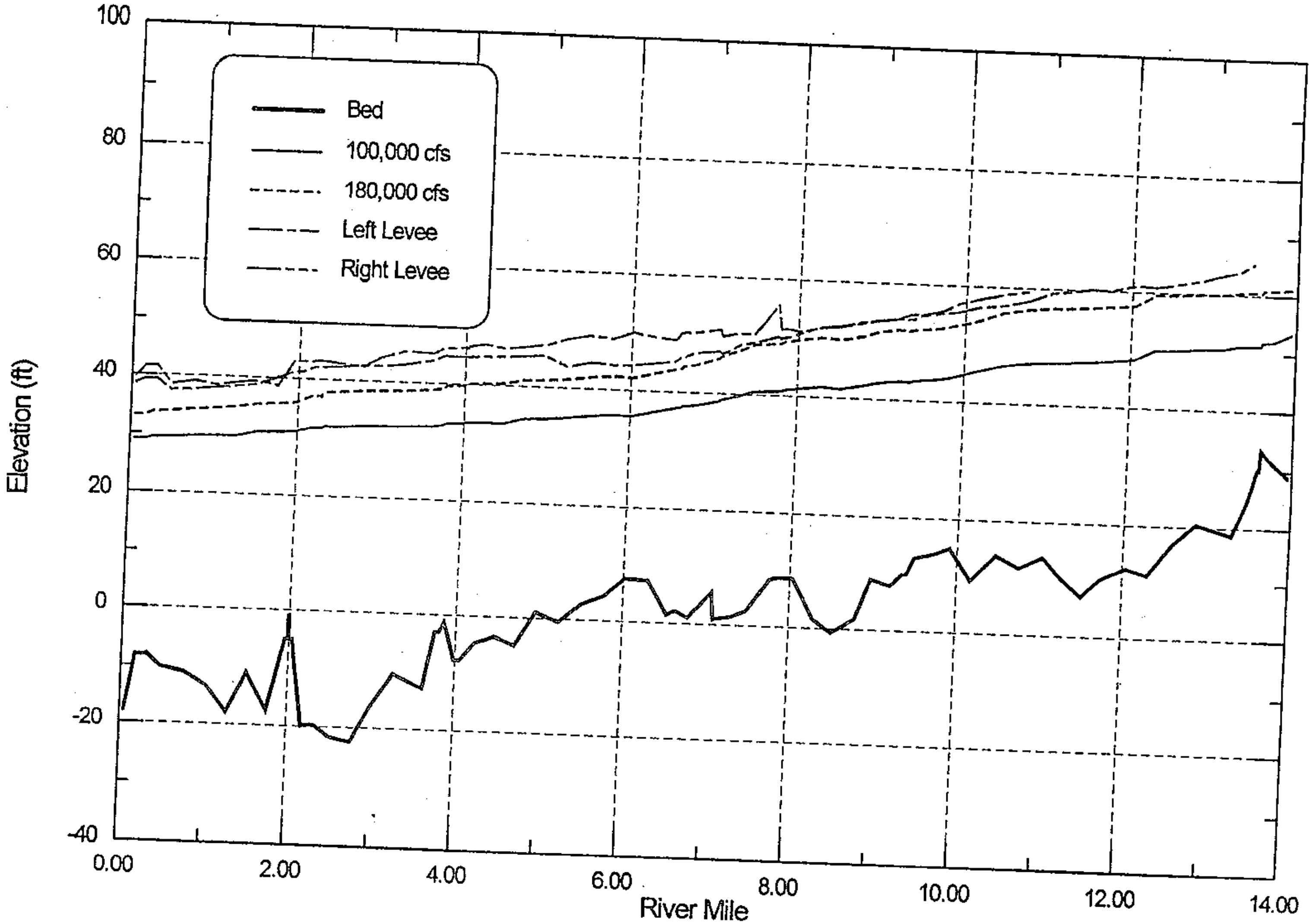


Figure 4.24. Levee profiles plotted with water surface profiles for 100,000 and 180,000 cfs.

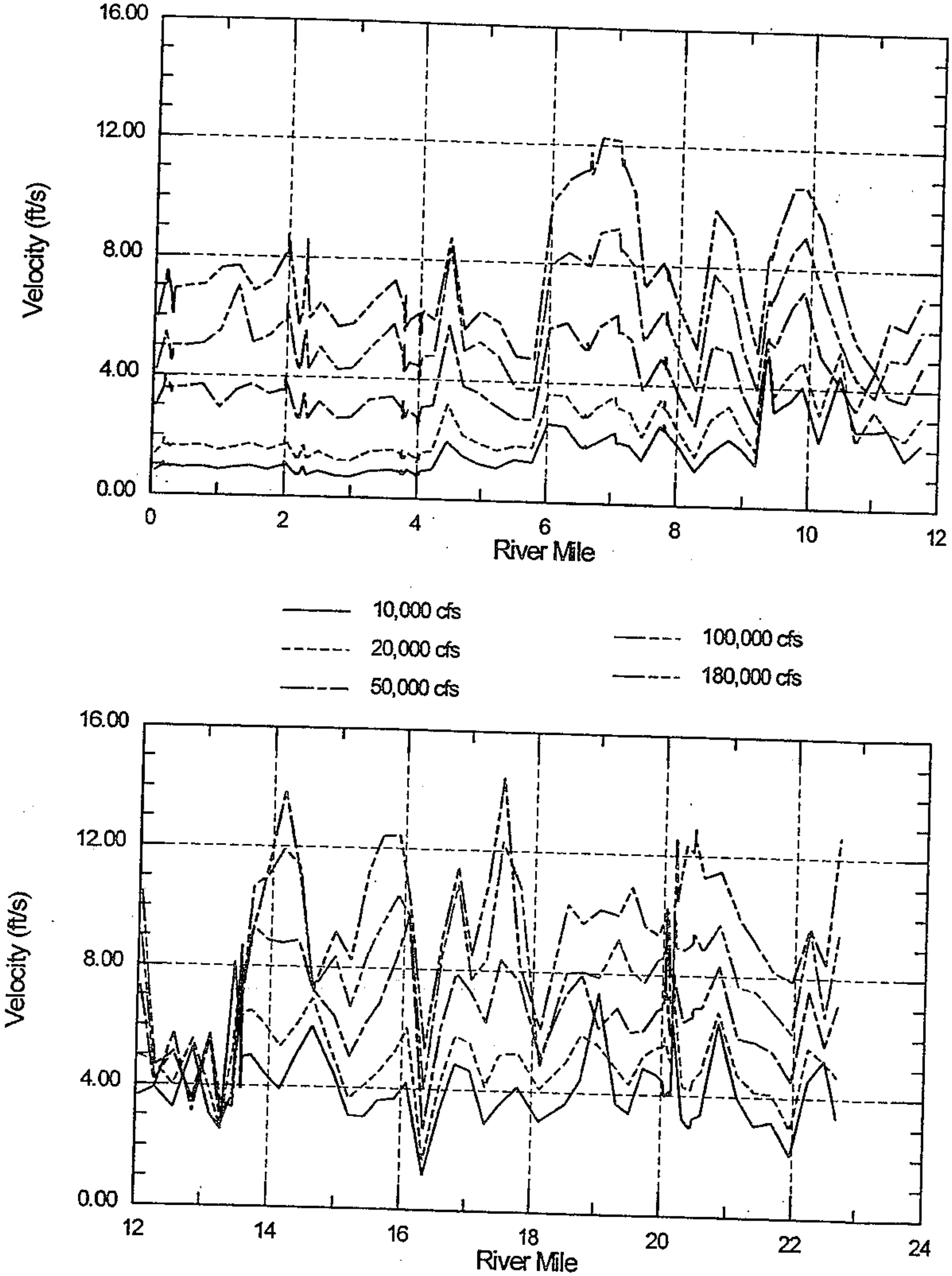


Figure 4.25. Main channel velocity profiles computed using single-valued, downstream stage-discharge relationship.

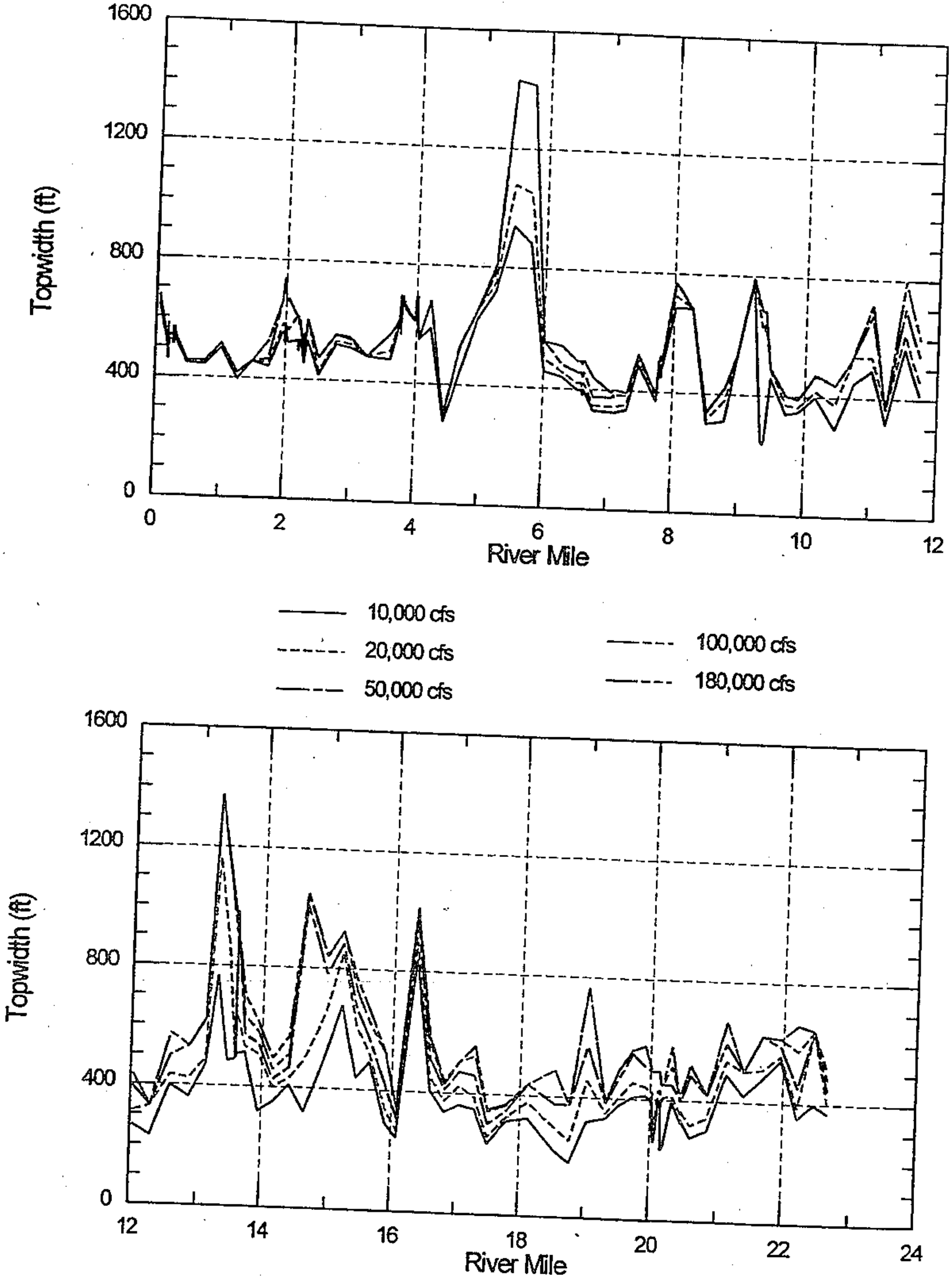


Figure 4.26. Main channel top width profiles computed using single-valued, downstream, stage-discharge relationship.

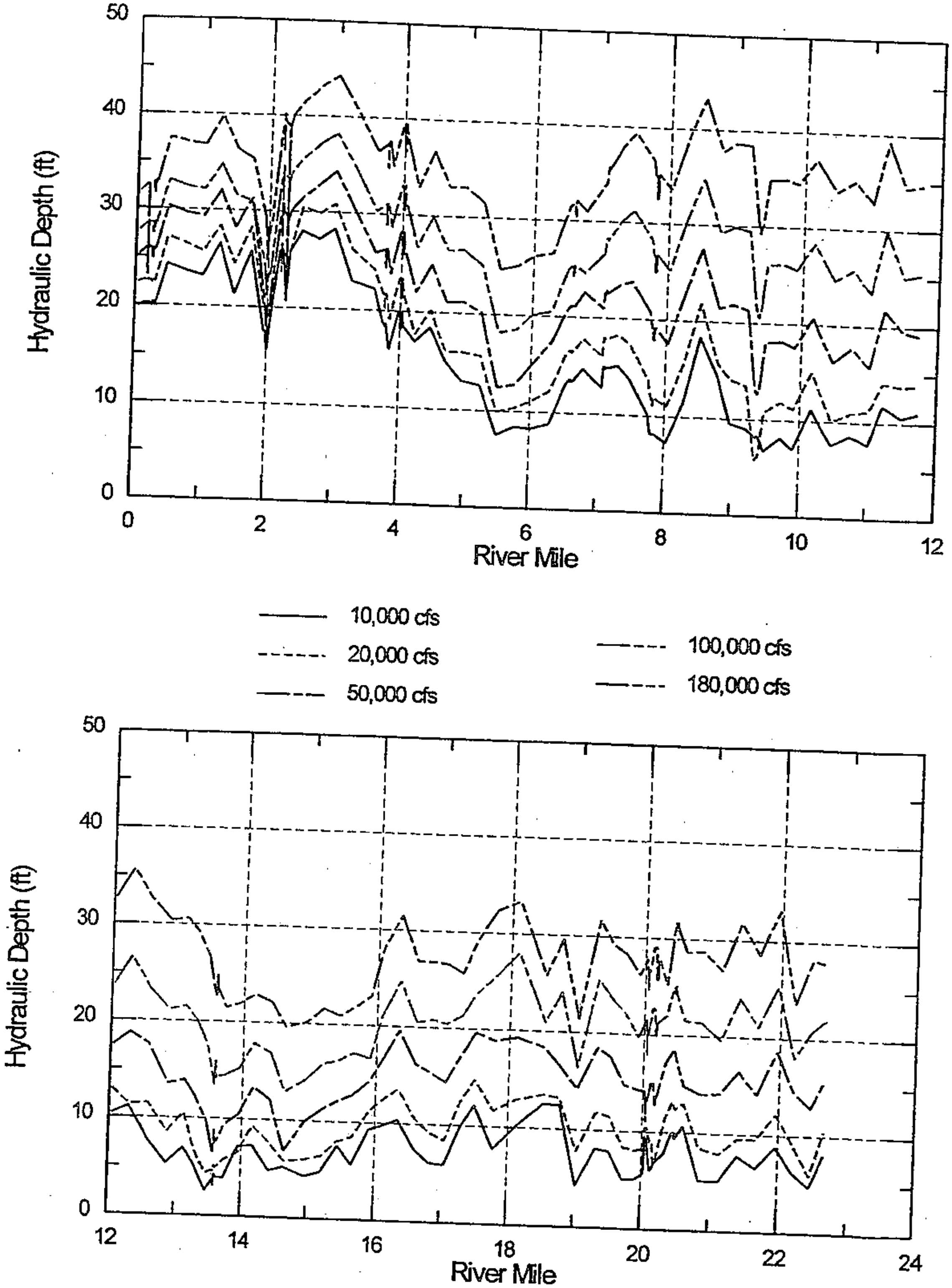


Figure 4.27. Hydraulic depth profiles computed using single-valued downstream, stage-discharge, relationship.

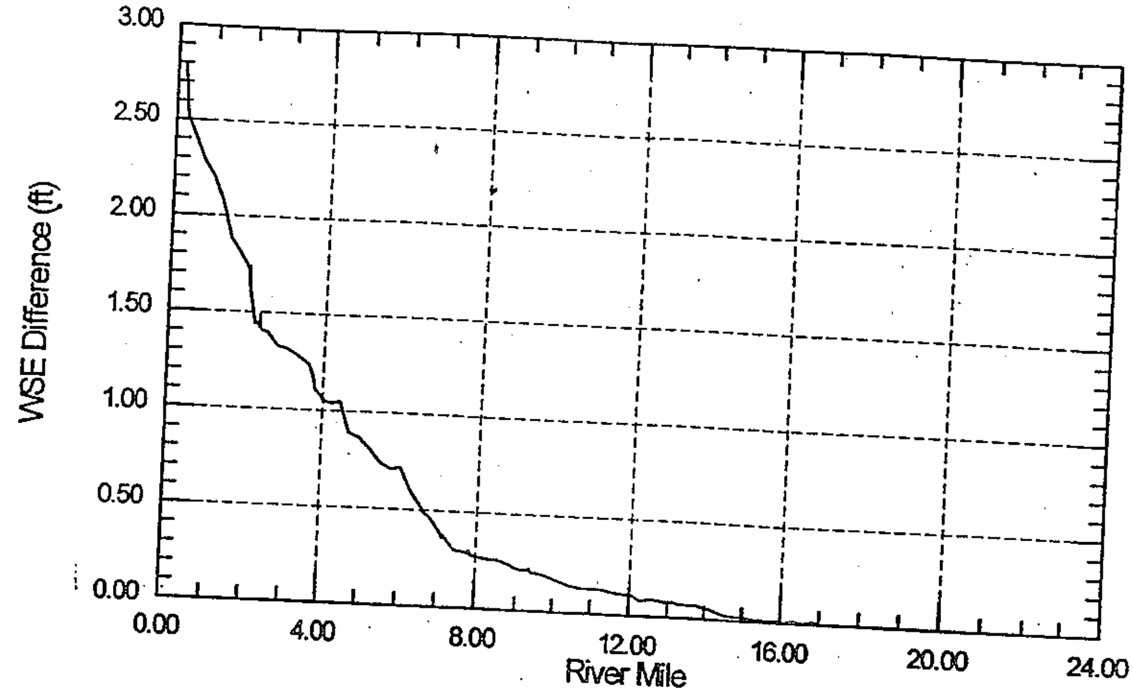


Figure 4.28. Water surface elevation differences at 180,000 cfs computed using the upper and lower bound, downstream, stage-discharge relationships.

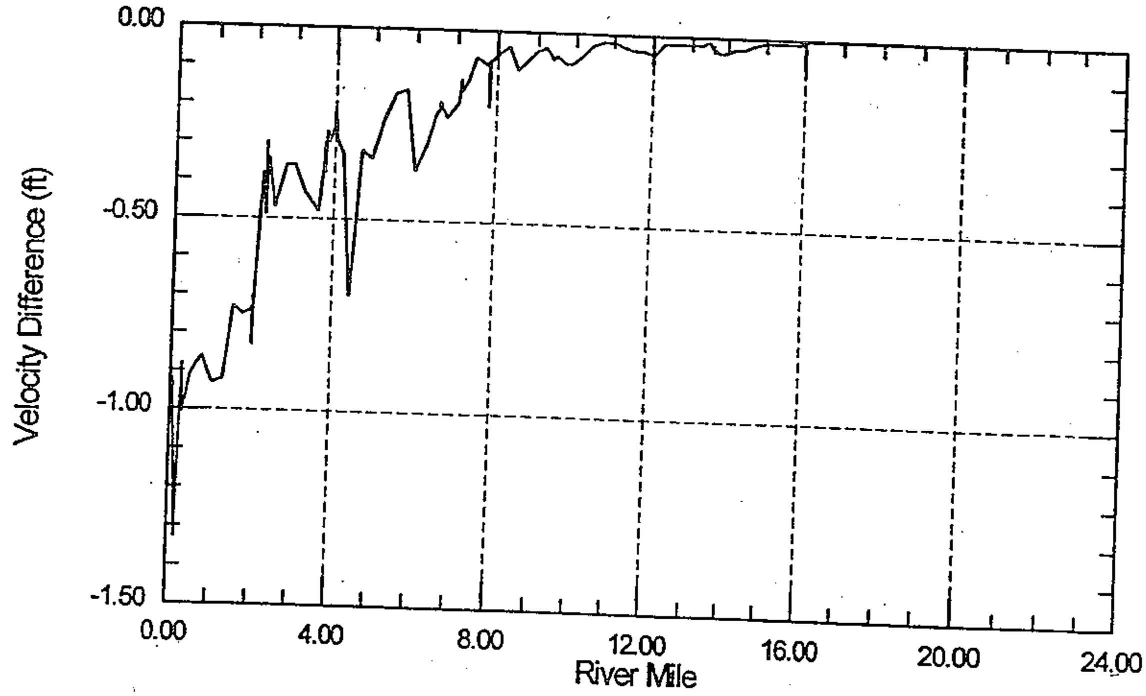


Figure 4.29. Main channel velocity differences at 180,000 cfs computed using the upper and lower bound, downstream, stage-discharge relationships.

4.3. Watershed Sediment Yield

Sediment delivered to the upstream end of the project reach from the American River watershed is significantly affected by Folsom and Nimbus Dams. Folsom Reservoir is approximately one million ac-ft facility that traps all the bed material load and a significant portion of the wash load brought in from upstream. Nimbus Dam, a power afterbay located just downstream of Folsom Dam, has a reservoir capacity of approximately 9,000 ac-ft. Because of the presence of these two dams, no bed material sized sediment is supplied to the project reach from upstream. A trap efficiency calculation was performed for Folsom Reservoir to determine the percentage of upstream wash load delivered to the project reach. Due to its small size and location just downstream from Folsom Dam, the trap efficiency of Nimbus Dam was neglected.

The trap efficiency for the wash load was evaluated using the procedures developed by Brune (1953) and Churchill (1948) in "Sedimentation Engineering" (ASCE 1975). Both procedures estimate the trap efficiency using average flow conditions and are thus applicable on an average annual basis. The results provided in **Table 4.7** indicate that from 90 to 100 percent of the wash load will be trapped. There is thus an insignificant quantity of sediment delivered to the project reach. Based on the wash load sediment yields derived in Chapter 3 for the MDE, the resulting average annual wash load yield to the Lower American reach is approximately 0.005 ac-ft/mi^2, which is based on the best estimate curves and ranges from a negligible yield using the lower limit curve to an upper limit of approximately 0.02 ac-ft/mi^2. For purposes of evaluating the stability of the lower American River and sediment yield to the Sacramento River, these result are considered to be negligible. If the trap efficiency of Folsom Reservoir is assumed to reduce to approximately 80 percent during the 100-year storm event, the best estimate wash load sediment yield to the study reach would be about 0.22 ac-ft/mi^2, which ranges from a lower bound of 0.04 ac-ft/mi^2 to an upper limit of 1.59 ac-ft/mi^2.

Table 4.7. Results of Trap Efficiency Calculations for Folsom Reservoir.

Method	Sediment Trapped (%)*
Brune (1953), lower envelope curve	90
Brune (1953), median curve	95
Brune (1953), upper envelope curve	100
Churchill (1948)	99

*Based on reservoir at flood control pool, 610,000 ac-ft of storage.

Potential source of sediment supply within the study reach include the NEMDC, the only significant tributary along the study reach, runoff from local storm drainage facilities, and bank erosion. The NEMDC enters the American River floodplain at RM 1.9, parallels the main American River channel, and flows directly into the Sacramento River. For this reason, sediment delivery from the NEMDC was not considered in the analysis. Runoff from local storm drains may deliver a small amount of sediment to the reach however, given the urbanized nature of most of the contributing area, the amount associated with source is also considered to be small.

As discussed in Chapter 2, some bank erosion is occurring within the reach under existing conditions. During high flows, this may be the most significant source of fine sediment to the study reach. Since no suspended sediment data are available with which to estimate the typical wash load concentrations, a quantitative estimate of wash load sediment yield derived from this source is not possible.

Considering the above observations, the wash load sediment yield from the Lower American River study reach is believed to be insignificantly small. This conclusion is supported by observation during large flow events where the water in the American River is quite clear (indicating that it is carrying little suspended sediment load).

4.4. Sediment Transport

A detailed sediment transport analysis of the study reach was performed to evaluate the vertical stability of the channel and to determine sediment yields to the Sacramento River under the various design scenarios. The sediment transport analysis involved the evaluation of watershed sediment yield, an incipient motion analysis of the channel bed material, detailed sediment routings, and sediment budget calculations. The results of the sediment transport analysis are presented in this chapter. An analysis of the lateral stability of the channel under the various design scenarios is presented in Section 4.5.

4.4.1. Incipient Motion Analysis

The bed of the American River along the study reach is primarily composed of gravel to cobble sized material. The resistance to motion of this coarse grained material is an important element in the vertical stability of the channel along the study reach. An incipient motion analysis was performed to determine the discharges required to mobilize the bed material sediments.

Figure 4.30 shows average bed material surface gradations developed from the sediment samples collected for this and previous projects. These curves were used in the incipient motion and subsequent sediment transport calculations. The surface gradation for the downstream approximately 2 mi of the study reach is believed to represent a relatively thin layer of sand deposited over the coarser grained channel bed during low flows. This material will be flushed during higher flows exposing the underlying coarser grained bed. For this reason, the average surface gradation developed for the next upstream reach (RM 2.00 to RM 7.94) was used in the calculations.

The incipient motion analysis was performed using Shield's relationship and grain shear stresses computed by using hydraulic results from the HEC-2 model. Incipient motion is defined as the point where the computed grain shear is equal to the critical shear of the bed sediments. The grain shear is given by:

$$\tau_{gs} = \gamma \ R' \ S_f \tag{4.1}$$

where τ_{gs} is the grain shear (lb/ft^2), γ is the unit weight of water (lb/ft^3), R' is the equivalent hydraulic radius associated with grain resistance, and S_f is the energy slope (ft/ft). The equivalent hydraulic radius can be evaluated through boundary layer theory and the semilogarithmic relation:

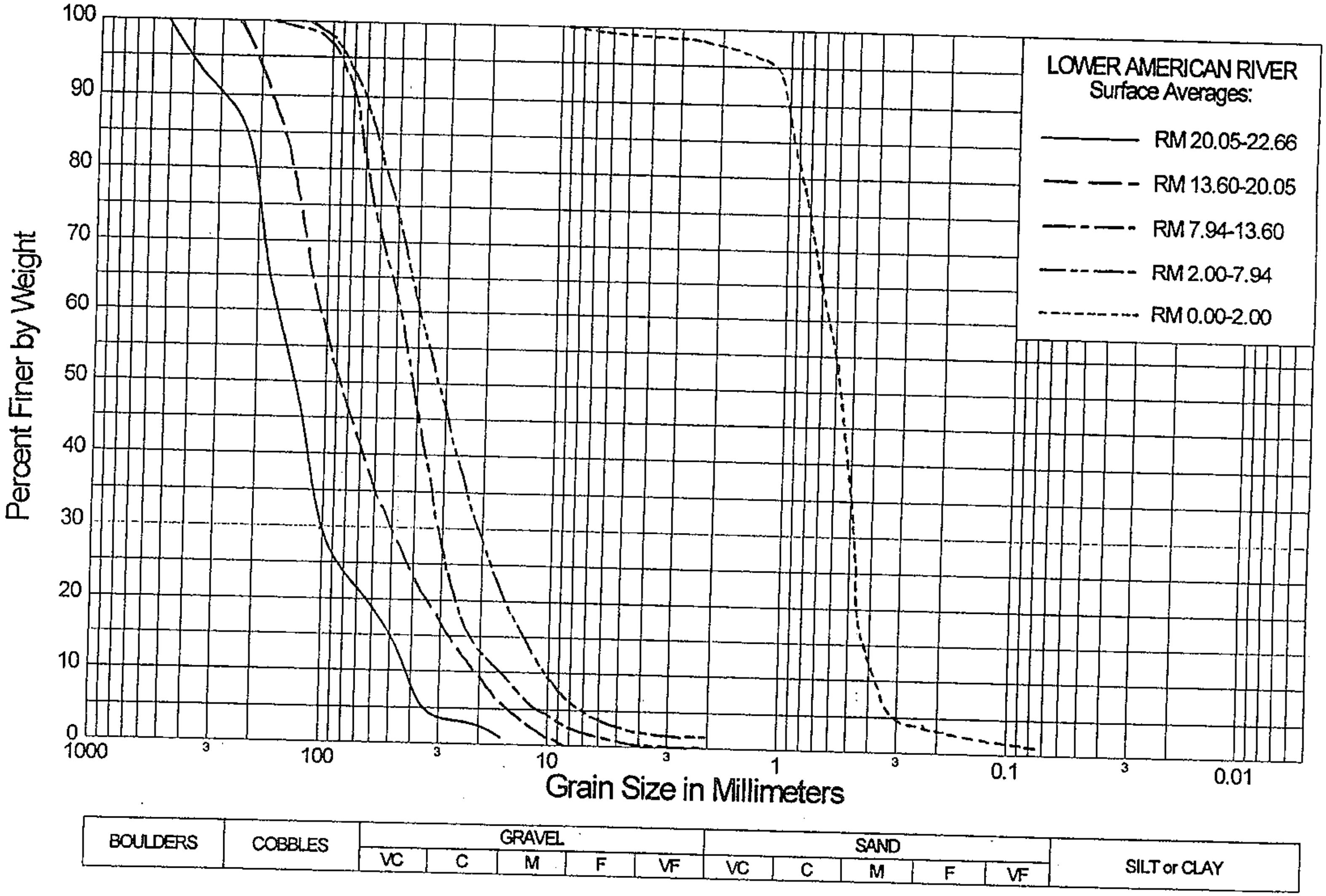


Figure 4.30. Average surface bed material sediment size distributions.

$$\sqrt{\frac{8}{f'}} = 6.25 + 5.75 \log\left(\frac{R'}{k_s}\right) \tag{4.2}$$

where f' is the Darcy-Weisbach friction factor associated with grain resistance and k_s is the characteristic size of the bed material (Keulegan 1938). The Darcy-Weisbach friction factor is related to the grain shear through the Darcy relationship:

$$\tau_{gs} = \frac{1}{8} f' \rho V^2 \tag{4.3}$$

where ρ is the water density (1.94 slugs/ft^3) and V is the average channel velocity (ft/s). According to Hey (1979), k_s is approximately 3.5 times the D_{84} particle size for gravel and cobble bed streams.

The critical shear stress required to initiate motion is given by the Shield's relation (Shields 1936):

$$\tau_c = \tau_{*c} (\gamma_s - \gamma) D_c \tag{4.4}$$

where τ_c is the critical shear stress (lb/ft^2), γ_s is the unit weight of the sediment (lb/ft^3), D_c is the particle size (ft), and τ_{*c} is the dimensionless critical shear stress. Studies by Parker et al (1982) and others have shown that the bed material in gravel and cobble bed streams will begin to mobilize at a dimensionless critical shear stress of 0.03 for the median (D_{50}) particle size.

Table 4.8 summarizes the D_{50} values and computed critical shear stresses for the surface bed material gradations presented in Figure 4.30. To evaluate incipient motion conditions along the study reach, dimensionless shear stresses were computed. Dimensionless shear stress is defined as the grain shear divided by the critical shear. When the dimensionless shear stress is less than 1.0, the bed is immobile, and when the dimensionless, shear stress is greater than 1.0 the bed is mobile. A dimensionless shear stress of 1.0 implies incipient motion conditions. Dimensionless shear stresses for discharges of 10,000, 20,000, 50,000, 100,000, and 180,000 cfs are plotted in **Figure 4.31**. The curves given in Figure 4.31 were developed using downstream water surface elevations from the single valued rating curve given in Figure 4.17. The curves show that the bed of the channel is generally immobile at discharges less than or equal to approximately 50,000 cfs. At greater discharges the bed becomes generally mobile, although isolated locations remain immobile even at 180,000 cfs. The relatively high computed critical discharges (between the 5- and 25-year event, depending upon design scenario, Table 4.2) are in agreement with observations made during the field reconnaissance and the geomorphic characteristics of the study reach.

The results given in Figure 4.31 are based on average shear stresses computed at each cross section. The actual shear stress acting on the channel bed varies across the channel due to variations in depth, roughness, velocity, and channel irregularities. An estimate of the magnitude of these effects was made by computing the shear stress distribution across the channel based on conveyance weighting concepts. The computations for a range of discharges showed that maximum shear stresses average 15 to 20 percent greater than the shear stresses based on cross sectionally averaged hydraulics with a maximum increase of up to 60 percent in specific locations. Although

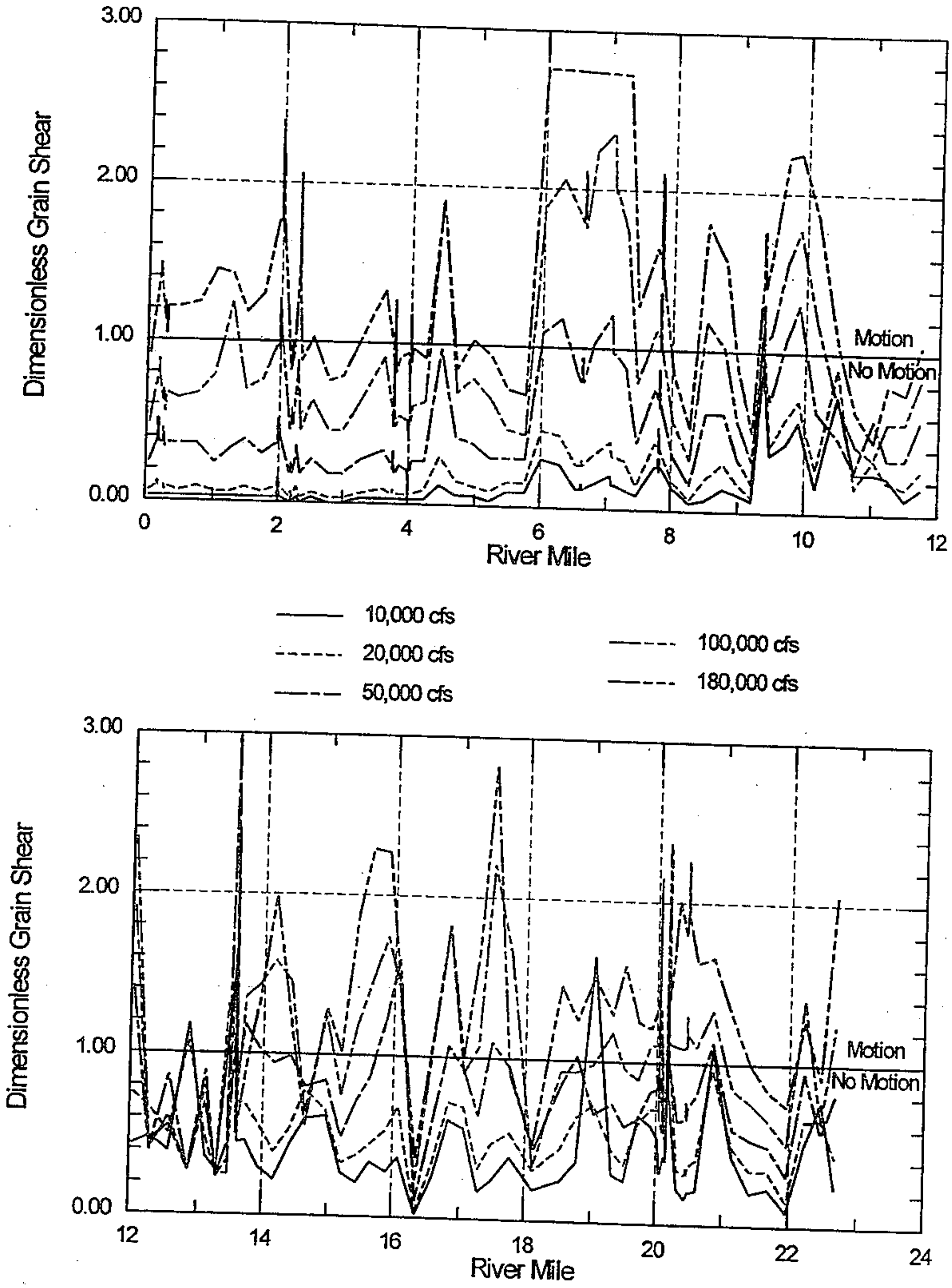


Figure 4.31. Dimensionless grain shear stress profiles computed using single-valued downstream stage-discharge relationship.

Table 4.8. Critical Shear Stress for Surface Layer Median Sediment Size (D_{50}).		
River Mile	D_{50} (mm)	Critical Shear* (lb/ft^2)
0.00 - 7.94	32.9	0.33
7.94 - 13.60	43.1	0.44
13.60 - 20.05	88.2	0.89
20.05 - 22.66	96.4	0.98
*Based on a shields parameter equal to 0.03.		

these calculations are based solely on changes in conveyance and do not account for 2- and 3-dimensional effects caused by channel irregularities, the maximum shear stresses are generally not significantly higher than the average shear stresses. The dimensionless shear stresses based on cross sectionally averaged hydraulics given in Figure 4.31 are thus a good representation of critical conditions required to mobilize the channel bed material.

4.4.2. Sediment Routing

To evaluate the stability of the channel bed and quantify sediment delivery to the Sacramento River under the different design scenarios, detailed sediment routings were performed. The sediment routings provide quantitative information on potential vertical changes along the study reach in response to the design storm events. Since the routing routines do not predict bank erosion or lateral shifting of the channel, bank stability was addressed separately.

Model Setup

The experimental version of the COE's HEC-6 computer program (modified by RCE) was used to perform the sediment routings. The nature of the modifications was discussed in Chapter 3. As with the MDE study reach, the bed material in the Lower American River is sufficiently coarse that the library version or the original experimental version of HEC-6 provided by WES could not be used to simulate sediment transport conditions.

Figure 4.32 shows a schematic of the river network developed for the simulations. The local inflow/outflow points used to represent the flow breakouts upstream of the levees are a simplification of the flow breakouts used in the HEC-2 modeling (Section 4.2.2). The simplification was made to reduce the total number of local inflow/outflow points to eight, the maximum allowed in the HEC-6 model. Rating curves for the breakouts were developed by combining those used in the HEC-2 analysis. **Table 4.9** provides data for the rating curves included in the model. As discussed in the hydraulic analysis, the total discharge in the reach upstream of the levees (upstream of approximately RM 13) was limited to 200,000 cfs for all scenarios, and the total flow in the levee reach was limited to 180,000 cfs for Scenarios 1, 3, 4, and 5 and to 190,00 cfs for Scenario 2.

Geometric data (cross sections, distances, and encroachments) for the HEC-6 model were obtained directly from the HEC-2 input files. Since the HEC-6 model does not contain routines for simulating flow through bridges, all explicit input data for bridges were

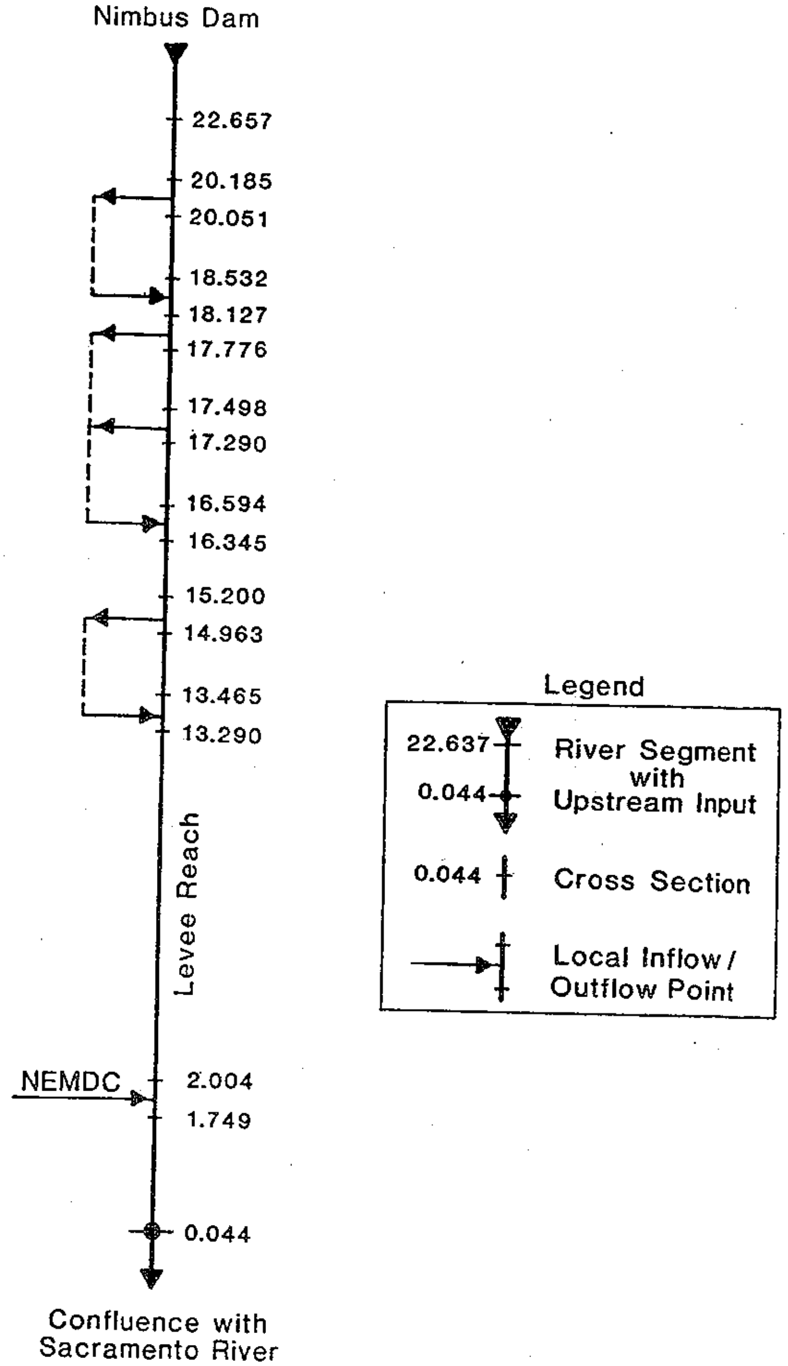


Figure 4.32. Schematic of HEC-6 model of the Lower American River.

Table 4.9. Rating Curves for Local Inflow/Outflow Points in HEC-6 Simulations.

River Mile	Local Inflow/Outflow (cfs)*							
	$Q_{u/s}$=50,000 (cfs)	$Q_{u/s}$=100,000 (cfs)	$Q_{u/s}$=115,000 (cfs)	$Q_{u/s}$=130,000 (cfs)	$Q_{u/s}$=145,000 (cfs)	$Q_{u/s}$=180,000 (cfs)	$Q_{u/s}$=190,000** (cfs)	$Q_{u/s}$=200,000 (cfs)
13.290-13.465	0	2,400	5,700	9,000	12,400	22,000	24,750	7,500***
14.963-15.200	0	-2,400	-5,700	-9,000	-12,400	-22,000	-24,750	-27,500
16.345-16.594	0	2,600	4,800	7,000	10,400	20,000	22,750	25,500
17.290-17.498	0	-2,600	-3,000	-3,000	-3,000	-3,000	-3,000	-3,000
17.776-18.127	0	0	-1,800	-4,000	-7,400	-17,000	-19,750	-22,500
18.127-18.532	0	800	1,900	3,000	5,200	8,000	8,800	9,600
20.051-20.185	0	-800	-1,900	-3,000	-5,200	-8,000	-8,800	-9,600

* Negative number indicates an outflow.
** This column applies only to Scenario 2 where the discharge in the levee reach is limited to 190,000 cfs.
*** Equals 17,500 cfs for Scenario 2

removed. Losses at bridges were accounted for by encroaching the channel to simulate the bridge opening. Closely spaced cross sections (particularly those at bridges) were removed to provide a more uniform cross section spacing for use in the sediment transport calculations. Moveable bed limits were set at the bank stations. Manning's n roughness values are the same as those determined in the HEC-2 hydraulic analysis.

Initial surface layer bed material size gradations used in the model are the same as those presented in Figure 4.30. As discussed in Section 4.4.1, the gradation for the downstream 2 mi of the study reach represents a layer of sand deposited over the coarser channel bed during low flows. Since this material will be flushed during higher flows and the underlying coarse material exposed to erosion, the gradation developed for the next upstream reach (RM 2.00 to RM 7.94) was used in the model. Minor adjustments were made to the measured-bed surface gradations during the calibration process to account for finer material that is under sampled in the Wolman pebble count.

The Parker bed-load equation linked with the Toffaleti relation for determining suspended bed-material load was used to compute the bed material transport capacity along the study reach. The Parker bed-load equation was used since it was developed using data from paved gravel- and cobble-bed streams with a range of sediment sizes similar to those found in the study reach. The Parker equation is based on the concept of equal mobility, which means that within certain limits, the full range of particle sizes found in the bed material matrix are mobilized within a narrow range of shear stresses. Thus, when the shear stress is insufficient to mobilize the median size of the bed material, the bed is armored, and the only sediment transported is throughput load of fine material derived from upstream sources. When the shear stress is sufficient to mobilize the median size, essentially the entire bed material matrix is mobilized. When the bed is mobile, the Parker relation indicates transport of even the larger sizes within the bed material matrix, which is consistent with the field evidence. For this reason, the Parker relation is believed to be the only available equation that will produce reasonable results for the study reach.

Due to Folsom and Nimbus Dams located just upstream of the project reach and the fact that the NEMDC is physically separate from the main channel of the American River, bed material sediment inflow from upstream on the mainstem and tributary sediment inflows were assumed to be zero.

Detailed sediment routings were performed for the 100-year event and the 5 design scenarios. Upstream hydrographs (Folsom release), downstream stages, and NEMDC hydrographs were obtain from the COE (Section 4.1, Figures 4.1 to 4.5, 4.7, and 4.9 to 4.13). Each of the 100-year upstream hydrographs have approximately 3 days of antecedent flows with a discharge of 20,000 cfs. This discharge has an exceedence frequency of approximately 1 to 2 percent based on the Fair Oaks Gage mean daily flow duration curve (Figure 4.16). This relatively high discharge is below the critical discharge for incipient motion for almost the entire study reach (Figure 4.31), and thus had little effect on the sediment routing results. Normally, the simulation of a design event should include a longer antecedent flow period (on the order of 1 year) and a succedent flow period (on the order of several years). The purpose of the antecedent flow period is to allow the model to stabilize through adjustment of the bed elevations and bed gradations before the actual design storm simulation. The purpose of the succedent flow period is to evaluate changes that would occur as the system re-adjusts to the large scale effects induced by the design storm event. Antecedent and succedent flows were not added to the design hydrographs for the simulations performed for this project due to the high critical discharges and stable nature of the channel bed.

To minimize the number of time steps in the simulations, the hourly flow and stage data obtained from the COE were averaged over time periods of variable length, with the length depending on discharge level. The relationship between time step length and discharge depends upon model stability considerations (with higher discharges requiring shorter time steps). Procedures outlined in COE Training Document No. 13, "Guidelines for the Calibration and Application of Computer Program HEC-6" (COE 1981) were used to determine stable time step lengths for the simulations. Maximum time step lengths estimated using these procedures were longer than what was needed to accurately reflect the variation in the design storm hydrographs. To capture this variation, the hydrographs were manually discretized with time step lengths ranging from 5 to 25 hr.

The SPI parameter, which controls the number of iterations in the Exner equation in a given time step, was determined by examining computed total loads at selected cross sections for a range of SPI values at discharges of 50,000 and 180,000 cfs. It was found that computed total loads converged to a relatively constant value at an SPI value of 50, which was used in subsequent simulations.

HEC-6 computes weighted hydraulic parameters for each reach prior to calculating transport capacity. The weighting scheme (input on the I5 card) can be varied to produce a model, which is stable, less sensitive, but allows longer time steps or more sensitive, less stable and require shorter time steps. The sensitive scheme was used in the simulations presented in this report.

Calibration

Measured sediment load data were not available for the project reach to calibrate computed sediment transport rates. Based on the discussion in the previous section, the Parker/Toffaletti relation is believed to be the best available approach for computing bed material transport capacities for coarse bedded streams such as the American River. Because the Parker bed load equation is formulated as an excess shear stress relation based on the concept of approximately equal mobility, this approach should provide a reasonably accurate prediction of the transported gradation as well as the transport capacity. Since the technique used to obtain the surface bed material gradations under-samples the fine portion of the gradation, it was necessary to adjust the tail of the distributions to reflect an appropriate amount of coarse sand and fine gravel.

The adjustments were made to produce reasonable agreement between the gradation of the transported material and the gradation of alluvial deposits along the reach (i.e., bars), which reflects the gradation of material transported at higher flows. The gradation of subsurface samples collected during the field reconnaissance was relatively consistent throughout the study reach. A single representative subsurface gradation developed from individual samples was, therefore, used for the entire study reach (**Figure 4.33**). The subsurface gradations were developed from bulk samples of sediment obtained from beneath the surface pavement. Because it is impractical to obtain large enough samples to contain the larger particles (i.e. cobbles) in representative quantities, the coarse end of the subsurface gradations do not reflect these sizes. The actual gradation of alluvial deposits will fall between the representative subsurface gradation and the surface gradation.

Figure 4.34 shows the final adjusted surface layer gradations developed after several iterations, which reflect increased percentages of gravel and sand sized material. The amount of increase varied among the samples but was generally in the range of 10 to 20 percent. Transported gradations predicted by the HEC-6 model over a range of discharges at several key locations are presented in **Figures 4.35 through 4.38** along with the adjusted surface bed material gradations and the representative subsurface gradation.

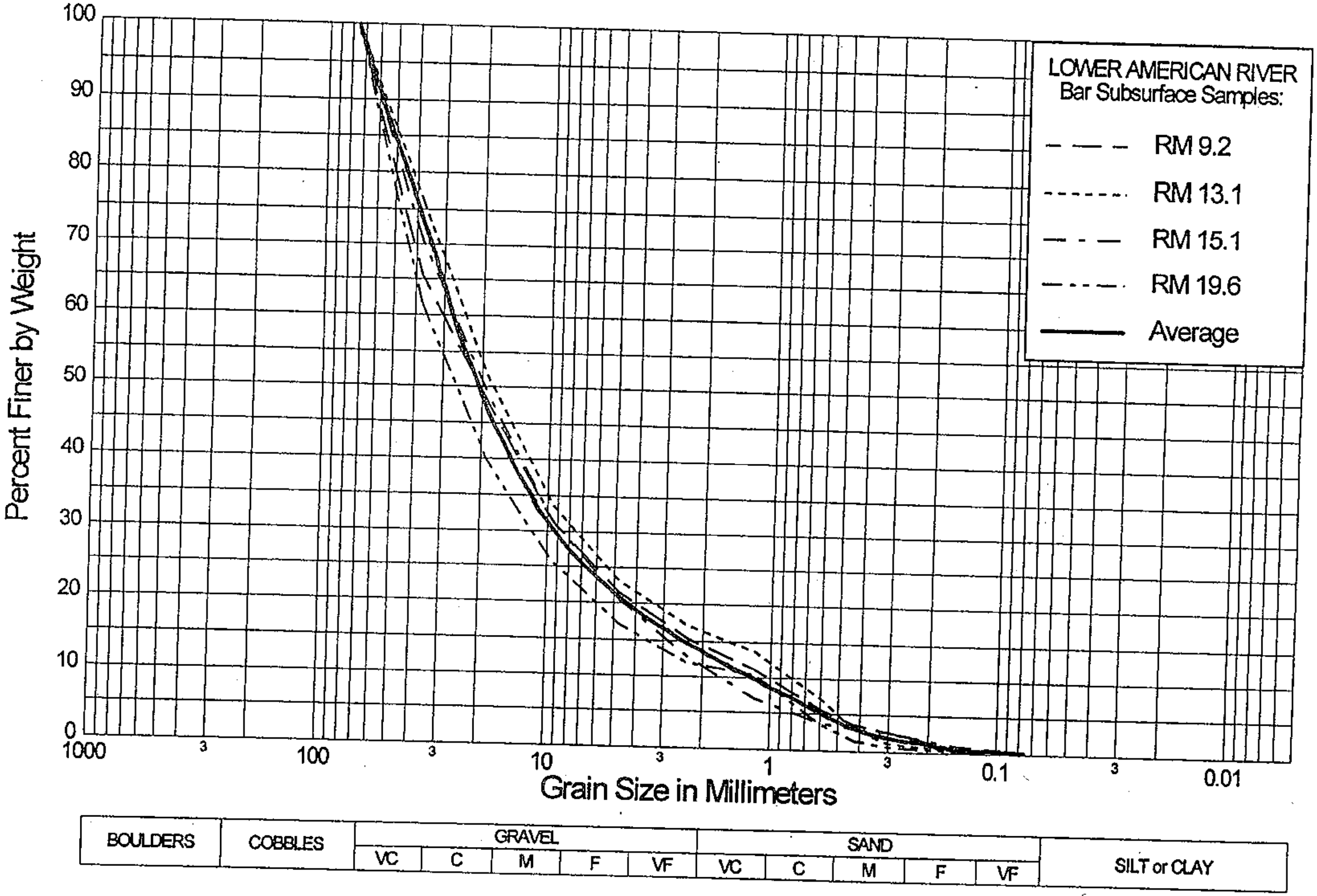


Figure 4.33. Average subsurface bed material sediment size distribution.

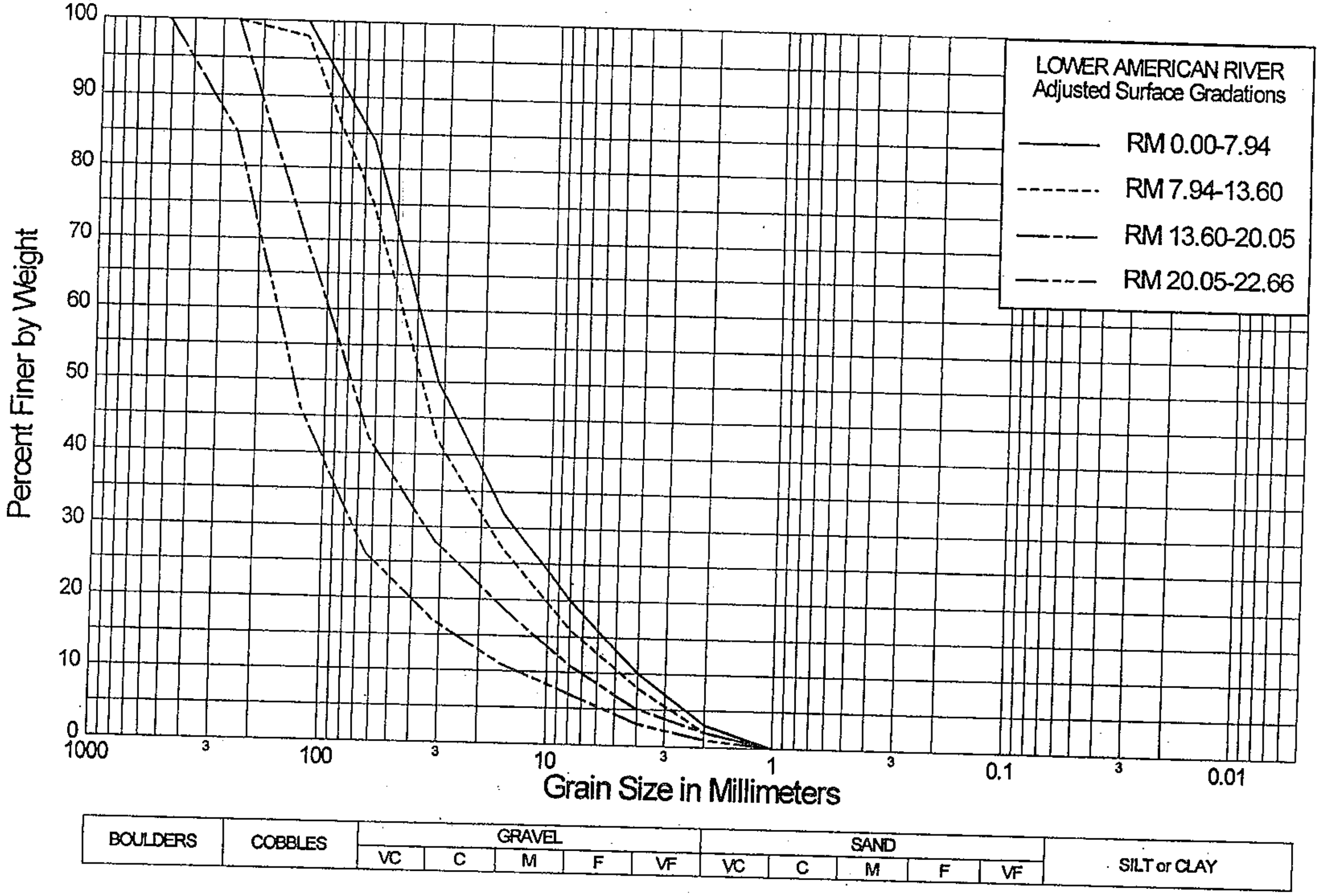


Figure 4.34. Adjusted surface bed material sediment size distributions.

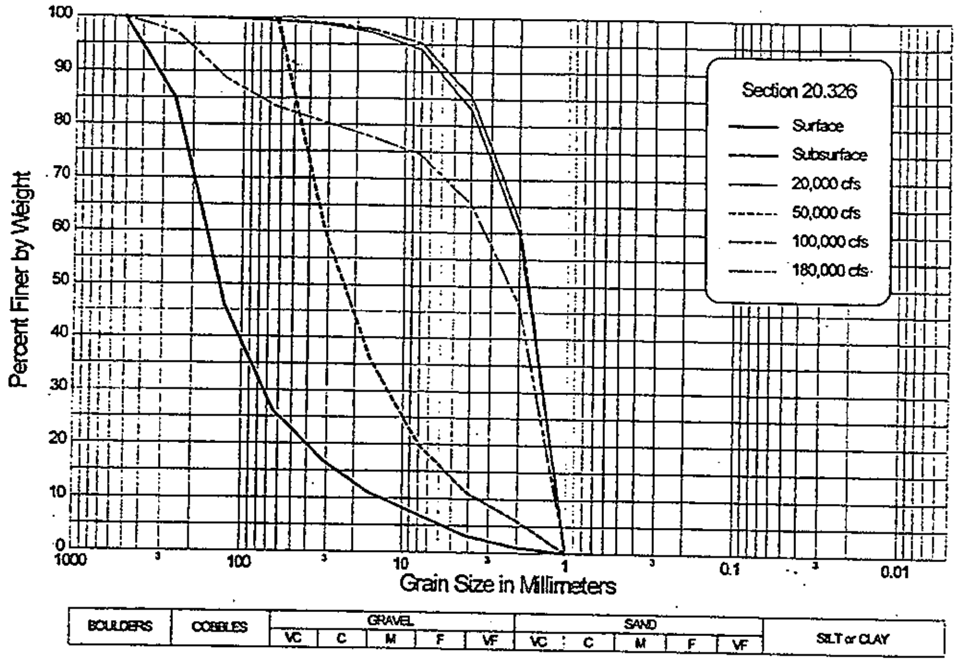


Figure 4.35. Transported bed material sediment size distributions, Section 20.326.

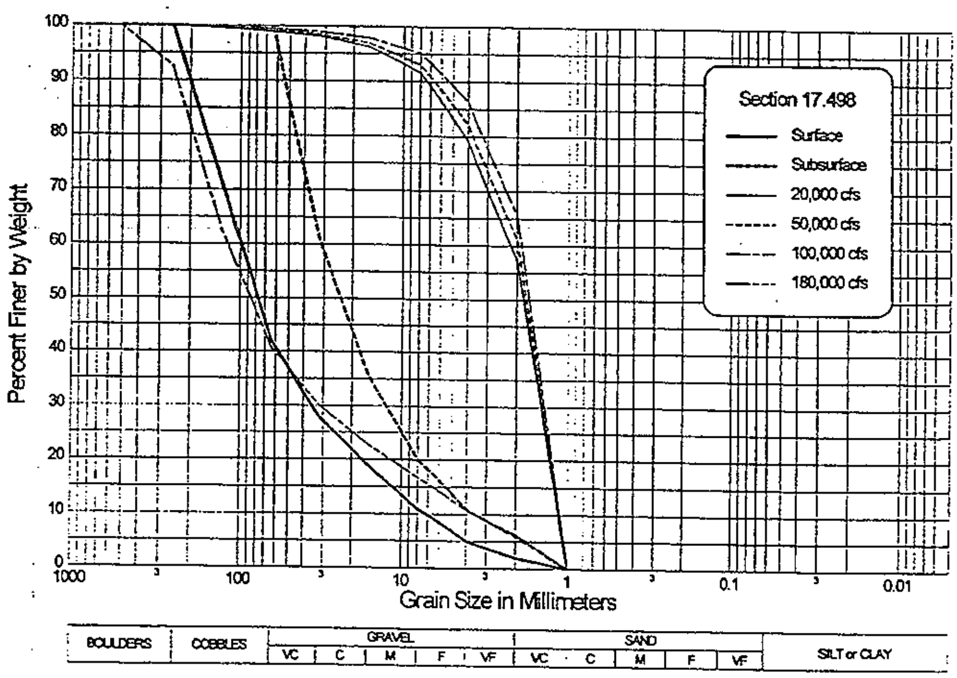


Figure 4.36. Transported bed material sediment size distributions, Section 17.498.

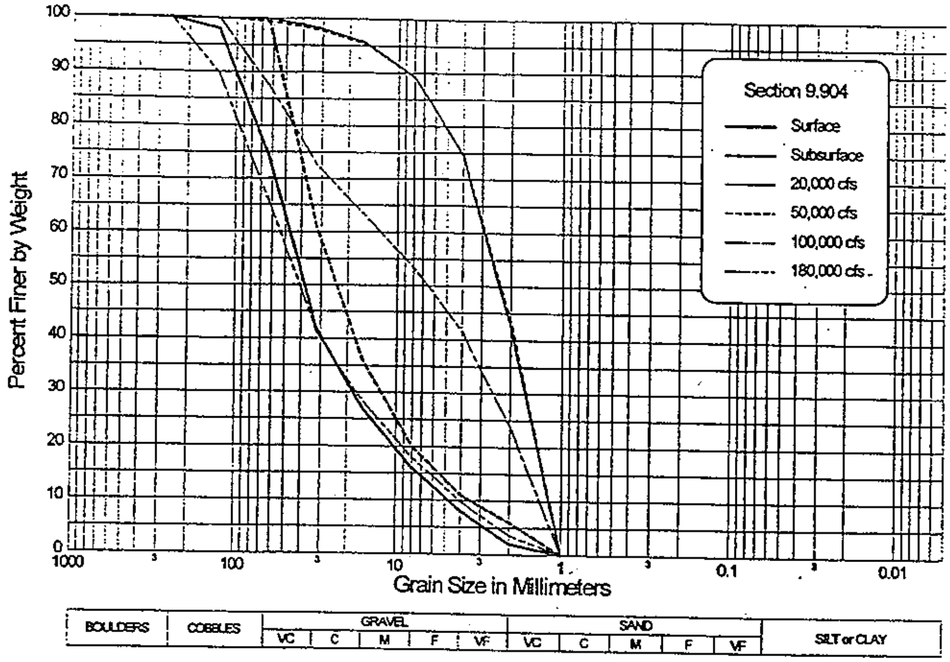


Figure 4.37. Transported bed material sediment size distributions, Section 9.904.

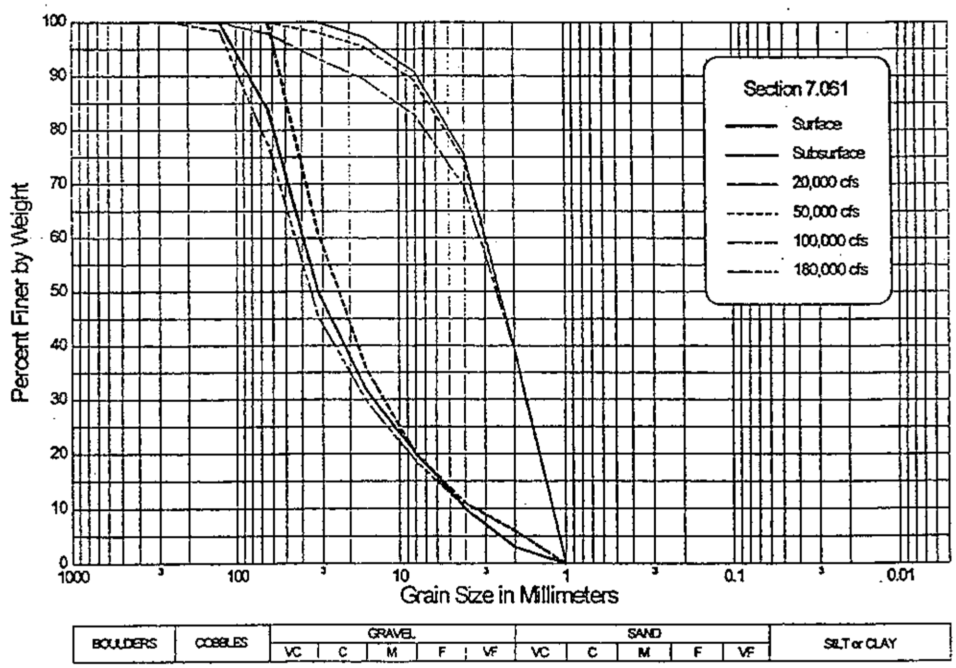


Figure 4.38. Transported bed material sediment size distributions, Section 7.061.

All of these figures show the same general trend of increasing size with increasing discharge. In general, the transported material is primarily sand and fine gravel at discharges (near and slightly higher than the critical discharge for mobilization of the bed material). As the discharge and thus bed shear increase significantly above critical, the transported gradation approaches the subsurface gradation in the sand and fine gravel sizes and the surface gradation in the coarser sizes. The predicted gradations are therefore believed to reasonable.

Results: Changes in Bed Elevation and Bed Gradation

Figure 4.39 shows cumulative bed-elevation changes along the study reach at the end of the 100-year simulation for each of the design scenarios. The figure shows that the bed of the channel is relatively stable under each design scenario with local areas of aggradation and degradation. The model results indicate no general aggradation/ degradation trends along the study reach. Local areas of aggradation tend to occur just downstream of local areas of degradation. The localized nature of the aggradation/ degradation is consistent with field observations and are generally associated with scour in high energy areas with the scoured material re-deposited at the next downstream location where the energy is reduced through channel widening or flattening of the gradient.

Locations of aggradation and degradation are similar for each design scenario. **Table 4.10** summaries the changes in bed elevation at key locations to facilitate a comparison among the different design scenarios. The table shows that cumulative bed elevation changes are similar among the scenarios at a given location. The scenario creating the most change varies depending on the specific location being considered; although, the maximum change occurs more frequently for Scenario 4, which suggests this case may have the greatest overall impact on the vertical stability of the channel bed. Given the relatively small amount of bed change predicted by the model (<2 ft in all locations and <1 ft for the majority) and the small difference among the scenarios, vertical instability within the study reach does not appear to be a significant problem.

The bed elevation changes shown in Figure 4.39 and tabulated in Table 4.10 are cumulative changes at the end of the simulation. Maximum simulated positive (aggradation) and negative (degradation) changes for each design scenario are plotted in **Figure 4.40.** The figure shows that maximum changes are similar to cumulative changes at the end of the simulation. This result is reasonable for the coarse bed conditions in the study reach since significant mobilization of the bed, which is required to cause changes in bed elevation, only occurs at very high flows. Since the bed material transport rates are relatively small, the potential for backfilling during the recessional limb in areas that scour during the high flows is small.

To determine the effects of downstream stage on the aggradation/degradation potential of the study reach, the 100-year simulation was repeated for each design scenario using downstream stages computed from the upper and lower bound rating curves presented in Figure 4.18. Simulated ending cumulative bed elevation changes for each design scenario are shown in **Figures 4.41 and 4.42** for the lower and upper bounds. The figures show that aggradation/degradation trends for each case are similar to the original simulations, which were based on the routed downstream stage hydrograph. As would be expected, aggradation/degradation depths are generally larger in the downstream approximately 8 mi of the reach (the lower bound rating curve [Figure 4.41] and smaller in that reach using the upper bound rating curve [Figure 4.42]). The differences are relatively insignificant.

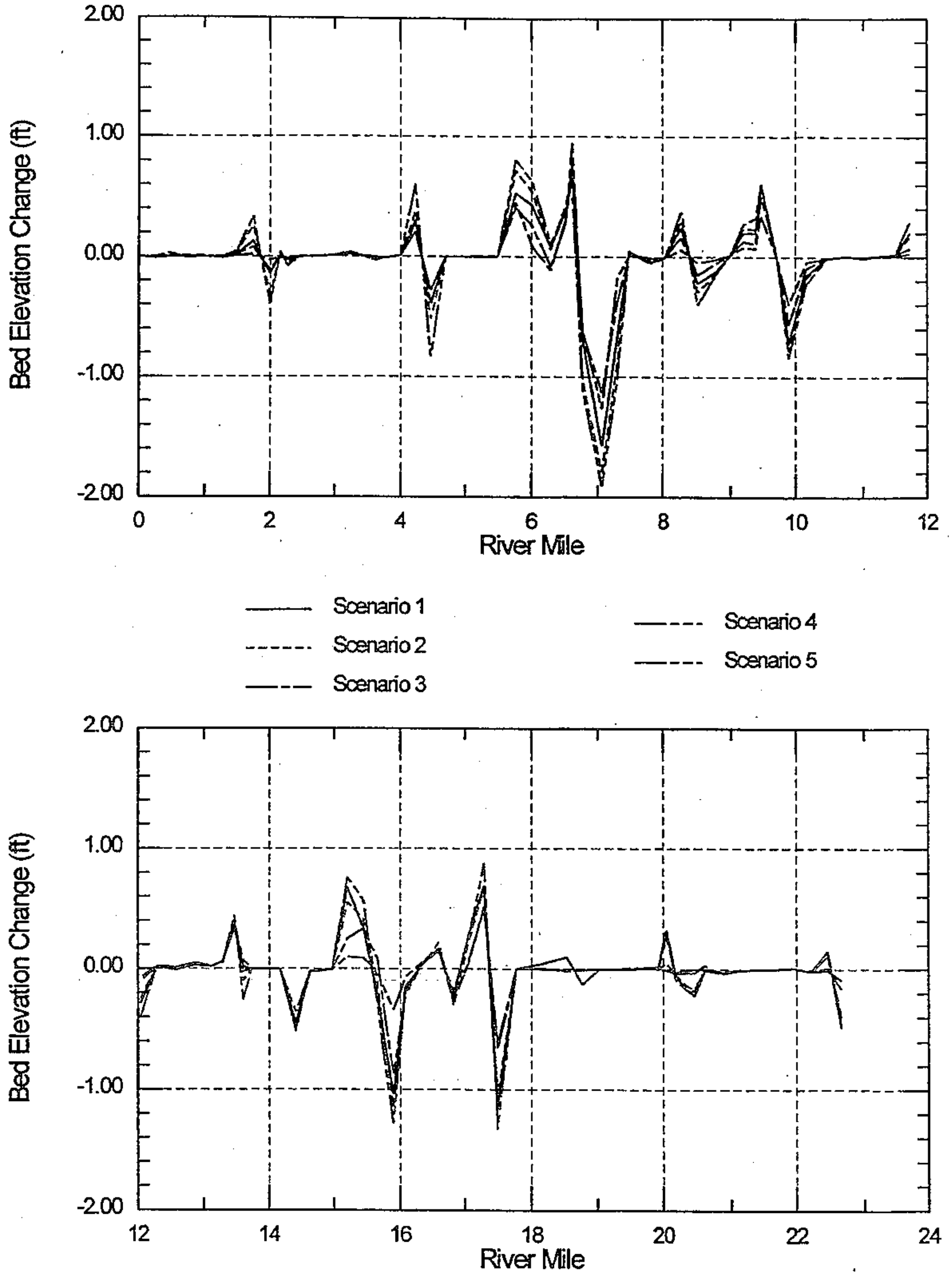


Figure 4.39. Simulated bed elevation changes, COE routed downstream stages.

Table 4.10. End of Simulation Cumulative Bed Elevation Changes at Key Locations, 100-Year Event (COE Routed Downstream Stages).

River Mile	Ending Cumulative Bed Elevation Change (ft)				
	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5
4.230	0.21	0.38	0.30	0.28	0.61
4.459	-0.28	-0.52	-0.40	-0.38	-0.83
5.770	0.53	0.72	0.41	0.81	0.45
6.618	0.66	0.65	0.78	0.49	0.95
7.061	-1.57	-1.79	-1.13	-1.91	-1.26
9.479	0.60	0.60	0.50	0.61	0.34
9.904	-0.72	-0.78	-0.56	-0.84	-0.39
14.418	-0.45	-0.52	-0.45	-0.36	-0.50
15.200	0.68	0.55	0.25	0.76	0.10
15.902	-1.04	-1.15	-0.87	-1.28	-0.34
17.290	0.69	0.67	0.50	0.88	0.52
17.498	-1.16	-1.06	-0.67	-1.33	-0.63

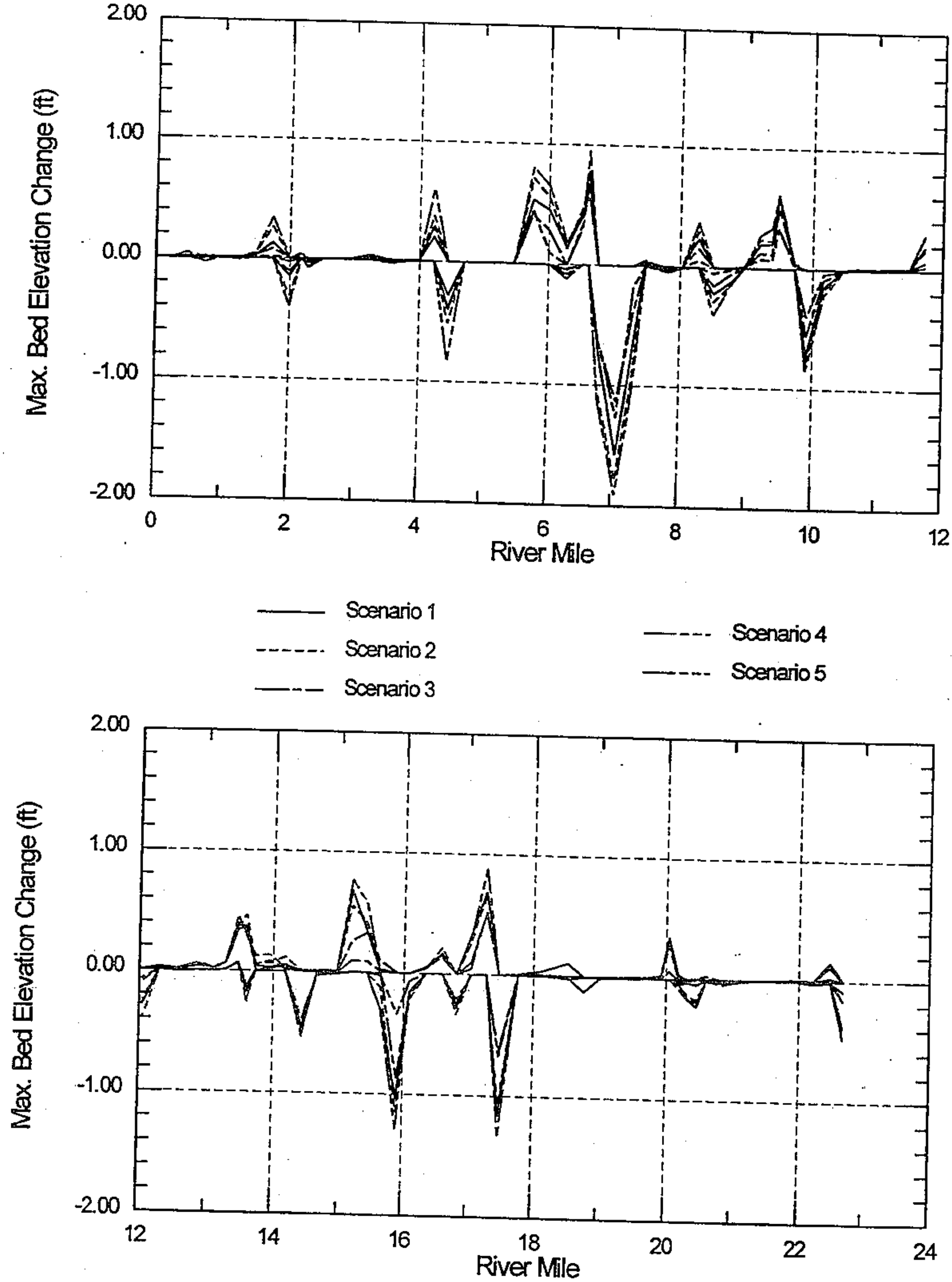


Figure 4.40. Maximum (positive and negative) simulated bed elevation changes, COE routed downstream stages.

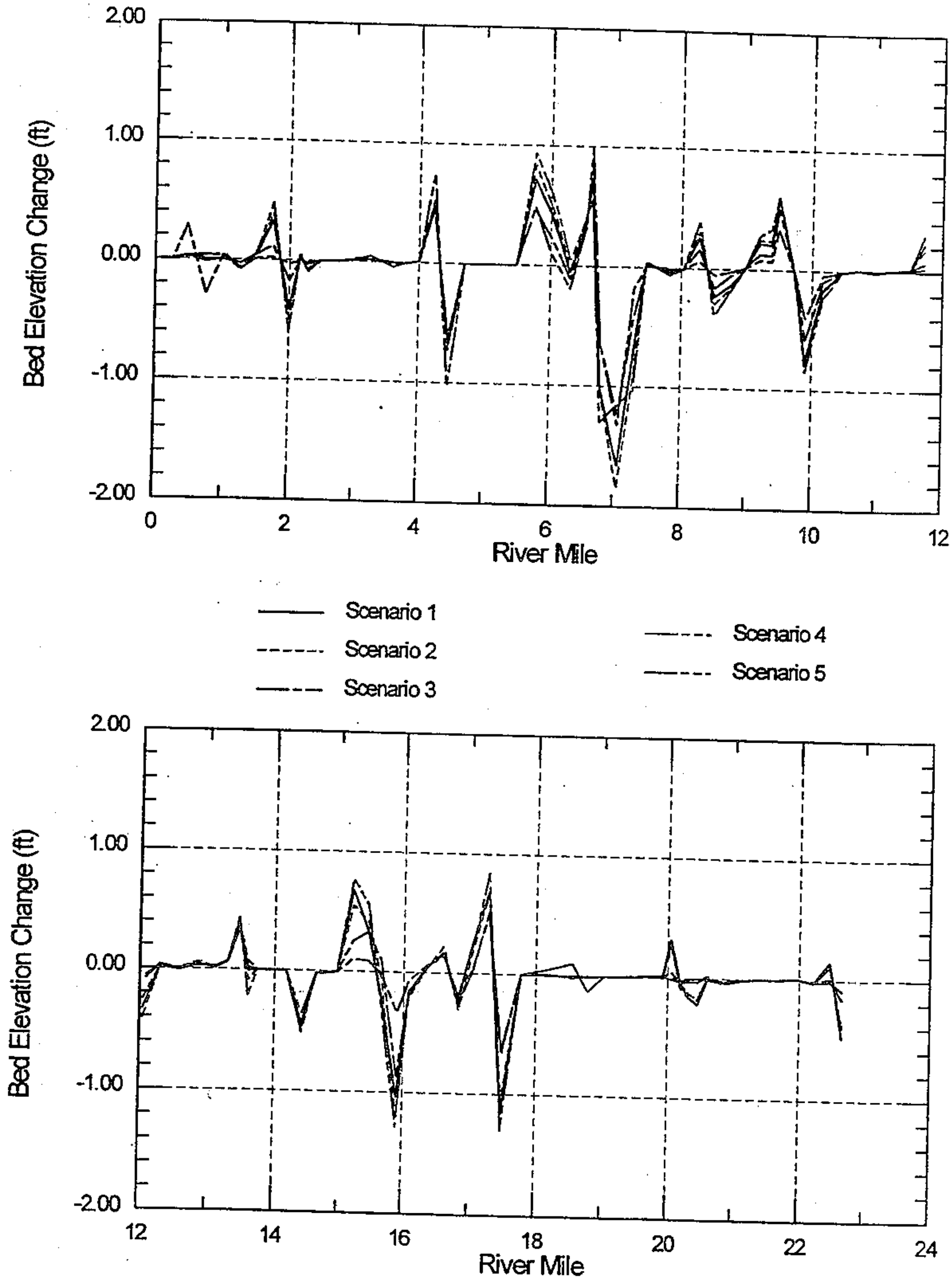


Figure 4.41. Simulated bed elevation changes, lower bound downstream stages.

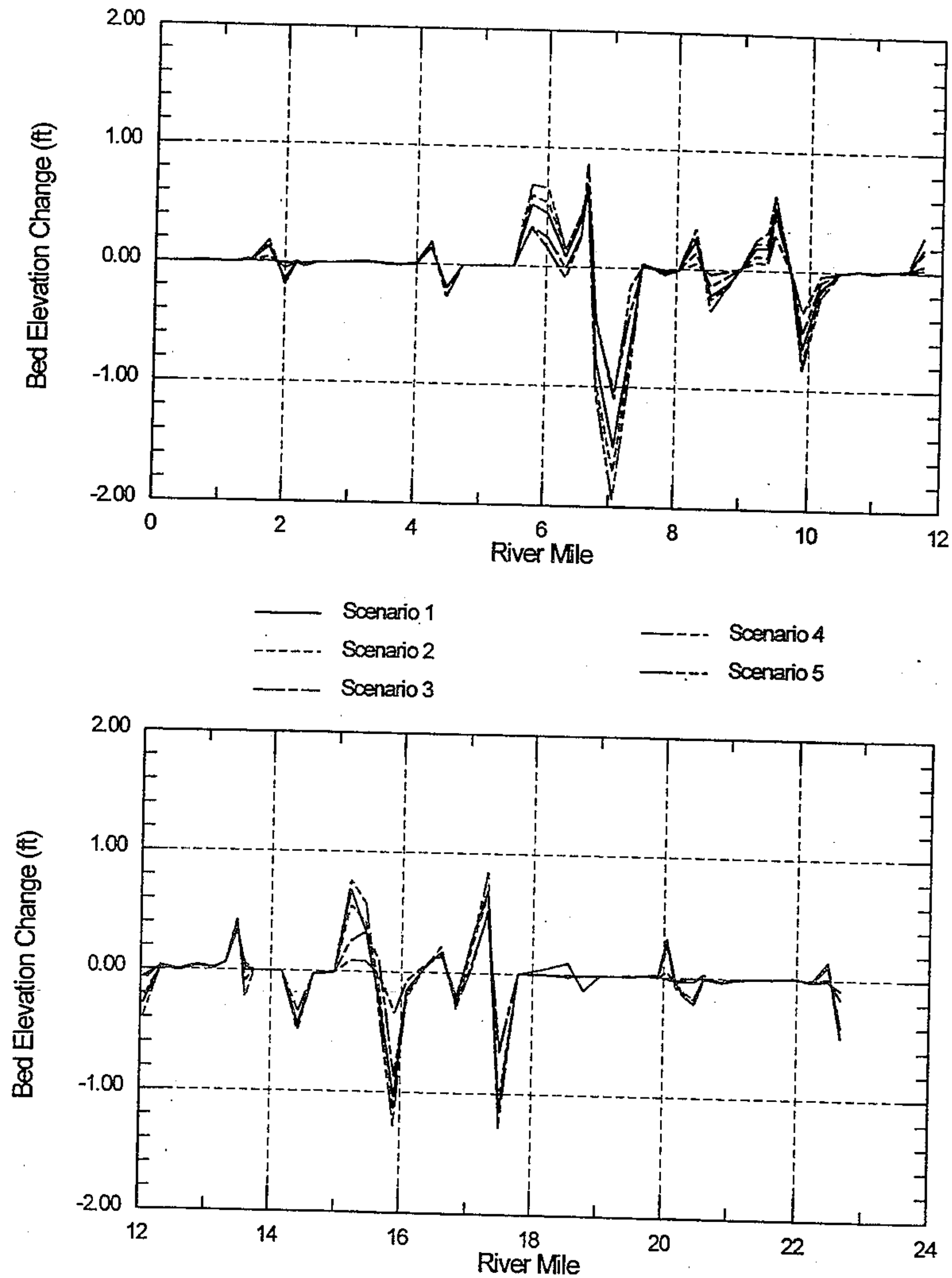


Figure 4.42. Simulated bed elevation changes, upper bound downstream stages.

As sediment is mobilized during the passage of a flood hydrograph and areas of aggradation and degradation develop, the grain size distribution of the channel bed changes due to sediment sorting. To illustrate changes in the bed material sediment size distribution along the study reach, the D_{16}, D_{50}, and D_{84} sediment sizes were plotted against time at selected cross sections where bed elevation changes were predicted by the model. The plots were developed for the 100-year event Scenario 1 simulation using the routed downstream stage hydrograph. Cumulative bed elevation changes versus time are included on the plots for comparison. **Figure 4.43** shows the changes at Cross Section 17.498, a section with approximately 1.2 ft of total degradation. The plot shows that the bed of the channel becomes coarser during degradation with the median (D_{50}) size increasing from approximately 110 mm at the beginning of the simulation to approximately 140 mm at the end. The coarsening is the result of a winnowing of the finer particles during degradation.

The change at cross section 17.290, an area of aggradation just downstream of cross section 17.498, is illustrated in **Figure 4.44**. The plot shows that the cross section aggrades throughout the simulation. In this case the bed material fines during the early portion of the simulation, which is the typical response to aggradation, but it coarsens during the latter portion of the simulation. The coarsening is probably due to the deposition of coarser material eroded from Cross Section 17.498 during the peak flows. **Figures 4.45 and 4.46** show the changes at cross sections 7.061 and 6.618. Similar to the previously discussed cross sections, cross section 7.061 is degradational and cross section 6.618, located just downstream is aggradational. In all the cross sections examined, changes in the surface bed gradation were minor, which reflects the relatively small aggradation/degradation trends along the project reach.

Results: Bed Material Sediment Budget

A bed material sediment budget was developed for the study reach for the 100-year event under each design scenario by accumulating the total quantity of sediment transported past selected locations during the HEC-6 simulations. The locations were selected to define subreaches having similar geomorphic and sediment transport characteristics. **Table 4.11** summarizes the subreach definitions. To facilitate evaluation of the results, the total bed material sediment load was divided into three sediment size groupings: (1) coarse sand through fine gravel (1 - 8 mm), (2) medium gravel through very coarse gravel (8 - 64 mm), and (3) fine cobbles through small boulders (64 - 512 mm). **Table 4.12** summarizes the results for each of the design scenarios. The results show that the overall study reach is degradational under all design scenarios with the quantity of material removed during the simulation varying from approximately 880 tn for scenario 1 to 1,370 tn for scenario 2. This result was expected since no bed material sized sediment is brought into the reach from upstream due to the presence of Folsom and Nimbus Dams. Aggradation/degradation trends along the study reach vary with design condition. The following general observations from the information in Table 4.12:

- Subreach 1 is degradational for all scenarios since the upstream supply is zero.
- Subreach 3 is aggradational for all scenarios due to it's relatively flatter gradient in comparison with the upstream reaches.

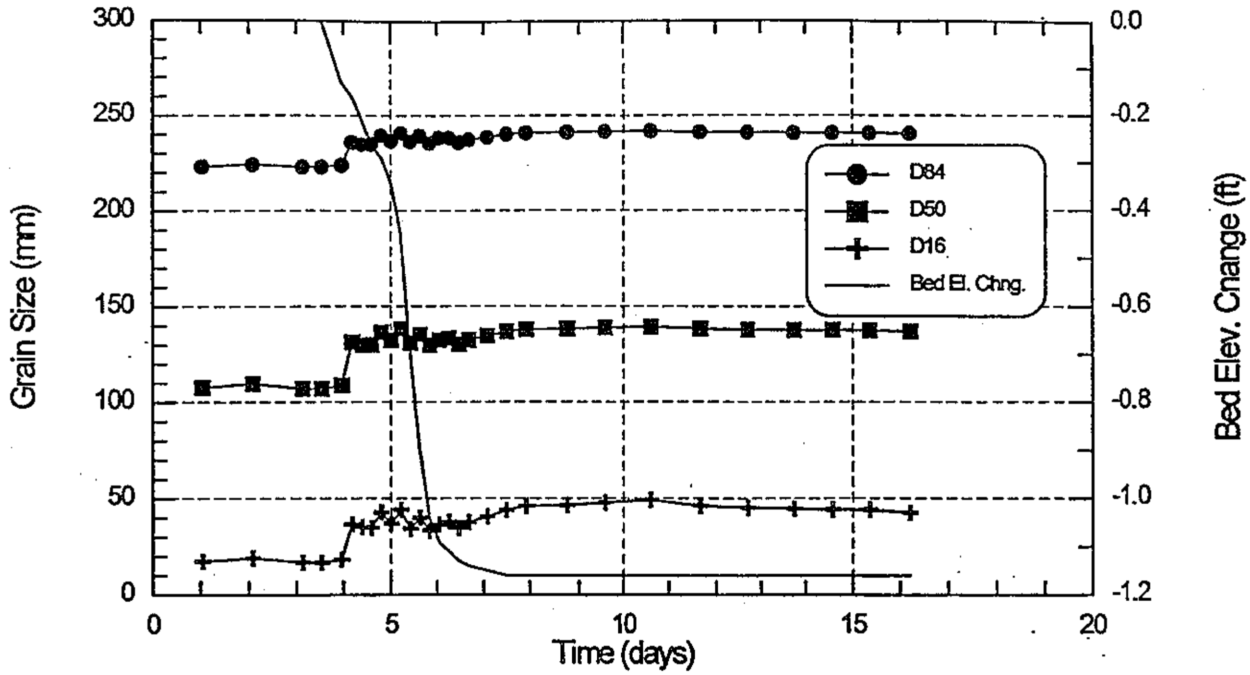


Figure 4.43. Bed material and bed elevation changes versus time, RM 17.498 (100-year event, Design Scenario 1).

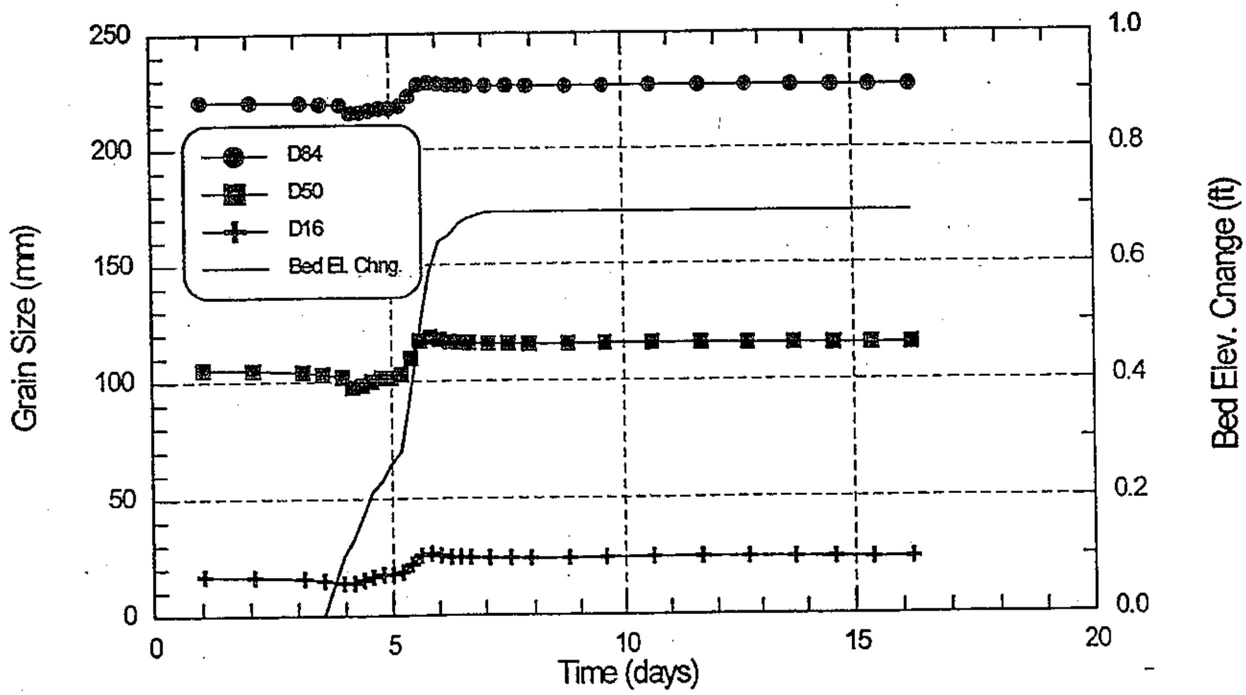


Figure 4.44. Bed material and bed elevation changes versus time, RM 17.290 (100-year event, Design Scenario 1).

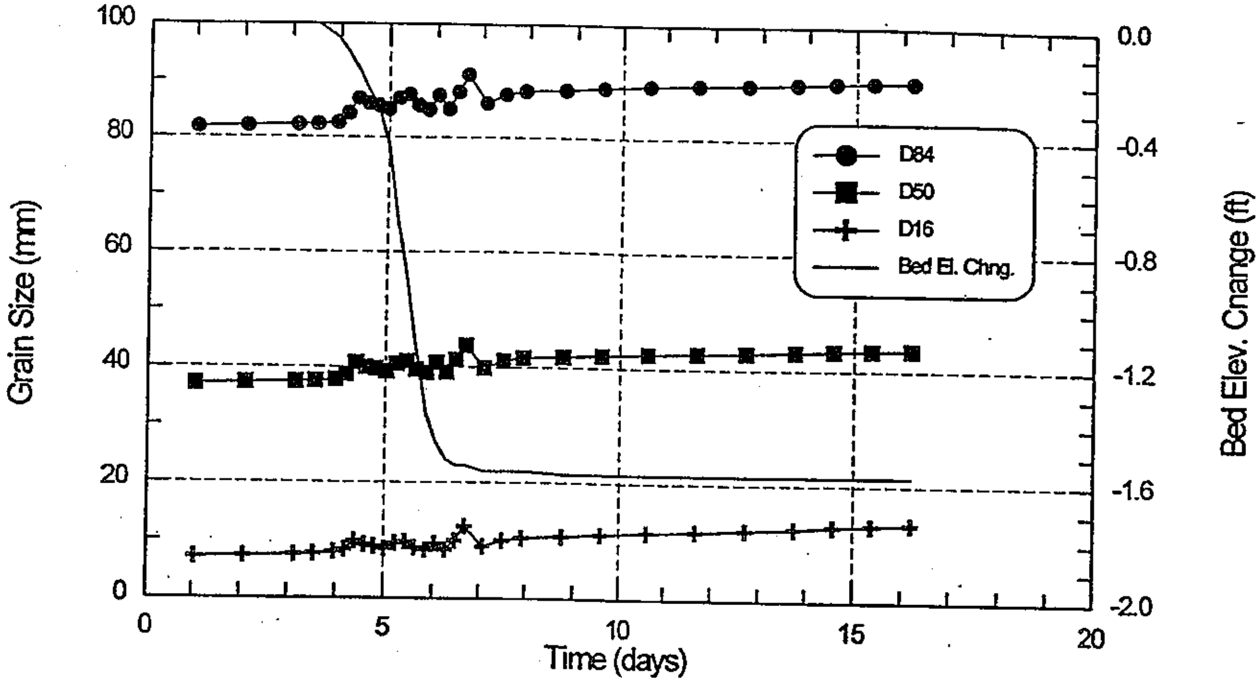


Figure 4.45. Bed material and bed elevation changes versus time, RM 7.061 (100-year event, Design Scenario 1).

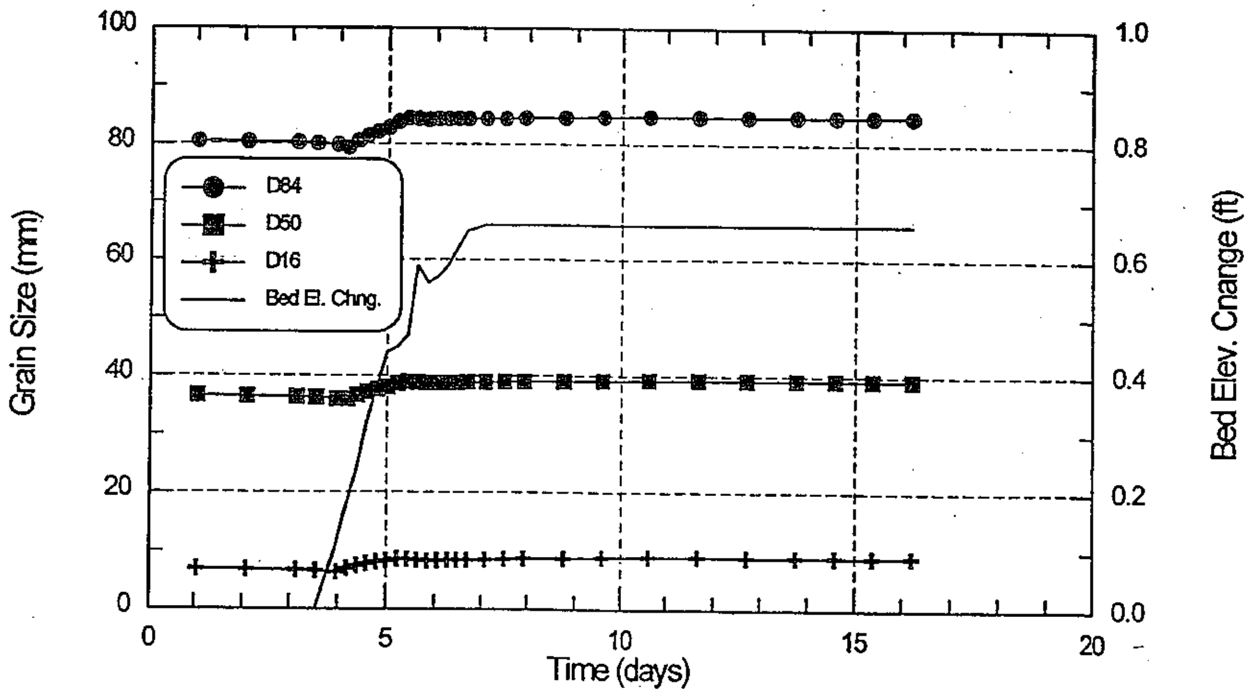


Figure 4.46. Bed material and bed elevation changes versus time, RM 6.618 (100-year event, Design Scenario 1).

Table 4.11. Subreach Delineation Used in Sediment Budget Calculations.		
Subreach Number	River Mile	Description
1	22.657	Fish weir downstream of Nimbus Dam
2	18.127	San Juan Rapid
3	13.290	Downstream end of expansion near Goethe Park
4	8.282	Upstream end of island between Howe and Watt Avenues
5	4.230	Upstream of Interstate 80
6	0.044	Confluence with Sacramento River

- The aggradation/degradation tendency within Subreaches 2, 4, and 5 vary depending on the with scenario; but the volumes are small in comparison to the amount in subreaches 1 and 3 (indicating that the bed is relatively stable in these reaches).

- The majority (generally >80 percent) of the material transported and deposited and/or eroded within the study reach is coarse sand to fine gravel size ranges (1mm-8mm).

- Even in subreaches 1 and 3, which show the greatest amount of aggradation/degradation, the volume of material is relatively insignificant, which corresponds to an average bed elevation change through the subreach for worst case conditions of less than 0.1 in. during the 100-year flood.

Table 4.12. Summary of Bed Material Sediment Budget, 100-Year Event, COE Supplied Downstream Stage.

Subreach	Aggradation/Degradation Amount (tn)			
	Total	Size Group 1 (1mm - 8mm)	Size Group 2 (8mm - 64 mm)	Size Group 3 (64 mm - 512 mm)
Scenario 1				
1	-3847	-3486	-254	-108
2	384	251	29	104
3	2429	2268	160	1
4	-187	-146	-40	-1
5	340	330	10	0
Study Reach Total	-881	-783	-95	-3
Scenario 2				
1	-4147	-3659	-348	-140
2	653	393	124	136
3	2381	2225	155	1
4	-287	-229	-55	-2
5	25	226	-167	-34
Study Reach Total	-1374	-1044	-292	-39
Scenario 3				
1	-4063	-3672	-302	-90
2	-747	-892	60	86
3	3791	3611	178	2
4	-323	-275	-47	-1
5	293	312	-16	-4
Study Reach Total	-1049	-915	-127	-7
Scenario 4				
1	-4270	-3771	-326	-174
2	-1355	-1487	-39	170
3	4505	4212	291	1
4	-165	-116	-46	-3
5	-58	154	-177	-35
Study Reach Total	-1344	-1008	-296	-40
Scenario 5				
1	-3802	-3590	-195	-17
2	507	507	-14	14
3	2143	1998	144	1
4	-594	-521	-71	-1
5	707	672	33	2
Study Reach Total	-1039	-934	-103	-2

Bed material sediment loads for individual events were developed by integrating a representative sediment rating curve over the individual hydrographs. If it is assumed that sediment passes through the lower portion of the study reach without net aggradation or degradation, a sediment rating curve representative of the middle portion of the study reach can be used for the calculations. **Figure 4.47** shows bed material sediment loads versus discharge for 10 cross sections between RM 12 and RM 14 computed during the HEC-6 simulation for Scenario 1 (100-year event, Sacramento District routed downstream stages). The figure shows that computed loads are scattered over two log cycles, with concentrations ranging from approximately 0.1 to 50 ppm. These very small concentrations indicate that bed material sediment loads are very small and bed material sediment yield to the Sacramento River will be minimal. To estimate the loads, three sediment rating curves were constructed using the computed loads, one that passes through the middle of the data and two that bracket most of the data (Figure 4.47). Sediment yields for individual size groupings were computed from relationships between the fraction of transported sediment in each group and the total sediment discharge.

Tables 4.13 and 4.14 summarize the bed-material sediment budget results for the 100-year event under each of the design scenarios using the upper and lower bound rating curves (Figure 4.18). The results show that use of the lower bound rating curve (Table 4.13) increases overall degradation and use of the upper bound rating curve (Table 4.14) reduces overall degradation. The effects tend to be concentrated in the lower subreaches, which is consistent with the computed bed elevation changes (Figures 4.41 and 4.42).

The above results indicate that the channel is vertically stable for the 100-year event under all the design scenarios. Because of this and the incipient motion analysis indicates that relatively high discharges are required to initiate bed material motion (Section 6.2), detailed sediment routings for other events were not performed. It can be concluded that the bed of the channel along the study reach will remain relatively stable (regardless of the design scenario).

4.4.3. Bed Material Sediment Yield to the Sacramento River

The average annual bed material sediment yield to the Sacramento River was developed by estimating the average annual bed material load generated within the study reach. This was accomplished by computing bed material sediment yields for individual storm events and integrating the results based on their relative probability of occurrence. This approach assumes that the bulk of the sediment is transported during individual flood events storm events. Given the finding that the channel bed is generally immobile at discharges less than about 20,000 cfs, this is considered to be a realistic assumption.

Bed material sediment loads for individual events were developed by integrating a representative sediment rating curve over the individual hydrographs. It is assumed that sediment passes through the lower portion of the study reach without net aggradation or degradation, a sediment rating curve representative of the middle portion of the study reach can be used for the calculations. Figure 4.47 shows bed material sediment loads versus discharge for 10 cross sections between RM 12 and RM 14, which were computed during the HEC-6 simulation for scenario 1 (100-year event, Sacramento District routed downstream stages). The figure shows that computed loads are scattered over two log cycles with concentrations ranging from approximately 0.1 to 50 ppm. These small concentrations indicate that bed material sediment loads are small and bed material sediment yield to the Sacramento River will be minimal. To estimate the loads, three sediment rating curves were constructed using the computed loads: one that passes through the middle of the data and two that bracket most of the data (Figure 4.47). Sediment yields for individual size groupings were computed from relationships between the fraction of transported sediment in each group and the total sediment discharge.

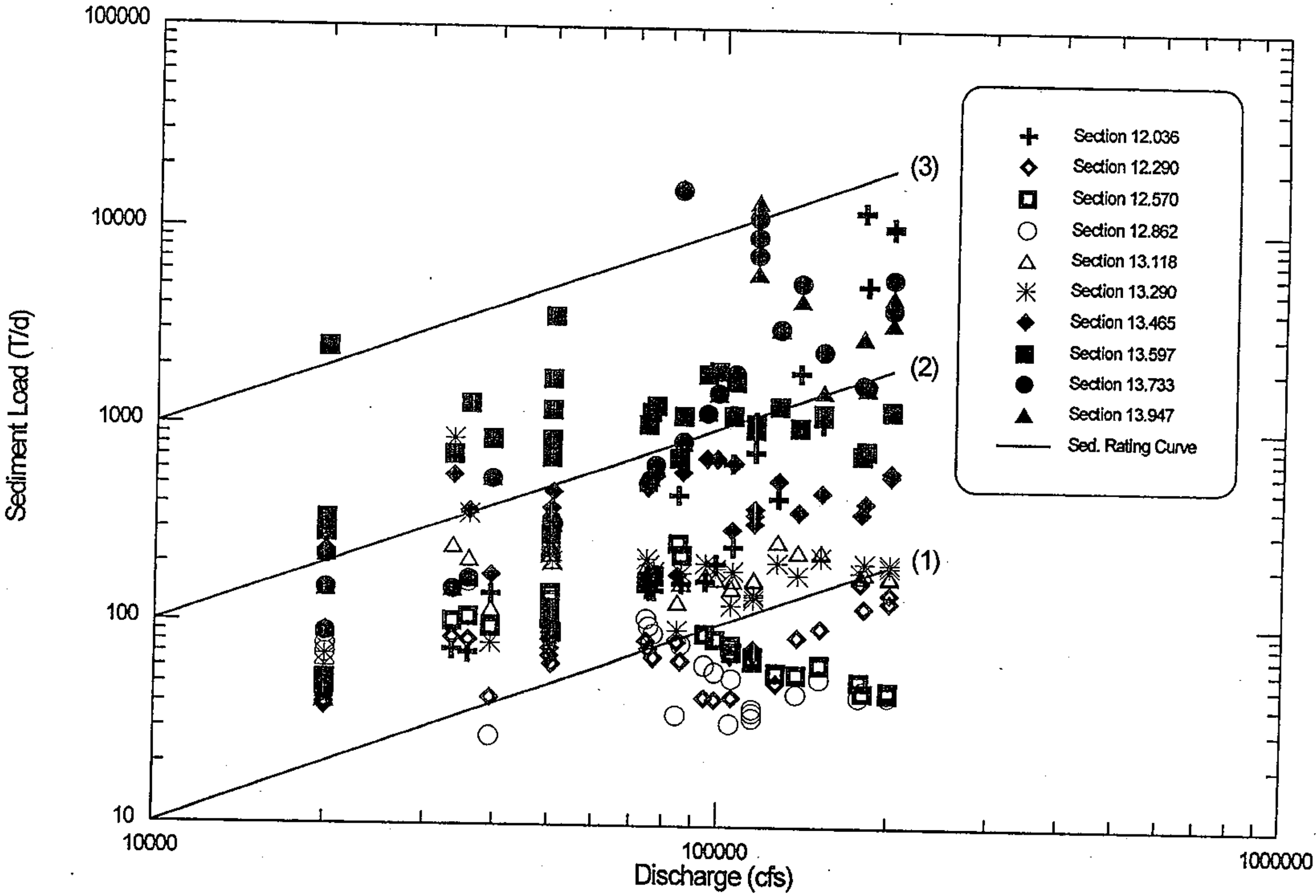


Figure 4.47. Computed bed material sediment loads versus discharge at several cross sections between RM 12 and RM 14 (100-yr event, Design Scenario 1).

Table 4.13. Summary of Bed Material Sediment Budget, 100-Year Event, Lower Bound Downstream Stage.

Subreach	Aggradation/Degradation Amount (tn)			
	Total	Size Group 1 (1mm - 8mm)	Size Group 2 (8mm - 64mm)	Size Class 3 (64mm - 512mm)
Scenario 1				
1	-6745	-6392	-209	-108
2	5679	5377	163	103
3	171	189	-19	2
4	-507	-452	-51	-3
5	-2804	-1014	-1428	-293
Study Reach Total	-4206	-2292	-1544	-299
Scenario 2				
1	-4147	-3660	-348	-140
2	708	431	141	137
3	2308	2170	137	1
4	-460	-376	-79	-6
5	-1942	-443	-1242	-257
Study Reach Total	-3533	-1877	-1391	-265
Scenario 3				
1	-4034	-3659	-292	-83
2	-401	-599	120	79
3	3413	3302	109	2
4	-422	-364	-57	-1
5	-347	-38	-261	-48
Study Reach Total	-1791	-1358	-381	-51
Scenario 4				
1	-4269	-3774	-323	-172
2	-779	-1025	77	169
3	3925	3751	173	1
4	-286	-215	-65	-6
5	-1746	-406	-1117	-223
Study Reach Total	-3154	-1668	-1254	-232
Scenario 5				
1	-3802	-3590	-195	-17
2	580	555	11	14
3	2052	1933	118	1
4	-731	-650	-79	-2
5	30	36	-5	-1
Study Reach Total	-1872	-1717	-149	-5

Table 4.14. Summary of Bed Material Sediment Budget, 100-Year Event, Upper Bound Downstream Stage.

Subreach	Aggradation/Degradation Amount (tn)			
	Total	Size Group 1 (1mm - 8mm)	Size Group 2 (8mm - 64mm)	Size Group 3 (64mm - 512mm)
Scenario 1				
1	-6748	-6394	-210	-108
2	5705	5400	164	103
3	192	207	-17	2
4	-189	-166	-22	0
5	362	400	-31	-6
Study Reach Total	-680	-553	-116	-9
Scenario 2				
1	-4147	-3659	-348	-140
2	747	480	130	137
3	2326	2175	151	1
4	-37	-5	-33	0
5	395	390	9	-3
Study Reach Total	-716	-619	-91	-6
Scenario 3				
1	-4038	-3663	-292	-83
2	-315	-518	124	79
3	3395	3286	107	2
4	-87	-57	-30	0
5	469	428	39	2
Study Reach Total	-576	-524	-51	-1
Scenario 4				
1	-4262	-3770	-321	-172
2	-609	-860	83	169
3	3796	3627	168	1
4	92	108	-16	0
5	334	325	11	-2
Study Reach Total	-649	-570	-74	-4
Scenario 5				
1	-3801	-3589	-195	-17
2	657	623	20	14
3	2054	1941	112	1
4	-238	-187	-51	-1
5	681	630	49	3
Study Reach Total	-648	-583	-65	0

The average annual sediment yield was computed by integrating the sediment yield frequency curve and developed using the computed sediment yields from individual events. **Figure 4.48** illustrates a typical sediment yield frequency curve. The integration was accomplished numerically using the results from the individual events. The numerical procedure involves summing the incremental trapezoidal areas established by the calculated sediment yields at discrete return periods. For the 2-, 3-, 5-, 10-, 25, 50-, 100-, and 200-year events analyzed in this study, the numerical integration results in the following relationship:

$$Y_m = 0.0075Y_{200} + 0.0075Y_{100} + 0.015Y_{50} + 0.04Y_{25} + 0.0\ Y_{10} + 0.117Y_5 + 0.15Y_3 + 0.333Y_2 \tag{4.5}$$

where Y_m is the mean annual sediment yield and Y_x is the sediment yield from the event with return period x.

Table 4.15 presents the computed average annual sediment yields for each design scenario using the three sediment rating curves given in **Figure 4.52**. The results for each sediment rating curve indicate that total yields are greatest for scenario 4 and smallest for scenario 2, although the differences are relatively small. The sediment distribution varies with the magnitude of the total load, with small total loads being composed mostly of sands and fine gravel (1 mm - 8 mm), and high total loads being composed of material in all the size groups. This variation is a direct result of the fact that at higher discharges the entire bed becomes mobile with the distribution of transported material approaching that of the subsurface material (Figures 4.35 through 4.38). Even using the upper bound rating curve, the computed bed material sediment loads to the Sacramento River are small.

4.5. Bank Erosion

The results of the sediment transport analysis presented in Section 4.4 indicate that vertical stability of the channel bed along the study reach of the American River is not a significant issue. Because of the importance of the levees in maintaining the flood carrying capacity of the reach and lateral stability of the channel is a significant issue. This chapter presents the results of quantitative analysis of the relative effects of the various design scenarios on lateral stability of the channel.

4.5.1. Analysis Procedure

The potential for increased bank erosion or damage to existing bank protection associated with the different flow scenarios must consider duration of the flows as well as their magnitude during a specific flow event. In addition, the relative effect of the range of possible flow events must also be considered since the infrequent, high discharges associated with the design event are not necessarily the most significant in causing bank erosion and lateral adjustment of the channel. This is in contrast to most design procedures for bank protection that only consider conditions for a specific discharge. To incorporate the effects of both duration and magnitude into the evaluation of the effects of the various scenarios, lateral stability analyses for this project were performed based on the concept of total work applied to the banks. Work was computed at specific locations for each design scenario using the results of the HEC-2 hydraulic modeling and measurements of bed geometry from available mapping. The resulting values were then

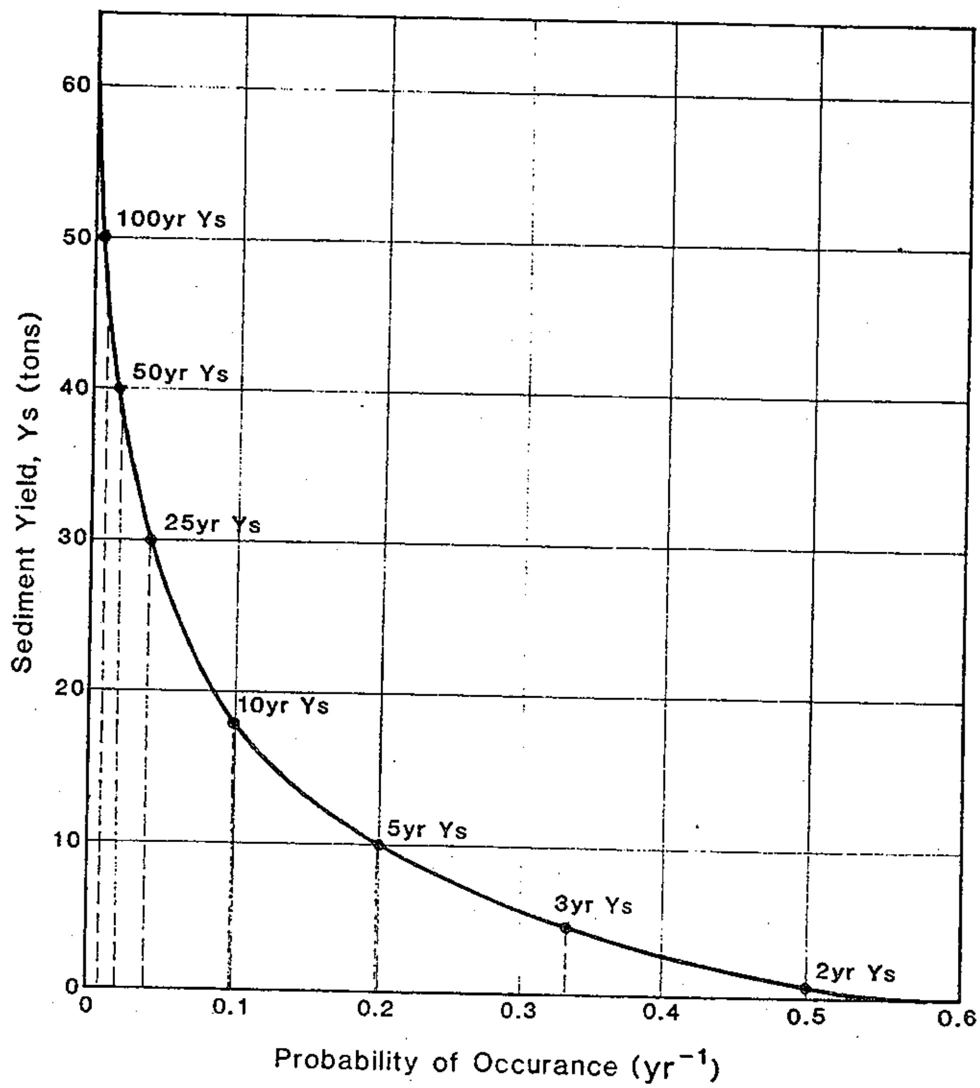


Figure 4.48. Typical sediment yield frequency curve.

Table 4.15. Average Annual Bed Material Sediment Yields to the Sacramento River.				
Design Scenario	Bed Material Sediment Yield (tn)			
	Total	Size Group 1 (1mm - 8mm)	Size Group 2 (8mm - 64mm)	Size Group 3 (64mm - 512mm)
Lower Bound Sediment Rating Curve				
Scenario 1	158	158	0	1
Scenario 2	157	156	0	1
Scenario 3	170	169	0	0
Scenario 4	172	172	0	0
Scenario 5	159	159	0	0
Median Sediment Rating Curve				
Scenario 1	1584	1186	164	234
Scenario 2	1570	1171	164	235
Scenario 3	1696	1316	156	223
Scenario 4	1724	1343	157	224
Scenario 5	1588	1188	165	235
Upper Bound Sediment Rating Curve				
Scenario 1	15838	5218	4373	6247
Scenario 2	15695	5149	4342	6203
Scenario 3	16956	6020	4503	6433
Scenario 4	17235	6176	4554	6505
Scenario 5	15880	5174	4409	6298

used as an index of the erosive power of the flow. The work index values for the existing conditions flow scenario (scenario 1) were related to the existing condition of the banks that were based on field observations and other available information which define the variation along the study reach. The work index values for the alternative flow scenarios were then compared to the existing conditions results at specific locations to evaluate the potential change in erosive power associated with each scenario.

Work is defined here as the product of the stream power expended on the banks and the incremental time over which it is applied. Bank stream power is the product of the average main channel velocity (V_{ch}) and the shear stress acting on the bank (τ_b). For a given flood event, the total work at a given bank location can be determined by integrating the bank stream power over the entire hydrograph:

$$W = \quad (V_{ch} \ \tau_b) \quad t \tag{4.6}$$

where W is the total work performed at a specified bank location and dt is the incremental time. The bank shear stress is computed as:

$$\tau_b = K_b \qquad h \quad f \tag{4.7}$$

where γ is the unit weight of water (62.4 lb/ft^3), d_h is the hydraulic depth, S_f is the energy slope, and K_b is a factor that accounts for the effect of channel curvature on the shear stress acting on the outside of a channel bend. The hydraulic depth is the ratio of cross-sectional area of the main channel (A_{ch}) and the flow top width in the main channel (EW). K_b depends upon the ratio of the radius of curvature to the channel top width and is determined using the relationship given in **Figure 4.49** (from SCS 1977). Equations 4.6 and 4.7 can be solved for a given event by discretizing the inflow hydrographs (and stages) into a series of time steps determining the main channel hydraulic conditions (V_{ch}, A_{ch}, EW, d_h, S_f) and integrating the results of the calculations at each time step over the duration of the event.

Hydrographs were provided by the COE for the 2-, 3-, 5-, 10-, 25-, 50-, 100-, and 200-year events for each flow scenario. Work index values were computed for each of these events to provide information with which to evaluate the relative effects of the more frequent, as well as, the infrequent high flow events. The integrated effect of all of the storm events was considered by computing a weighted average work index using Equation 4.5 with the work index value for each storm substituted for the sediment yield as follows:

$$W = 0.0075W_{200} + 0.0075W_{100} + 0.015W_{50} + 0.04W_{25} + 0.0 \quad W_{10} + 0.117W_5 + 0.15W_3 + 0.333W_2 \tag{4.8}$$

where W_i is the work index value for each return period event and W_a is the weighted average value. W_a can be interpreted as the average annual work index associated with storm events and the individual terms in Equation 4.8 represent the annual contribution of each event to the total.

In addition to the storm events, work index values were also computed by integrating the annual flow duration curve (Figure 4.16, post Folsom Dam). These results demonstrate the cumulative effects on an average annual basis of all of the recorded flows in the study reach since closure of Folsom Dam in 1955.

A computer model was developed using the theory presented in Equations 4.6 and 4.7 to perform the calculations. The model is linked with the HEC-2 program to determine the hydraulic conditions for each time step. Input to the model consists of an HEC-2 input file set up for a single water surface profile, digitized hydrographs for the upstream inflow to the study reach (Folsom release), NEMDC flow, and downstream American River stage, and cross sections and the radius of curvature for critical locations where bank work calculations are desired. **Table 4.16** summarizes the cross sections used to evaluate conditions at key sites along the study reach as identified in the geomorphic analysis. - The table also summarizes the radius of curvature of the bend at each location determined from available mapping.

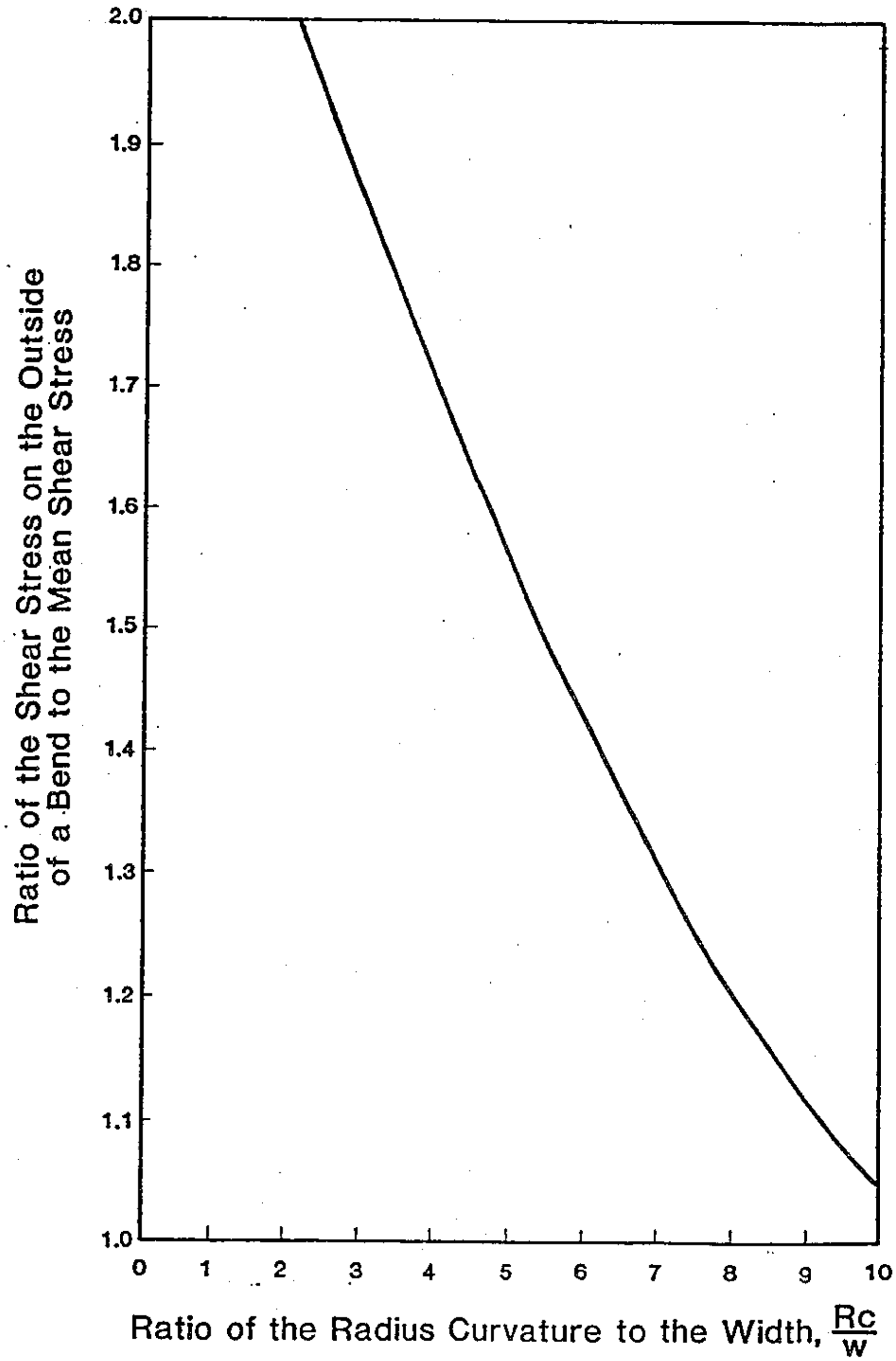


Figure 4.49 Increase in shear stress on outside of a bend.

Table 4.16. Summary of Locations considered in Bank Work Analysis.

River Mile*	Radius of Curvature (ft)	Main Channel Width (ft)	R_c/W	Existing Protection	Bank Location
0.161	1,900	541	3.51	no	right
0.474	straight	456	N/A	no	right
0.756	straight	456	N/A	no	right
2.174	straight	611	N/A	no	left
2.193	straight	607	N/A	no	left
2.504	2,800	474	5.91	yes	left
2.766	2,800	555	5.05	yes	left
3.256	straight	485	N/A	no	right
3.744	1,400	693	2.02	no	right
3.763	1,400	693	2.02	no	right
4.230	straight	679	N/A	yes	left
4.948	1,800	650	2.77	yes	left
5.230	1,700	810	2.1	no	right
5.497	1,70	1,431	1.19	no	right
6.291	straight	543	N/A	yes	left
6.528	straight	494	N/A	yes	left
6.778	straight	441	N/A	yes	left
7.294	2,200	393	5.6	yes	left
8.513	straight	341	N/A	no	right
8.786	straight	420	N/A	yes	right
9.205	straight	793	N/A	no	right
10.462	straight	441	N/A	no	left
12.036	straight	434	N/A	no	left
12.290	straight	328	N/A	no	left
16.345	1,300	1,016	1.28	no	left
17.776	1,700	367	4.63	no	right
18.775	1,300	373	3.49	no	left
19.320	straight	386	N/A	no	left
20.606	2,900	519	5.59	no	right
21.138	straight	666	N/A	no	left

*Corresponds to HEC-2 cross section closest to site.

4.5.2. Results

Existing Conditions

Figure 4.50 is a plot of the work index values derived from integration of the annual flow duration curve at each of the sites considered in the analysis. The two lines shown in the figure represent the results based on the upper and lower bounding rating curves at the mouth of the American River used as the downstream control for the backwater analysis. Sites at which there is existing bank protection are also indicated in the figure. It is important to note that the computations were performed at available cross sections in the HEC-2 model. As previously described, these cross section were selected to represent sites that were identified during the field reconnaissance and geomorphic analysis as having existing erosion problems or nearby locations where erosion protection measures have been installed. The work index values at a particular site should be considered to represent typical conditions along the indicated bank in the vicinity of the cross section.

The difference in results between the upper and lower bounding rating curves appears to be relatively small for the range of flows in the annual flow duration curve, which diminish to essentially zero approximately 5 mi upstream of the mouth. The results for the lower bounding curve tend to be higher than the upper bounding curve because of the lower water surface elevation at the mouth.

Comparison of the results along the reach indicates that the work applied to the channel banks in approximately 5 mi downstream of the study reach (downstream of the bend between Business 80 and H Street) is relatively low compared to upstream reaches. Work in the bend between RM 5 and RM 6 is high, which results from a combination of the channel curvature and local steepening of the channel slope. Values in the straight reach between the bend and just downstream of Watt Avenue (RM 9.25) are generally less than those through the bend, but they are higher than in the downstream 5 mi. The site at approximately RM 10.5 has a relatively high work index value due to a local contraction of the flows by the erosion resistant bank material along the left bank at that location. Sites upstream of the end of the left bank levee (RM 11) generally have higher values due primarily to the effects of the steeper gradient in this reach. The exception occurs at RM 16.35 where flows across the bar on the right side of the channel (inside of the bend) relieve the stress on the left bank.

Sites identified without existing bank protection in the downstream approximately 5 mi of the reach are characterized by either intermittent erosion or are the result of local conditions that cause a local increase in the stress on the banks. For example, sites at RM 3.74 and RM 3.76, which have relatively high work index values, are in the vicinity of the Southern Pacific Railroad (SPRR) Bridge, and the site at RM 0.16 is at the relatively sharp bend in the right bank just upstream of the confluence with the Sacramento River. With the exception of these three sites, the work index values for this reach are less than 100 and average in the range of 50 to 60 for the upper and lower bounding rating curves. As discussed in the geomorphic analysis, this reach appears to be relatively stable, laterally.

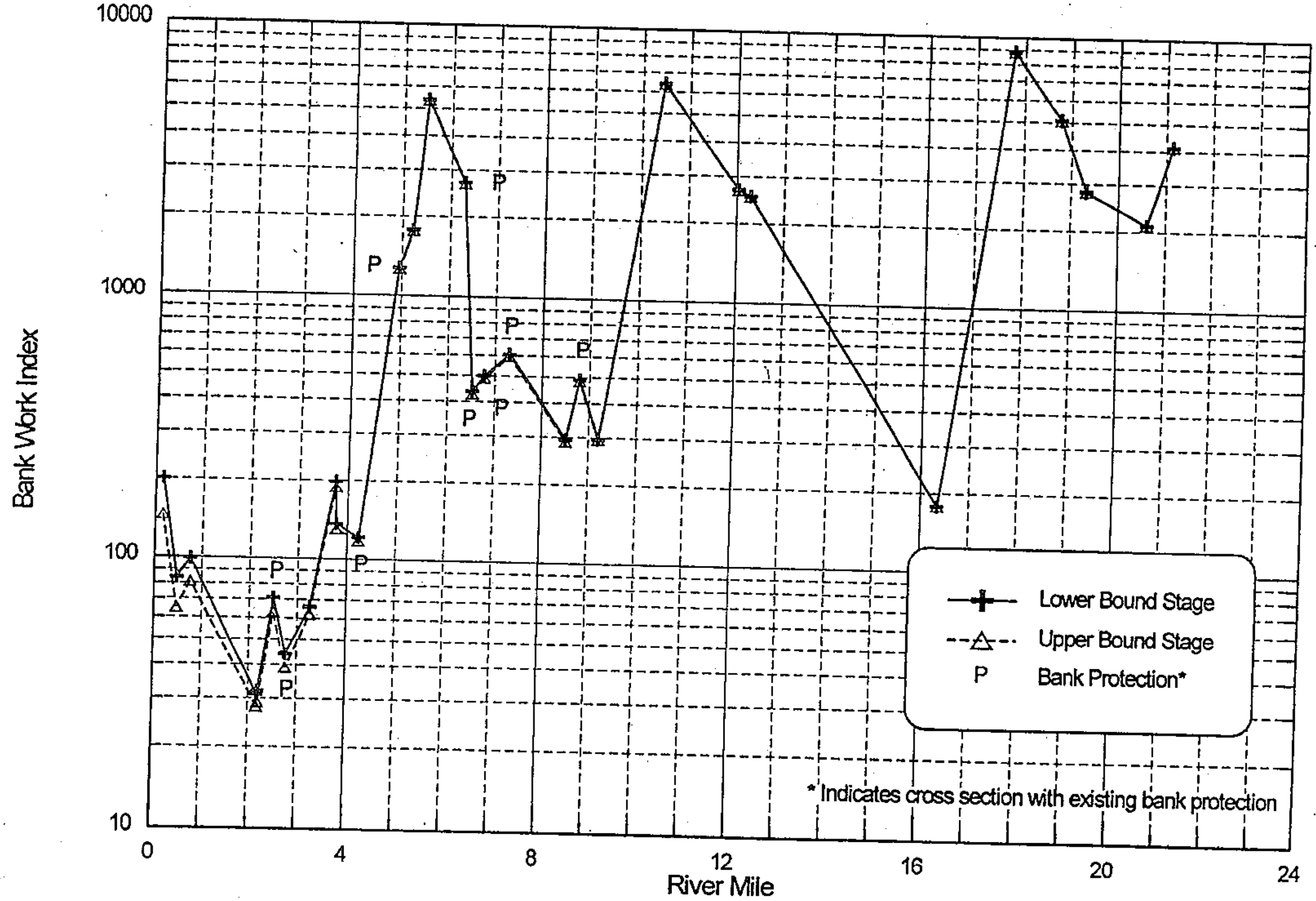


Figure 4.50. Work index values based on integration of the annual flow duration curve (lower bound rating curve).

Locations between RM 5 and the upstream end of the left bank levee (RM 11) have significantly higher work index values. Most of these locations have existing bank protection, presumably because the higher stress on the banks has created bank erosion problems that required corrective action in the past. Locations in this reach that do not have existing bank protection (outside of the bend between RM 5 and RM 6, right bank at RM between 8.5 and 9.0, and just downstream of Watt Avenue RM 9.2) have intermittent bank protection in various states of repair. They are currently eroding and are probably in need of corrective action. Most of the locations between the upstream of the end of the left bank levee and Nimbus dam with high work index values generally have erosion resistant Pleistocene-age material in the bank.

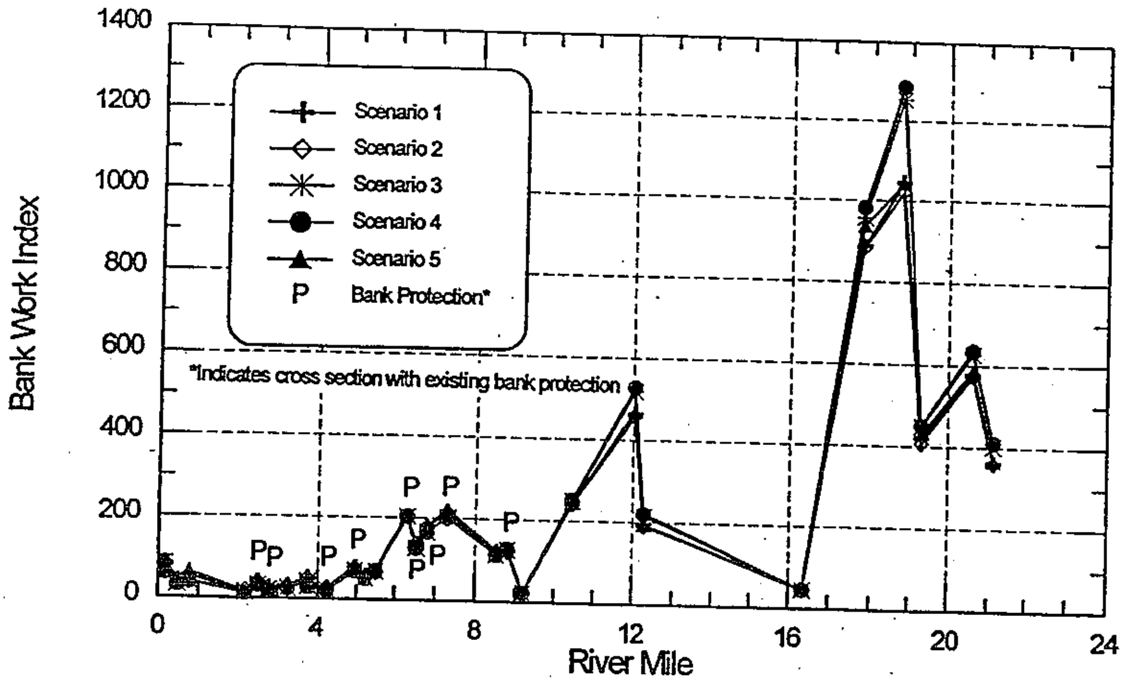
Figure 4.51a shows the weighted average work index values based on the 2-, 3-, 5-, 10-, 25-, 50-, 100-, and 200-year floods for all of the flow scenarios. Hydraulic computations used to derive these values were based on the lower bounding rating curve for the downstream control. These results indicate the same general trend as those based on integration of the flow duration curve with the exception that conditions in the bend between RM 5 and RM 6 do not appear to be as severe relative to the other locations considered in the analysis. The reduction in relative magnitude of the work index values at this location is probably the result of flow across the bar on the inside of the bend, which widens the effective flow area and relieves the pressure on the right bank under high flow conditions. Figure 4.51b shows the values plotted for the reach up to the end of the left bank levee at an expanded scale. The weighted average work index values for the relatively stable reach downstream of the bend at RM 5 are less than about 50 for the existing conditions flow scenario (scenario 1), which is quite low compared to upstream reaches.

Based on the results shown in Figures 4.50 and 4.51, work index values of about 100 based on integration of the annual flow duration curve, and 50 based on the weighted average of the various return period flood events, provide reasonable threshold values with which to identify potential lateral stability problems at unprotected sites.

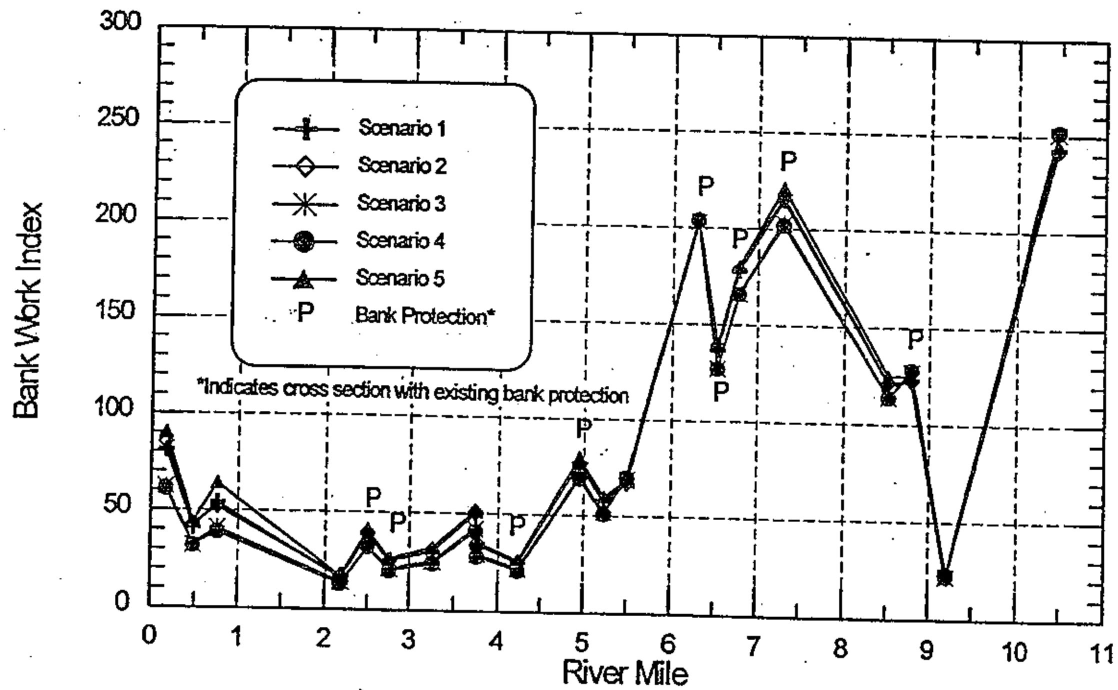
Results of the work index computation for the 100-year flood for each of the five flow scenarios are shown in **Figure 4.52a and 4.52b**. These results are based on the routed downstream stage hydrographs provided by the COE. Figures 4.51 and 4.52 show that the general trend in work index values along the study reach. It is the same for all five flow scenarios for both weighted average and 100-year flood conditions. Additionally, the figures indicate that the differences among the various scenarios are relatively small in most locations. With the exception of the site in the vicinity of cross section (and RM) 0.76, none of the alternative flow scenarios (scenarios 2 through 5) increase the weighted average work index above the threshold value of 50, which were discussed in the previous section. At RM 0.76 scenario 5 appears to increase the value from about 50 to about 60.

Alternative Flow Scenarios

To facilitate comparison of the results among scenarios, the percent difference in weighted average and 100-year storm work index values compared to Scenario 1 (existing conditions) was computed based on the lower bounding downstream rating curve **(Figures 4.53 and 4.54)**. Simplified versions of Figures 4.53 and 4.54 7.6 are

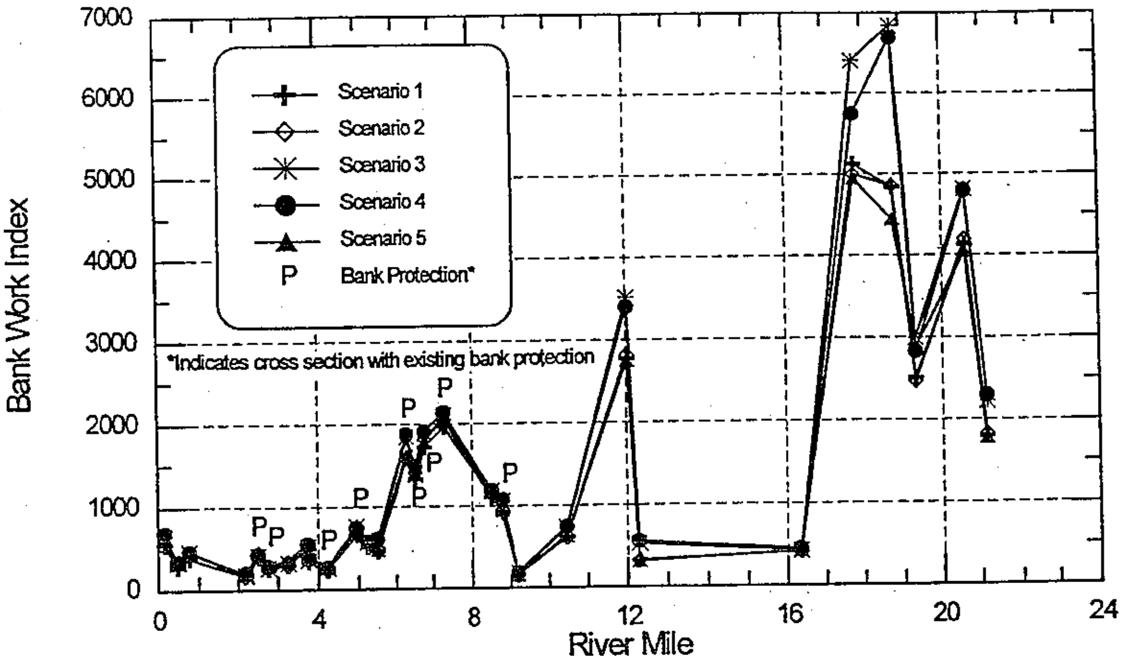


(a) Entire study reach

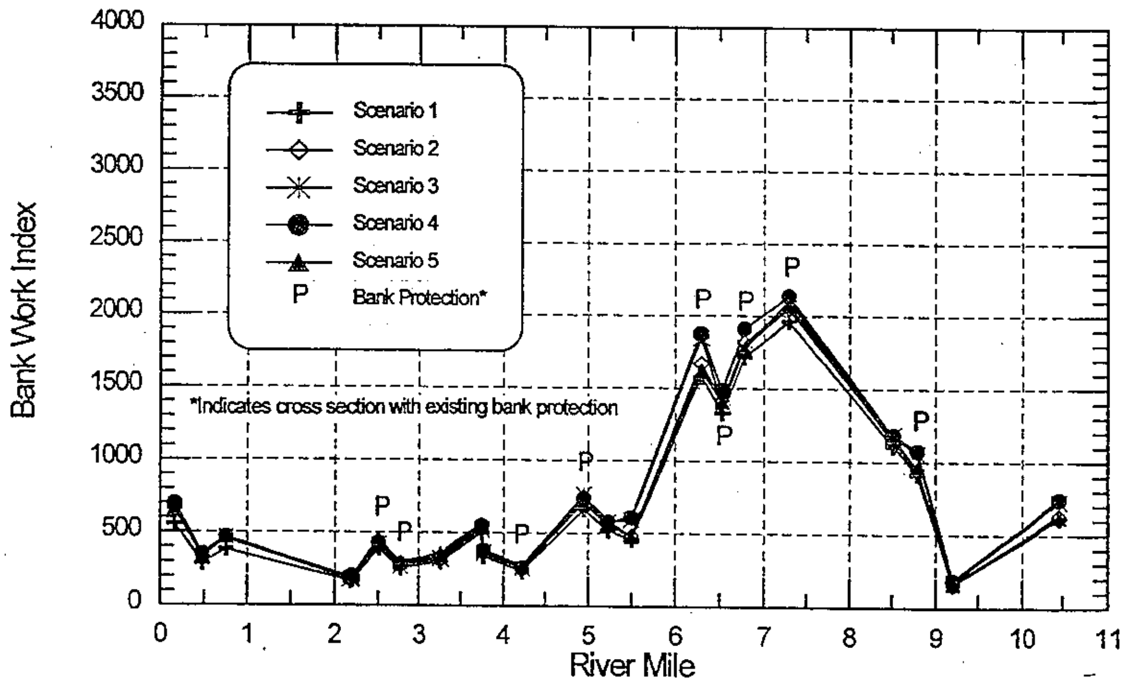


(b) Downstream eleven miles

Figure 4.51. Weighted average annual work index values based on the 2-, 3-, 5-, 10-, 25-, 50-, 100-, and 200-year floods for the five flow scenarios (lower bound rating curve).



(a) Entire study reach



(b) Downstream eleven miles

Figure 4.52. Work index values for the 10-year flood for the five flow scenarios (COE routed downstream stage hydrograph).

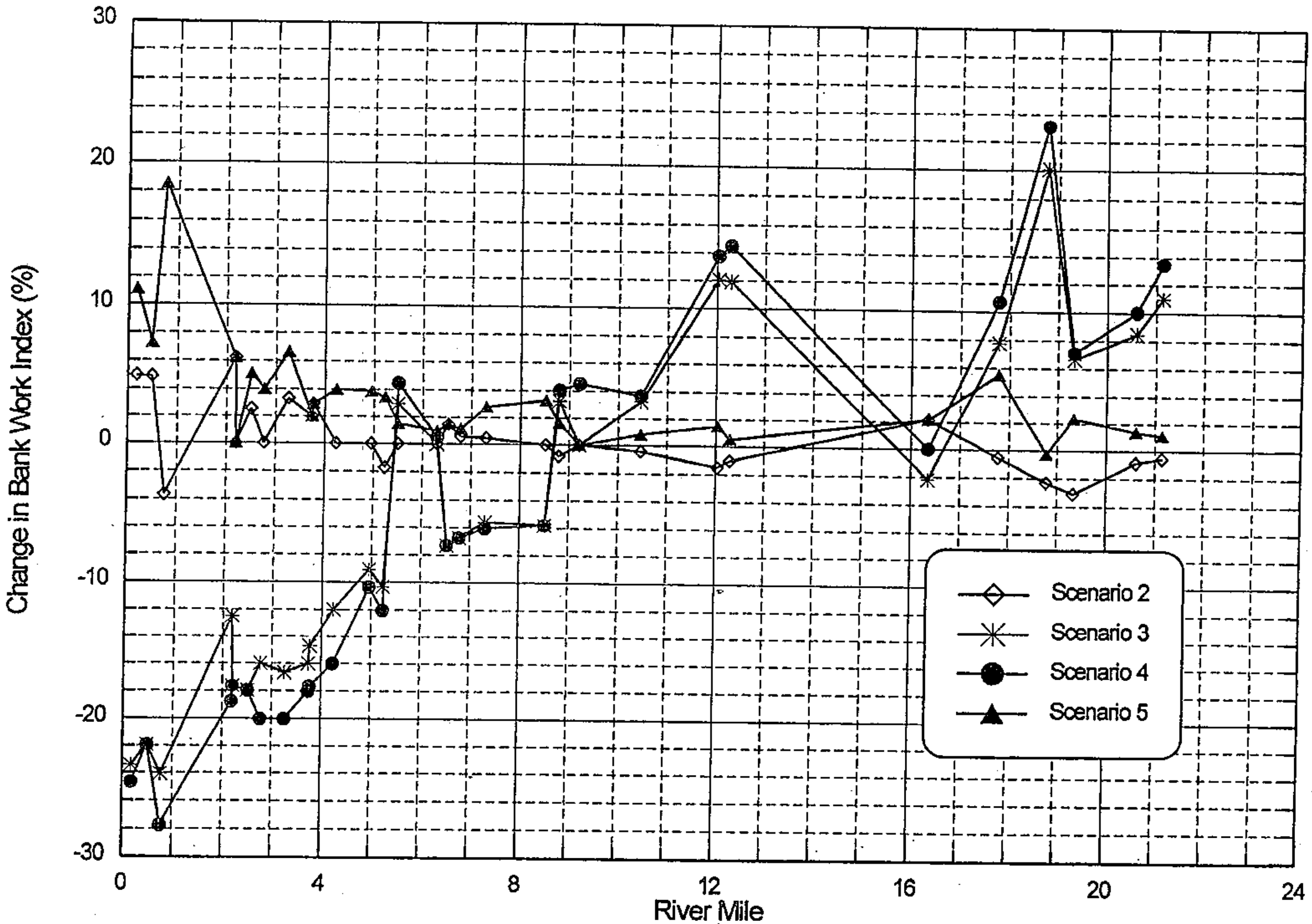


Figure 4.53. Percent difference in weighted average annual work index for the four alternative flow scenarios, compared to existing conditions (Scenario 1), using lower bound rating curve.

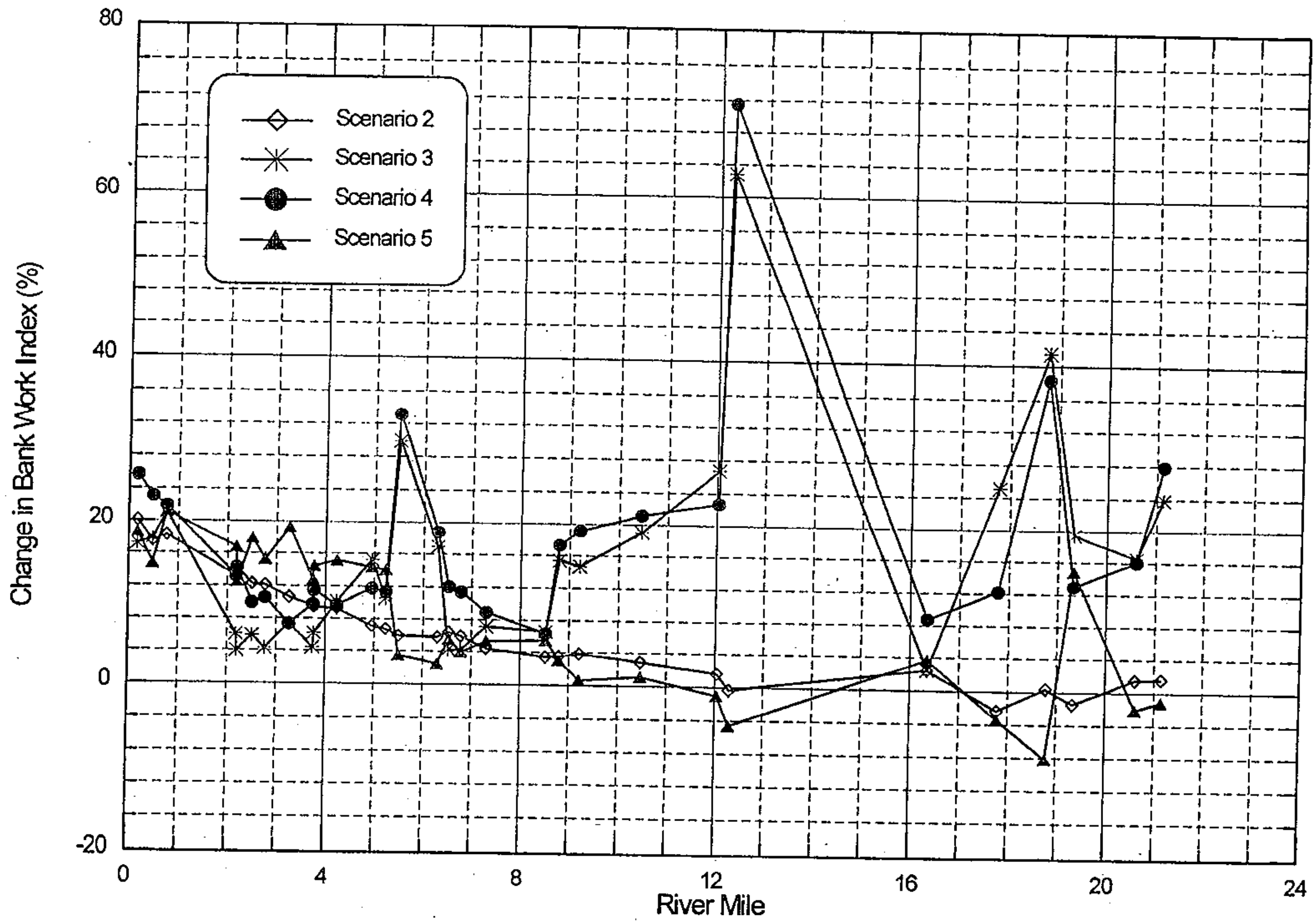


Figure 4.54. Percent difference in work index for 100-year flood for the four alternative flow scenarios, compared to existing conditions (Scenario 1), using COE routed downstream stage hydrograph.

shown in **Figures 4.55 and 4.56** to conceptually illustrate the primary trends evident in the more complex plots. As shown in these figures, the weighted average work index values for Scenario 2 (400,000 ac-ft storage, 145,000 cfs objective release) are nearly the same as for existing conditions. This occurs because the storm events that make the largest contribution to the weighted index values (5-, 10-, and 25-year) are nearly the same for both scenarios. Within the reach between the mouth and the upstream end of the left bank levee (RM 11.0), scenario 3 (590,000 ac-ft storage, 115,000 cfs objective release) and scenario 4 (650,000 ac-ft storage, 180,000 cfs objective release) appear to reduce the weighted average values by varying amounts from 20 to 30 percent near the mouth tapering to less than 10 percent upstream of H Street. These two scenarios appear to increase the weighted average values in the reaches upstream of the levees by amounts varying up to about 20 percent. For scenario 5 ("Recommended Project", Dry Dam), the weighted average values are approximately 10 percent higher than existing conditions downstream of the 12th Street bridge (RM 2), which reduce to about 5 percent higher between the 12th Street bridge and the upstream end of the left bank levee, and less than 5 percent higher from that point to upstream end of the study reach.

The percent difference for the 100-year flood among the four alternative flow scenarios and existing conditions is similar in the downstream approximately 5 mi of the study reach (with the difference decreasing from about 20 percent higher near the mouth to about 10 to 15 percent higher near RM 5). Upstream of that point, the differences for Scenarios 3 and 4 generally average about 20 to 30 percent higher; while Scenarios 2 and 5 continue to be nearly the same as existing conditions. In fact, Scenario 5 is actually somewhat less than existing conditions in most of the upstream 10 mi of the study reach.

The factors contributing to the above differences are complex, relating to the magnitude, duration, and timing of the flows that occur within each hydrograph for each scenario. In spite of these complexities, the following general conclusions can be drawn regarding the potential effects of the four alternative scenarios on the lateral stability of the Lower American River channel:

1. The differences among any of the four alternative scenarios and existing conditions in the leveed reach are relatively small (generally less than 10 percent for the weighted average values and less than 20 percent for the 100-year storm). Except for the site near RM 0.76, the differences are insufficient to increase the work index values above the threshold value of about 50 for weighted average conditions.

2. For weighted average conditions, the scenarios involving increased storage in Folsom Reservoir (scenarios 3 and 4) appear to reduce the potential for bank instability in the downstream approximately 5 mi of the reach. This is probably because these scenarios reduce the magnitude of the peak flow for the more frequent floods that have the largest contribution to the overall work index value.

3. Scenarios 3 and 4 appear to increase the potential for lateral instability in the reaches upstream of the project levees for both weighted average and 100-year flood conditions. This is most likely due to the increased duration of high to moderate, in-bank flows compared to existing conditions as the additional stored water is released from Folsom Reservoir.

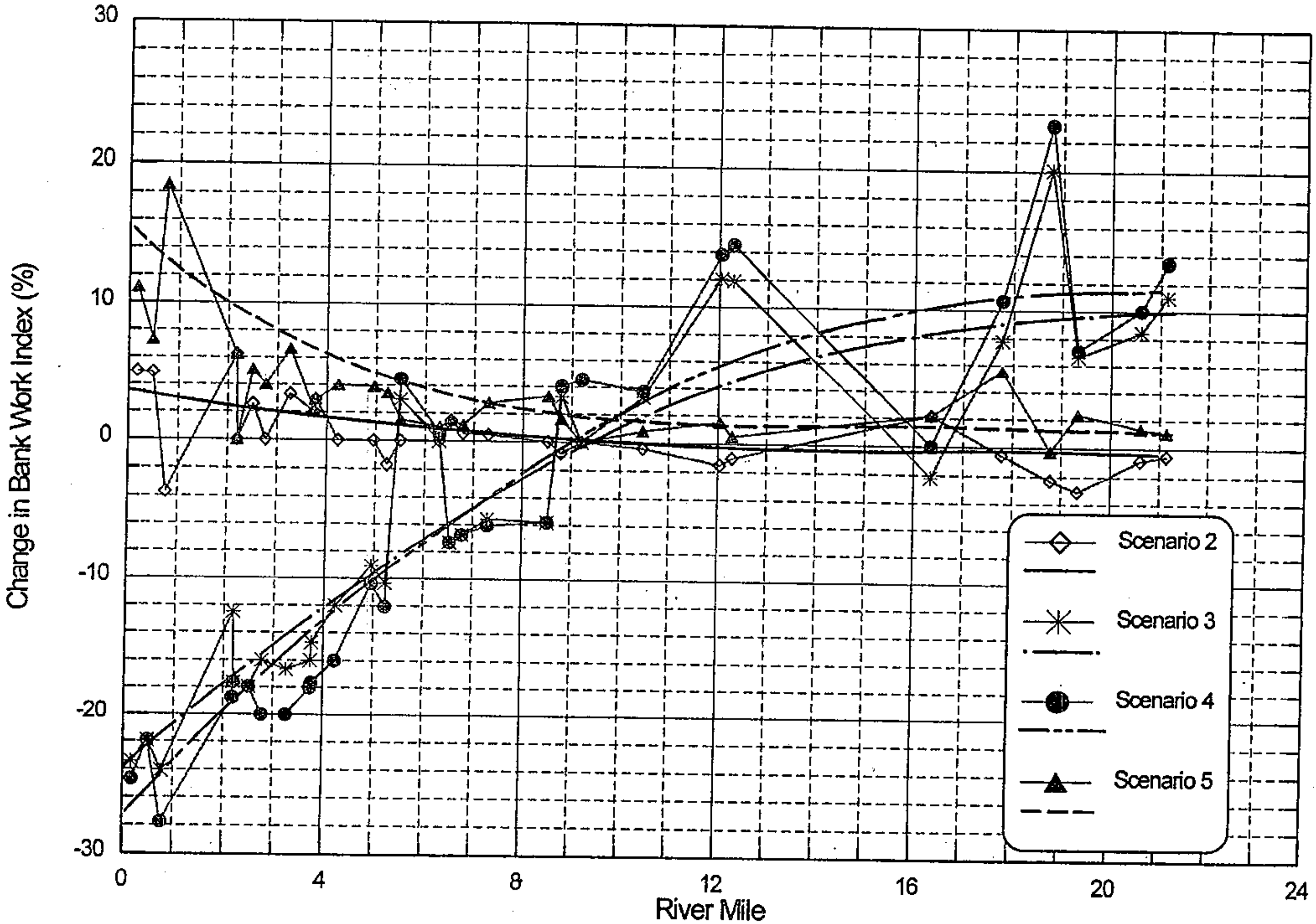


Figure 4.55. Percent difference in weighted average annual work index for the four alternative flow scenarios, compared to existing conditions (Scenario 1), using lower bound rating curve.

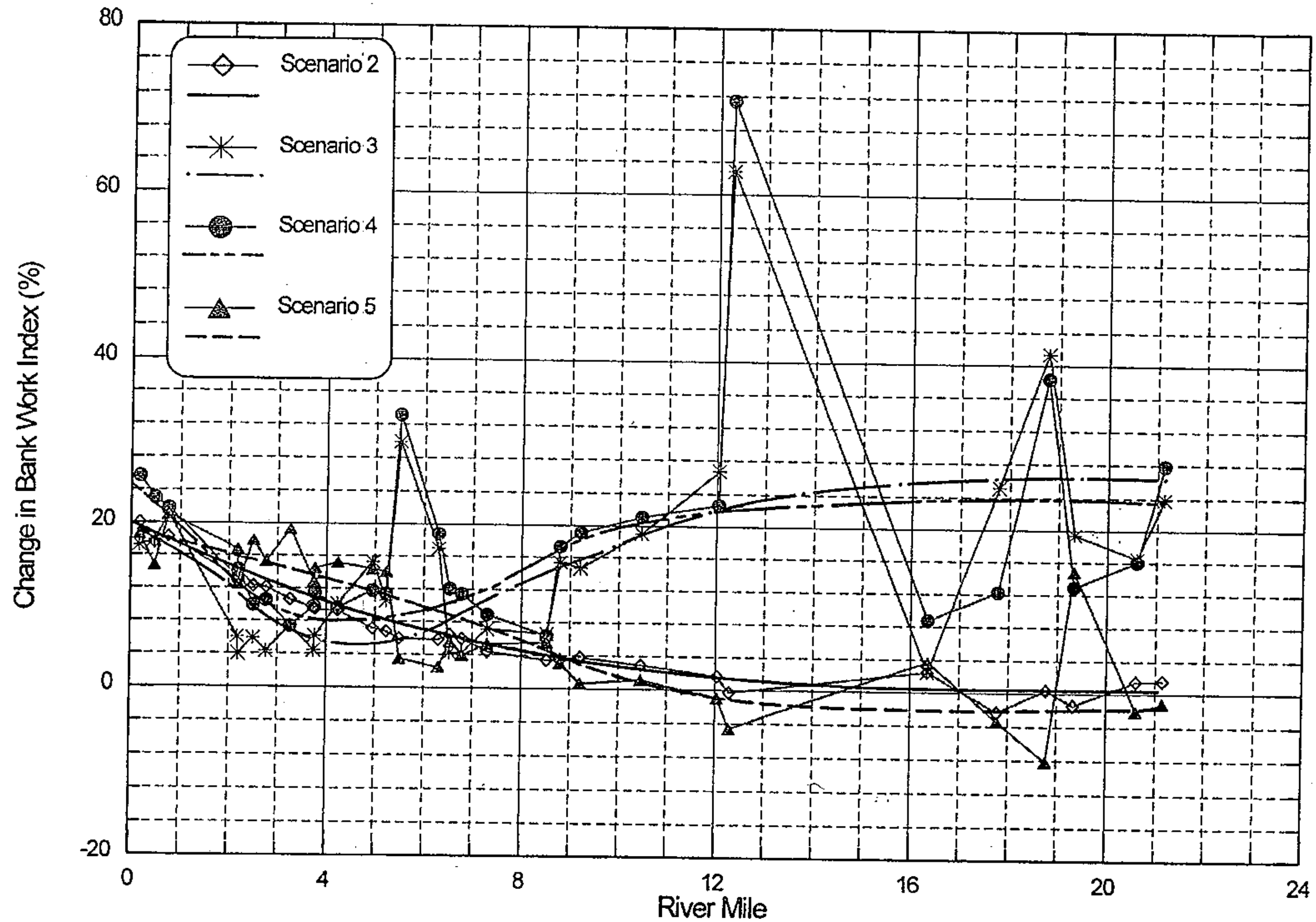


Figure 4.56. Percent difference in work index for 100-year flood for the four alternative flow scenarios, compared to existing conditions (Scenario 1), using COE routed downstream stage hydrograph.

4. The FEMA 100-year scenario (scenario 2) increases the potential for bank instability slightly in the downstream approximately 5 miles of the study reach for both weighted average and 100-year storm conditions. This reduces to approximately the same as existing conditions upstream of that point. The increase in the lower 5 miles is probably because under existing conditions, flows in excess of 180,000 cfs are released from Folsom Dam during the 100-year and larger floods, which cannot be contained within the project levees. For this scenario, the maximum release is 180,000 cfs. This, of course, implies that the levees would fail (or at least overtop) during the 100-year flood under existing conditions, which would have catastrophic consequences.

5. The Recommended Project, Dry Dam scenario (scenario 5) shows generally the same trend as scenario 2. The exception is that the increased stress in the lower 5 miles is more significant. Although the peak release from Folsom Dam is only 115,000 for this scenario, it occurs over a much longer period of time which increases the total work on the channel banks.

4.5.3. Identification of Bank Erosion Sites Based on SRBPP Criteria

Prioritization of bank protection requirements for the Sacramento River Bank Protection Project (SRBPP) has been based historically on direct erosion threat potential to the Sacramento River Flood Control Project (SRFCP) levees (WET 1991; Fischer et al. 1991). For the Sacramento River and its tributaries, excluding the Lower American River, site prioritization has been based on the projected rate of bank line retreat at actively eroding bank sites reaching a 30-foot wide buffer strip on the river side of the levee within a prescribed 10-year project execution period commencing in 1992 (WET 1991). Because of the highly urbanized nature of the lands outside the Lower American River levees, the width of the buffer strip on the river side of the levees was increased to 50 ft.

On the basis of these criteria and an assumed average rate of bank erosion of 4.8 ft/yr, it was determined that any eroding bank on the Lower American River that was within 77 ft of the 50-foot wide buffer would qualify for high priority status (WET 1991). More comprehensive bank line erosion data (MSA 1993; refer to Section 2.5.3) have not significantly altered the average rate. Based on the above criteria, four high priority sites were identified within the leveed reach of the Lower American River: RM 2.1L, RM 4.8L, RM 8.6R, and RM 10.3R (Table 2.3). The total length of eroding bank encompassed by the four sites is approximately 1,850 ft. Also identified was a total of about 1,500 ft of damaged revetment that will require some rehabilitation.

Bank work index values derived from integration of the flow duration curve were computed for all of the bank erosion sites (Figure 4.50) identified within the project reach of the Lower American River (Table 2.3). The bank work index values for each of the previously identified priority sites are: 2.1L (29), 4.8L (1300), 8.6R (300), and 10.3R (300). With the exception of RM 2.1L, the other priority sites all exceed the threshold value (100) of the bank work index. RM 2.1L is affected by backwater from the downstream bridge and the erosion is probably related to local effects. In fact, both sites at RM 2.1L and RM 4.8L are gaps in existing rock riprap revetments. From a practical point of view, bearing in mind that levee threat is the reason for the site prioritization, rock revetment is the only solution to the erosion threat at these sites. Although other forms of bank protection may work at these sites, the proximity to the levees does not allow for any margin of error.

Bank work index values confirm that the prioritized sites at RM 8.6R and 10.4L should be protected to prevent threat to the levees. Both sites are located in split-flow reaches related to the capture of old sand and gravel mining pits. Full bank rock revetment could be used at either site because of the proximity of the levee, but as an alternative rock dikes could also be utilized.

As shown on Table 2.3 and the accompanying set of plates, there are other sites of bank erosion within the leveed reach, but these sites do not pose a threat to the levees within the terms of the criteria that have been established for the SRBPP. Upstream of the project levees (RM 14R, RM 12L), bank erosion sites were also identified, and many of the sites have extremely high bank work index values (Figure 4.50). However, many of the eroding sites are composed of Pleistocene-age sediments that tend to be consolidated, cemented, and relatively erosion resistant.

4.5.4. Identification of Bank Erosion Sites Based on LAR Criteria

Under a modification to the contract (Modification P000) a LAR-specific set of criteria for bank erosion sites were developed.

For the Lower American River a number of different criteria for identifying bank/levee protection requirements have been advanced previously. These include the SACBANK criteria utilized by WET (1991), which incorporated a 50-foot wide buffer strip riverward of the toe of the levee, an average (1969 to 1986) lateral migration rate of 4.8 ft/year and a time to construct the project of 10 years. The buffer strip was incorporated into the criteria because of uncertainty associated with quantifying lateral migration rates (range of 1.4 to 13.9 ft/year) because these rates are heavily dependent on the occurrence of significant morphogenetic flood events. Further, the composition of the channel bank materials is spatially varied and extremely complex and made more so by the effects of upstream hydraulic and dredge mining. Because of the age of the data base (1986 aerial photography) and the date of the analysis (1993) a further 7 years was added to the time base which resulted in a conclusion that any site located within 82 feet of the 50-foot wide buffer would qualify as a high priority site.

Criteria for site selection and prioritization were also advanced by the Sacramento District in a memorandum entitled: "Draft Bank Protection Criteria for Lower American River and Influencing Reaches of Sacramento River" (Dated 23 February, 1994). Three criteria were identified as follows:

1. Criteria 1. Bank has already or is expected to erode into the levee section (IV:3H) levee side slope projected to thalweg elevation) prior to construction of the American River Project.

2. Criteria 2. Bank is within 150 feet of the levee section (Note that no time frame was addressed).

3. Criteria 3. Protection provided to protect valuable environmental resources (Note that no definition or quantification of resource values was made).

SAFCA proposed three classes (criteria) in an Agenda Item (D) for the Lower American River Task Force Meeting on March 15, 1994, as follows:

1. Class 1. The threat to the levee or other infrastructure is imminent requiring immediate action.

2. Class 2. There is a threat to the levee or other infrastructure in the near term requiring immediate action unless there is clear evidence that the bank is stable (Note that no time frame was specified).

3. Class 3. There is a low probability of immediate damage to the levee or other infrastructure but there may be an immediate risk to other environmental or recreational resources possibly requiring immediate action (Note that there is no definition or quantification of resources or values).

When all of the proposed criteria are evaluated it is clear that immediate risk to the project levees is the highest priority, but quantification is problematic. Longer term threats to the levees or other significant infrastructure elements during the lifespan of the project are also of concern. Further it is also evident that there is concern for protection/preservation of biological resources within the American River corridor especially given the very constrained nature of the river. The somewhat static nature is a result of upstream dam construction and the very high incidence of Pleistocene age or older outcrop along the course of the lower river, both of which significantly limit the potential for supplying the sediment which is integral to the formation of hydro-geomorphic features that form the substrate for the riparian communities.

Existing levee and infrastructure protection measures along the LAR have experienced damage from a variety of sources including anthropogenic activity. Many of the river cobble revetments were installed in the late 1950s and have generally performed very well. However, damage to these revetments, especially in the toe region (no toe trenches were utilized during construction), requires that they be repaired. Some of the bridge abutments along the LAR have been protected in the past and the revetments have also experienced damage and as such should be repaired. Under existing conditions, a number of bridge abutments have been identified that require protection primarily because of the uncertainty introduced by extremely complex hydraulic conditions generated by the bridge piers and abutments that are very difficult to model or predict (Refer to Federal Highway Administration Hydraulic Engineering Circular, HEC No. 18, 1991).

Criteria for Lower American River

The following criteria (Types 1, 2, 3) for channel bank protection have been developed to encompass the above stated concerns. With the exception of the environmental values, an attempt has been made to provide quantitative and repeatable criteria that are based on the existing knowledge of the dynamics of the LAR. The following assumptions have been used in development of these criteria:

1. The average bank height along the LAR project reach is about 15 feet and therefore projection of the levee side slope (IV:3H) results in a levee section that projects 50 feet riverward of the toe of the levee. Invasion by the river of the levee section is an unacceptable condition.

2. The average rate of lateral migration of the river is 5 ft/year.

3. Immediate threat sites will be constructed within 2 years (Refer to letter from The Reclamation Board to District Engineer, Sacramento District COE, dated 8 July, 1994).

4. The Lower American River Project will be constructed within 12 years.

5. The project life for installed revetment on the Lower American River is 50 years.

6. Damaged existing revetments and other infrastructure protection measures require immediate repair (See No. 3).

7. The width of the environmental resources to be protected will be sufficient that protection of the resource will not result in degradation of the resource value.

Type 1. Require Protection Within 2 to 12 years

1.a. Immediate requirement for construction of new protection because the river has already, or will within a 2-year period, erode into the levee section. All sites within 60 feet of the levee toe are included in this category.

1.b. New protection will be required prior to construction of the Lower American River Project (12 years) because the river will have invaded the levee section within that time period. All sites between 60 and 110 feet of the toe of the levee are included in this category.

1.c. Immediate repair of existing revetments that have been damaged since original emplacement.

1.d. Immediate repair of existing bridge abutment protection that has been damaged since original emplacement.

1.e. Immediate protection of identified currently unprotected bridge abutments.

Type 2. Require Protection Within 50 years

New protection required within the project life of the Lower American River Project because the river will have invaded the levee section. Sites between 110 and 300 feet of the levee toe are included in this category.

Type 3. Protection of Environmental Resources

New protection required primarily for stands of riparian vegetation that have sufficient width such that loss of vegetation consequent on construction of the protection will not degrade the resource value significantly. A key element of this category should be the inability of the resource to be regenerated by lateral migration of the channel because of floodplain width constraints imposed by the distance between the levees. No assignment of a timeframe has been made for this category. It could be assumed that further loss of the resource is unacceptable in which case Type 3 sites would fall into an immediate action category. Average lateral migration rates can be used to assess the future rate of resource loss.

List of Bank/Levee Protection Sites

Identification of a list of bank and levee protection sites was the objective of this work item. Sites identified by both the COE and SAFCA were meshed together in a memorandum by CESPK-ED-D, dated 25 June, 1994, entitled "Comments on SAFCA and COE Identified Bank Protection Sites on Lower American River Main Channel Banks and Project Levees." This list of sites was reviewed on the basis of field inspection observations, knowledge of the dynamics of the river, previous geomorphic and sediment engineering analyses of the LAR (WET 1991; RCE/Ayres 1993), and the criteria listed above.

Bank Protection Sites

Bank work index values that incorporate the planform, longitudinal and cross section characteristics of the channel were determined for all identified bank erosion and revetment repair sites for the 100-year event for each of the 5 LAR flow scenarios utilizing the COE routed downstream stages (Refer to Section 4.1). Since the values of the bank work index for each of the flow scenarios did not vary much, an average value was determined for each of the investigated sites. Previous analyses of work index values for the 100-year event for the 5 flow scenarios (Figure 4.52a, b) indicated that previously revetted sites had values > 300 and this was then considered to represent a threshold value. With the exception of the two sites at RM 2.1L and 2.2L where bank erosion is governed by local factors (bridges), the remainder of the sites investigated have work index values in excess of 400.

Utilizing the criteria developed above, the sites along the LAR were classified into the three principal Types and appropriate sub-types. Within the 2-year period (immediate) Types 1.a. (1,400 ft) and 1.e. (2,500 ft) constitute new construction and Types 1.c. (5,000 ft) and 1.d. (1,100 ft) involve repair of existing sites. Within the time frame of execution of the LAR project (12 years) 8,970 ft have been identified as Type 1.b. sites. During the life of the LAR a further 5,400 ft have been identified as Type 2 sites. Type 3 sites encompass 4,100 ft but no time frame other than within the life of the LAR has been assigned. The pertinent data and information for each of the identified sites are provided in **Table 4.17**.

Also included in Table 4.17 are recommendations for bank protection methods for each of the sites. Recent conceptual methods of bank protection developed for SAFCA have yet to be fully constructed and tested. Concerns regarding rock size and gradation, constructability, stage - discharge - duration and frequency relations for the proposed berms, and potential liability issues associated with incorporation of large woody debris must all be addressed before firm recommendations for usage can be made. Regardless of the site specific characteristics of any identified site and regardless of the classification of the site, all bank protection methods considered must at a minimum incorporate a rock-based toe (cobble or quarry rock).

Repair of the toe of existing cobble revetments will generally involve the use of quarry rock. However, it has been suggested that cobbles be utilized for repair at RM 0.20L and that steps and pathways be incorporated to permit recreational access to the river. Repair of bridge abutments will most probably require the use of quarry rock, as will construction of new bridge abutment protection. Because of greater distance from the levee and therefore lower risk it may be possible to utilize more unconventional bank protection methods in Type 1.b., Type 2, and Type 3 sites.

Table 4.17. Lower American River Bank Protection Sites.

Site (River Mile)	Distance to Levee Toe (feet)	Bank Work Index	Bank Protection Needs by Classification Type & Sub-type: Type 1a Immediate (Channel Bank) (feet)	Type 1b Within 12 years (Channel Bank) (feet)	Type 1c Repair (Exist Channel Bank Prot) (feet)	Type 1d Repair (Exist Bridge Abutmt Prot) (feet)	Type 1e New Bridge Abutment Protection (feet)	Type 2 New Bank Protection Project Life (feet)	Type 3 Environmental Resource Protect (feet)	Recommended Bank Protection Methods	Site Location	Engineering Justification
0.00L				1200						Fullbank rock riprap	TLB at mouth of LAR	Protect against unusal hydrodynamic conditions in confluence area.
0.15R						100				rock riprap or articulated concrete mattress tie into existing rock riprap on Sacramento River	Jiboom Street Bridge (TRB Abutment)	Repair existing protection
0.15L							100			rock riprap	Jiboom Street Bridge (TLB Abutment)	Protect bridge abutment
0.20R							200			rock rirap	I-5 Bridge Abutment (TRB Abutment)	Protect bridge abutment/pier
0.20L						200				cobble riprap	I-5 Bridge Abutment (TLB Abutment)	Repair existing cobble site
2.00L						200				rock riprap	12th & 16th Street Bridges (TLB Abutments)	Repair existing protection
2.00R							200			rock riprap	12th & 16th Street Bridges (TRB Abutments)	Protect bridge abutment/pier
2.10L	60	183	1000							fullbank rock riprap no waterside berm	Between 16th Street & Bicycle Bridges	Protect project levee
2.20L	60	195	400				100			rock riprap	Bicycle Bridge (TLB Abutment)	Protect bridge abutment/pier
2.20R							100			rock riprap	Bicycle Bridge (TRB Abutment)	Protect bridge abutment/pier
2.20L										fullbank rock riprap no waterside berm	Between Bicycle Bridge & UPRR Crossing	Protect project levee
2.30L							100			rock riprap	UPRR Bridge (TLB Abutment)	Protect bridge abutment/pier
2.30R							100			rock riprap	UPRR Bridge (TRB Abutment)	Protect bridge abutment/pier
2.30R	1750	595							1200	rock dikes to allow for curvilinear bankline SAFCA design with berm	Upstream of UPRR Bridge	Protect environmental resources
3.75R							100			rock riprap	SPRR Bridge (TRB Abutment)	Protect bridge abutment/pier
3.75L						100				rock riprap	SPRR Bridge (TLB Abutment)	Repair existing protection
3.75L	105	420		900						fullbank rock riprap with no enchroachment remove mid-channel bar and old bridge piers	Between SPRR and I-80 Bridge Crossings	Protect project levee
4.00R							200			rock riprap	I-80B Bridge (TRB Abutment)	Protect bridge abutment/pier
4.00L						200				rock riprap	I-80B Bridge (TLB Abutment)	Repair existing protection
4.40L	110	1824		2500						fullbank rock riprap SAFCA design w. berm -- transitions may be problematic	River Park Levee	Protect project levee
5.50R		524			200					rock riprap	Cal Expo Drainage Outfalls (two)	Repair existing protection

Table 4.17. Lower American River Bank Protection Sites. (continued)

Site (River Mile)	Distance to Levee Toe (feet)	Bank Work Index	Bank Protection Needs by Classification Type & Sub-type: Type 1a Immediate (Channel Bank) (feet)	Type 1b Within 12 years (Channel Bank) (feet)	Type 1c Repair (Exist Channel Bank Prot) (feet)	Type 1d Repair (Exist Bridge Abutmt Prot) (feet)	Type 1e New Bridge Abutment Protection (feet)	Type 2 New Bank Protection Project Life (feet)	Type 3 Environmental Resource Protect (feet)	Recommended Bank Protection Methods	Site Location	Engineering Justification
5.75R	530	492		1200						SAFCA design with berm rock dikes	Cal Expo Area	Protect project levee and power transmission towers
6.60L										rock riprap	H Street Bridge (TLB Abutment)	Protect bridge abutment/pier
6.60R							100			rock riprap	H Street Bridge (TRB Abutment)	Protect bridge abutment/pier
6.80L		2806			1500		100			rock riprap toe no berm on waterside	Sac State Levee	Repair existing cobble site
7.10L										rock riprap	Guy West Bridge (TLB Abutment)	Protect bridge abutment/pier
7.10L		2451			1500		100			rock riprap toe no waterside berm	Sac State Levee	Repair existing cobble site
7.35L (Restoration)										existing protection in place	Sac State Levee (TLB Berm)	Potential Riparian Vegetation Restoration Site
7.65R	180	839						500		SAFCA design - may have transition problems rock toe with upperbank vegetation	Campus Commons Levee	Protect project levee
7.75R							200			rock riprap	Howe Ave Bridge (TRB Abutment)	Protect bridge abutment/pier
7.75L							200			rock riprap	Howe Ave Bridge (TLB Abutment)	Protect bridge abutment/pier
7.75R	450	542							1100	SAFCA design with berm rock toe w/ upperbank vegetation, or rock dikes	Upstream of Howe Ave Br (TRB)	Protect environmental resources
8.00R	240	476						2500		SAFCA design with berm rock dikes or rock toe w/ upperbank vegetation	Upstream of Howe Ave Br (TRB)	Protect project levee
8.55R		1159			1800					rock riprap toe w/ waterside construction rock dikes with toe rock betweem dikes	Upstream of Howe Ave Br. (TRB)	Repair existing cobble site
8.90R (and/or Restoration)	300	411							1800	rock dikes or windrow revetment toe rock riprap	D/S Watt Ave Bridge (TRB)	Protect environmental resources; and/or potential restoration site
9.25R							150			rock riprap	Watt Ave Bridge (TRB Abutment)	Protect bridge abutment/pier
9.25L							150			rock riprap	Watt Ave Bridge (TLB Abutment)	Protect br abutm/pier
9.70L	70	1994		1530						SAFCA design with berm rock dikes with toe rock between dikes	U/S Watt Ave Bridge	Protect project levee
10.20L	70	1918		1640						SAFCA design with berm rock dikes with toe rock between dikes	U/S Watt Ave Bridge	Protect project levee
11.90R (Restoration)										rock dikes	Near Carmichael Treatment Plar	Potential restoration site
12.80R (Restoration)										rock dikes	Near Carmichael Treatment Plar	Potential restoration site
15.40L	150	2580						2400		rock dikes with rock toe between dikes SAFCA design with berm	Nr TLB Sewage Trtmt Plnt	Protect new project levee

Table 4.17. Lower American River Bank Protection Sites. (continued)

Site (River Mile)	Distance to Levee Toe (feet)	Bank Work Index	Bank Protection Needs by Classification Type & Sub-type							Recommended Bank Protection Methods	Site Location	Engineering Justification
			Type 1a Immediate (Channel Bank) (feet)	Type 1b Within 12 years (Channel Bank) (feet)	Type 1c -Repair- (Exist Channel Bank Prot) (feet)	Type 1d -Repair- (Exist Bridge Abutmt Prot) (feet)	Type 1e New Bridge Abutment Protection (feet)	Type 2 New Bank Protection Project Life (feet)	Type 3 Environmental Resource Protect (feet)			
20.15R							150			rock riprap	Sunrise Blvd Bridge (TRB Abutment)	Protect bridge abutment/pier
20.15L							150			rock riprap	Sunrise Blvd Bridge (TLB Abutment)	Protect bridge abutment/pier
22.80R						150				grout existing rock riprap	Hazel Ave Bridge (TRB Abutment)	Grout existing protection for higher than design flows
22.80L						150				grout existing rock riprap	Hazel Ave Bridge (TLB Abutment)	Grout existing protection for higher than design flows
TOTALS			**1400**	**8970**	**5000**	**1100**	**2500**	**5400**	**4100**			

The risk associated with levee failure at Type 1.a. sites dictates that conventional rock riprap be utilized at these sites. Both identified sites (RM 2.1 L and RM 2.2L) are located in reaches heavily influenced by bridge effects (12th, 16th Street Bridges, former UPRR Bridge and present UPRR Bridge) and represent gaps in existing fullbank rock revetments. Encroachments into the channel as a result of the addition of berms to the riverward side of the revetments would not be advisable in these reaches because of the possibility of increasing stress on the opposite bank, bridge piers and bridge abutments.

Bridge effects, primarily constriction of the cross section as a result of the presence of old bridge piers has caused deposition of a mid-channel bar between the SPRR bridge and the I-80 bridge. The mid-channel bar in turn is causing erosion of the left bank (RM 3.75L) which is threatening the project levee (Type 1.b. site). Removal of the old bridge piers and mid-channel bar would reduce the stress on the left bank and may well lower the category for this site to Type 2.

At RM 5.75 R, a Type 1.b. assignment was made not because of the proximity to the levee, but because power line transmission towers are located within the critical distance for this category.

Potential riparian vegetation restoration sites were located during field inspections and these are listed in Table 4.17. In the main, protection of these types of sites should be minimal since they do not meet any of the listed criteria. However, if the sites are of use for restorative purposes they should be protected. Rock dikes are probably the most effective means of protecting the sites. Reduction in elevation of many of these types of sites will be required since they are either terraces formed as a result of vertical accretion and subsequent degradation of the channel or are severely perturbed sites as a result of mining. Mine tailings are in general deficient in fine sediments required to hold moisture and promote plant growth. Materials removed from the terraces could be used to increase the fines content of the tailings.

Levee Slope Protection

Within the American River Watershed Investigation, Feasibility Analysis (COE 1991) a recommendation for approximately 37,000 lineal feet of levee slope protection was included in Table M-4-1. The recommended sites were field visited on June 23, 1994. Overbank flow velocities (HEC-2) were reviewed for the various flow scenarios. In combination, the information was used to assess the requirements for levee slope protection. **Table 4.18** summarizes the levee slope protection sites and the controlling factors at each site are described briefly in the following paragraphs. The listing of sites provided in Table 4.18 assumes that there are no geotechnical reasons requiring emplacement of levee slope protection.

RM 0.0L - Although flow velocities against the levee are expected to be low because of concurrent flows on the Sacramento River, there is the possibility of extreme local turbulence at the confluence and wind generated waves. Articulated concrete block protection, buried and grassed would achieve the desired degree of protection at this site. Burial of the blocks would reduce the possibility for man-made disturbance in an area of very heavy use.

Table 4.18. Lower American River Levee Slope Protection Sites.

Site (River Mile	Levee Protection Length (ft)	Recommended Bank Protection Methods	Site Location	Engineering Justification
0.00L	1,100	Buried concrete block revetment	TLB at mouth of LAR	Protect against local turbulence and wind generated waves at confluence
1.70	400	Rock riprap	Northgate Blvd. crossing of Natomas East Main Drain	Bridge abutment protection
1.90R	400	Rock riprap	400 LF U/S of Del Paso Road levee gate closure	Complex flow conditions caused by multiple bridge openings
6.40R	4,000	Rock riprap		Constricted river reach Right overbank velocities of 3.5 to 6.4 fps
7.20	3,500	Rock riprap		Constricted river reach High right overbank velocities up to 5 fps
TOTAL	9,400			

RM 1.7 - Rock riprap should be used to protect the bridge abutments where Northgate Blvd. crosses the Natomas East Main Drain.

RM 1.9R - The project levee slope requires rock riprap protection from the Del Paso Road levee gate closure upstream for a distance of about 400 feet because of very complex hydraulic conditions caused by multiple bridges.

RM 6.4R - This reach of the river is constricted and right overbank flow velocities for the various flow scenarios range from 3.5 to 6.4 ft/sec. For this reason the use of rock riprap on the levee slope is recommended.

RM 7.2R - This reach of the river is constricted and right overbank flow velocities for the various flow scenarios are relatively high and range up to 5 ft/sec. For this reason the use of rock riprap on the levee slope is recommended.

Based on the information contained in Table 4.18, it appears that there is a requirement for about 9,400 LF of levee slope protection within the LAR project reach.

4.5.5. Review of Bank Protection Measures

Many erosion control structures and techniques are available to the engineer for controlling the bank erosion that naturally occurs along alluvial streams. The literature is large concerning these measures; however, only a brief review will be presented of the various types of erosion control techniques. Diagrams are utilized to acquaint the reader with typical construction features.

Bank protection measures can be grouped into at least five categories: revetment, dikes, vegetation, alignment change, and bank drainage. Revetment includes the placing of stone or concrete on the channel bank to resist the erosive forces of stream flow. Dikes, commonly called groins or spurs, direct away or reduce erosive forces along the channel bank by diverting the stronger currents. Vegetation can be substituted in place of stone, concrete, steel, or other materials at some erosion sites; however, the success of vegetative measures depends upon survival of the vegetation and upon substrate stability. The vegetation vulnerability should be considered in site evaluation. In some cases, alignment changes are appropriate; however, these situations require careful consideration of the consequences.

In general, alignment changes should be considered only to maintain favorable alignments or to enhance prior alignments. Successful alignment change usually requires dike or revetment construction. Drainage of the floodplain directly into the channel can cause gullying along the bank and can result in the stream bank being saturated, which increases the likelihood of bank failure due to mass wasting. This is commonly exemplified where overbank flows re-enter the main river channels. Upper bank drainage should be directed away from the channel and should be collected into either a small channel tributary to the main stream or slope drains. Revetment, dike, and vegetative methods are discussed in the following sections. Various combinations of these methods can provide effective bank protection.

Conventional bank protection has involved some sort of structural measures to resist or reduce boundary shear. Shear resistance measures are called revetment. Shear reduction measures generally consist of some type of dike structure.

Revetments

Traditional methods of bank revetment involve placement of a facing of erosion-resistant material, such as stone riprap, concrete rubble, articulated concrete mattress, asphalt, vegetation, gabions, and others (Thackston and Sneed 1982). Newer forms include the following: used-tire matting, membrane/soil cement systems, chemical stabilization, honeycomb matrices, and cement-filled fabric bags. Each of these materials probably has performed satisfactorily at specific locations. Factors to consider include aesthetics, durability of the system materials, resistance of components to movement by the flow, flexibility of the system, and susceptibility to vandalism. For example, used tires banded together by steel bands have worked, but the banding material may corrode. The tires are not visually appealing; and if the band breaks, the tires are readily moved by flow.

For the above materials and techniques to provide successful erosion control for full bank revetment, the eroding bank must be graded to a stable angle and smoothed in preparation for placement of the material. A ratio of 3.0 horizontal to 1.0 vertical generally is considered a stable bank slope angle; although installations observed on the Sacramento River and tributaries include many locations with a stable 2H:1V or 2.5H:1V revetted bank. Considerable disruption of the existing bank and overbank areas often results from placement of this type of erosion control system (King 1986). **Figure 4.57** illustrates a cross section view of typical stone revetment.

Stone riprap can be used without bank shaping. The U.S. Army Corps of Engineers (USACOE 1981) has utilized placement of toe (**Figure 4.58**) and windrow riprap (**Figure 4.59**) successfully. With both of these methods, some additional loss of top bank results as erosion processes act to reduce the bank angle.

Where banks are being undermined by toe erosion, toe riprap is commonly used (USACOE 1989). To prevent flanking, stone tiebacks are generally used on each end of the revetment. Toe riprap may be combined with upper bank vegetation, or original upper bank vegetation can be preserved during rock emplacement. The height requirement of the rock will be site specific, depending on bank stratigraphy, exposure to wave action and hydrology. Effective use of toe revetment where additional bank retreat is unacceptable (e.g., at levee toe) requires highly erosion-resistant upper bank materials. Where upper bank materials are prone to erosion during high flows, some additional upper bank retreat must be acceptable, or toe riprap should not be applied. The combination of toe riprap and upper bank vegetation has been used successfully where undercutting from toe erosion has been a problem.

Windrow revetment consists of stone placed on topbank or in a trench that is setback from and parallels the bank. As the river migrates laterally into the rock, it is launched onto the bank and forms a protective cover on the ungraded, natural bank. Major concerns with windrow revetment include cost and the proper formation upon launching of an adequate rock toe to provide effective, long-term bank protection. Rock windrow is applicable where substantial levee setback exists.

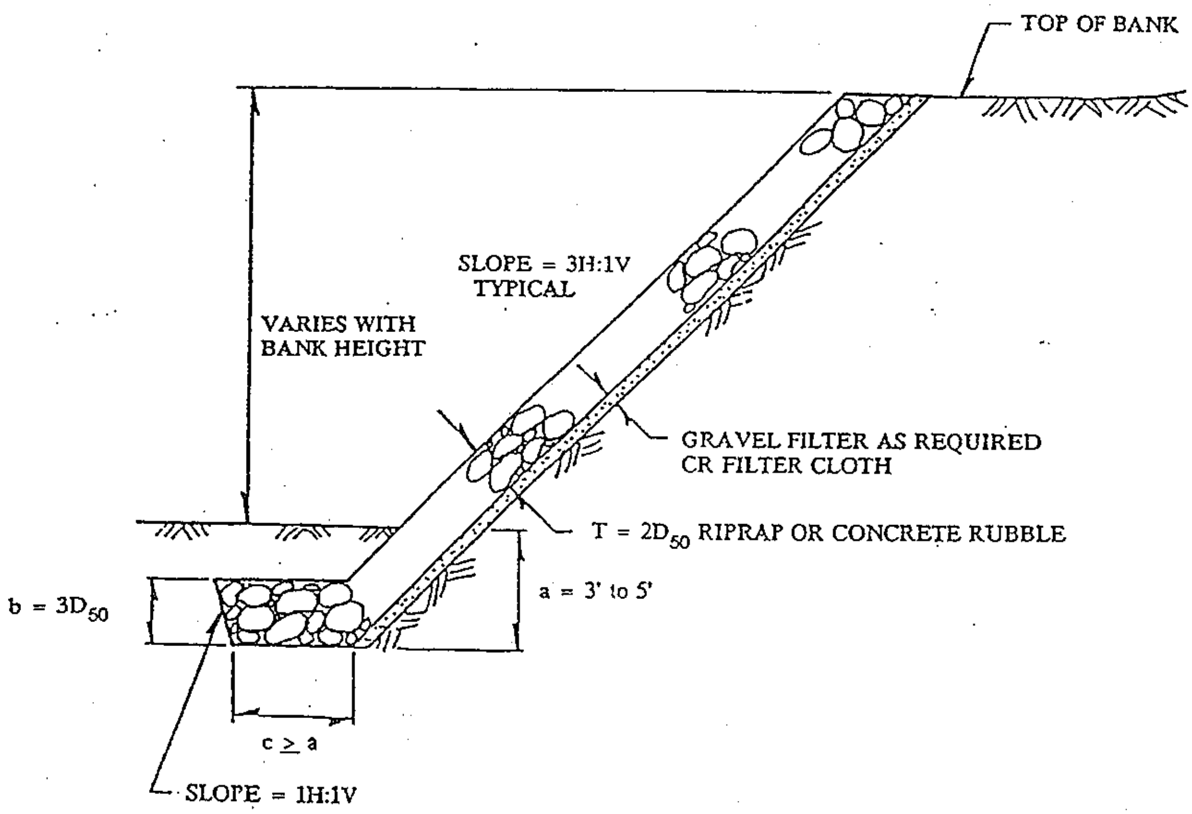
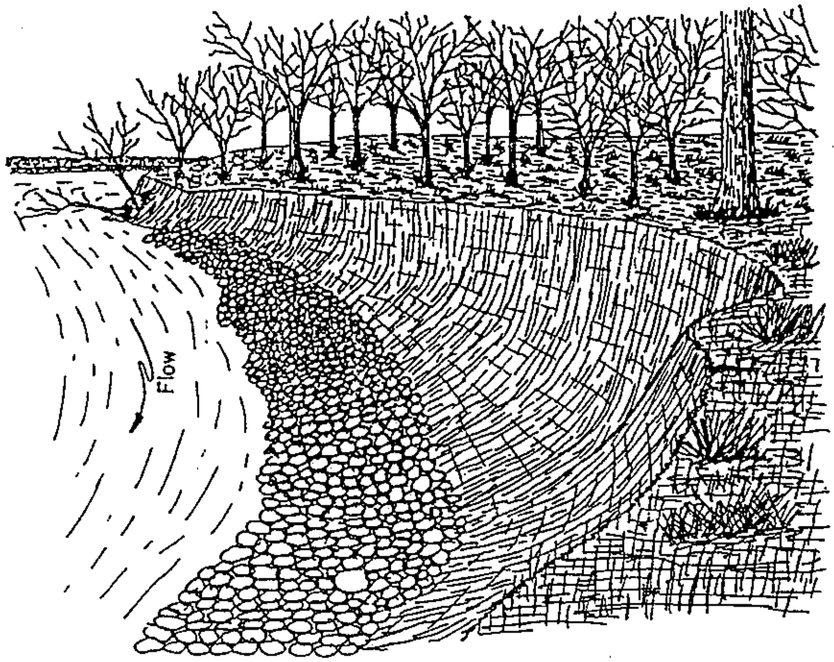
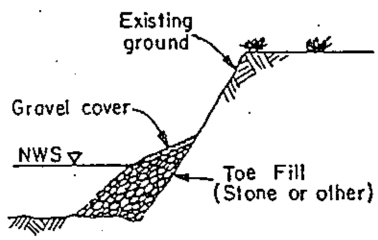


Figure 4.57. Typical stone revetment diagram.



COMPOSITE REVETMENT



TYPICAL SECTION

Figure 4.58. Typical toe riprap diagram (from USACOE 1981)

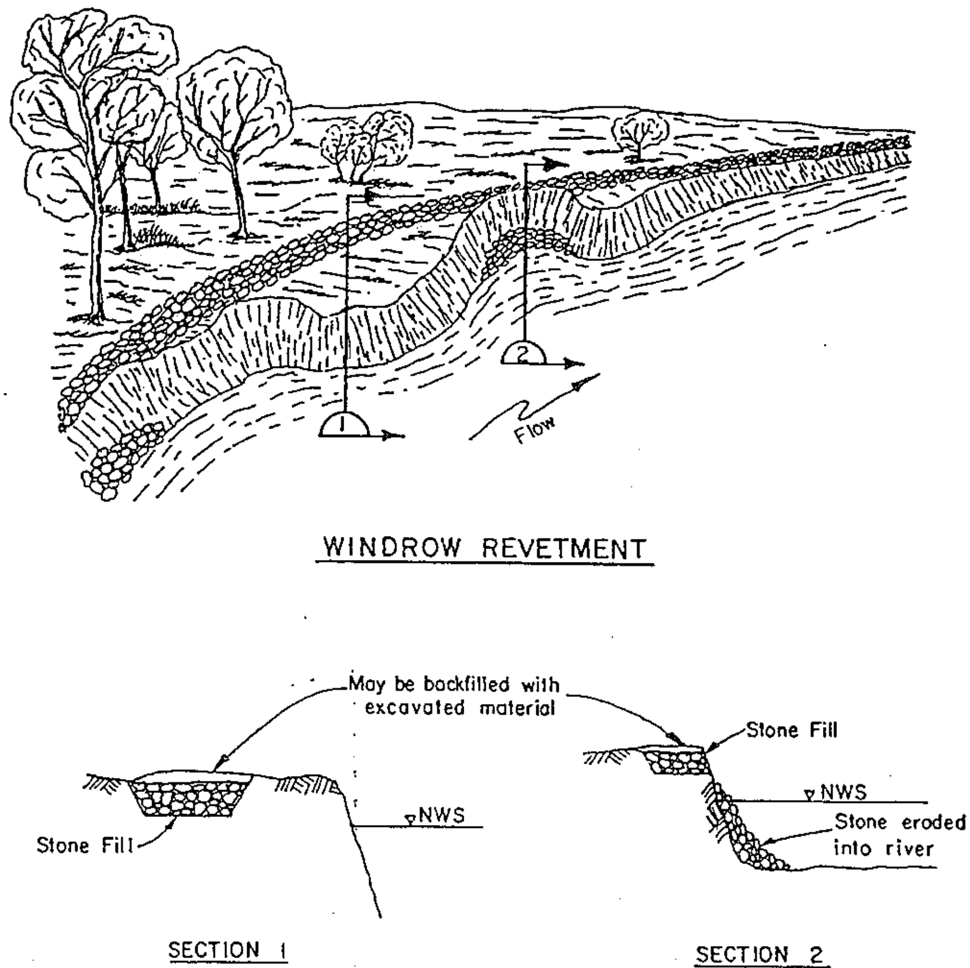


Figure 4.59. Typical windrow riprap diagram (from USACOE 1981).

When it is available, stone riprap has been consistently recognized as the most effective material for bank revetment (Barnes 1971; Karaki, et al. 1975; CalTrans 1970; USBR 1978; USACOE 1973). Concrete or masonry rubble can serve in much the same way as stone riprap, and can be equally effective as stone riprap if sizes and dimensions are properly controlled (Cunningham 1934; USACOE 1973). River cobbles may be used in place of quarry stone; however, with river cobble riprap, proper gradations can be difficult to achieve. As the cobbles are rounded, river cobble riprap will not interlock to the extent of quarry stone; consequently, the river cobble will be more prone to erosion during high flows. River cobble riprap constitutes a major proportion of revetments emplaced along the Sacramento River before 1973.

Fabricated concrete can take a wide variety of forms in providing suitable revetment measures. Concrete fabrications can include: slope paving (CalTrans 1970), cellular concrete blocks (Amber 1976; Cox 1971; Parsons 1965), pre-mixed sack revetment (CalTrans 1970; Finch 1939), articulated concrete mattress (Carey 1966; Gilland 1930), and pre-fabricated tetrapods (CalTrans 1970; Danel and Greslau 1963). Asphalt has been used in at least three forms, including slope paving (ASCE 1965; Bessen 1939; CalTrans 1970), asphalt blocks (Auakian 1969; CalTrans 1970; Visser and Claessen 1975), and bituminous mattress (Asphalt Inst. 1934; Van Asbeck 1964).

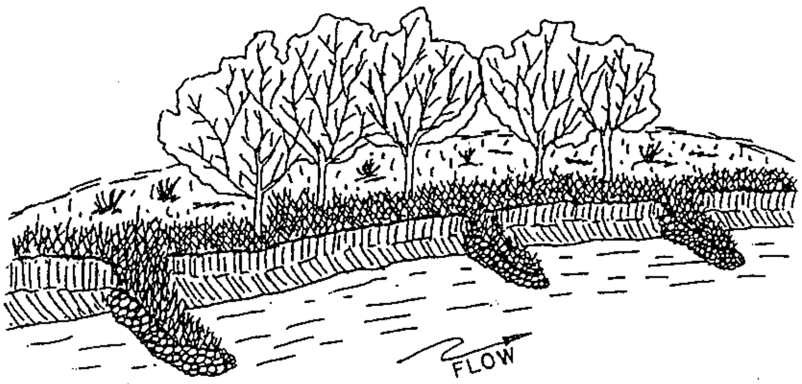
Vegetation used as revetment can include grassing (Barnes 1971; Schiechtl 1980), brush matting (Gray and Leiser 1982; Barnes 1971), and log-and-cable (James 1967; Weller 1970). Revetment methods utilizing geosynthetics universally incorporate vegetation as an integral part of the methodology (Koerner 1986).

Rock-and-wire revetment methods include gabion baskets (Crews 1970; Fajin 1974; Lavignino 1974), mattresses (Nash 1944; Posey 1973), and combinations of rock, timber, and wire mesh (Anderson 1908; Grant and Fenton 1948; Tilton 1939). Gabions consist of rock-filed wire baskets that are lashed together to form a continuous structure. Gabions require less stone than riprap and can be emplaced on steeper slopes (WES 1989). Where utilized on the Sacramento River and tributaries, gabions commonly fail due to scour beneath the baskets.

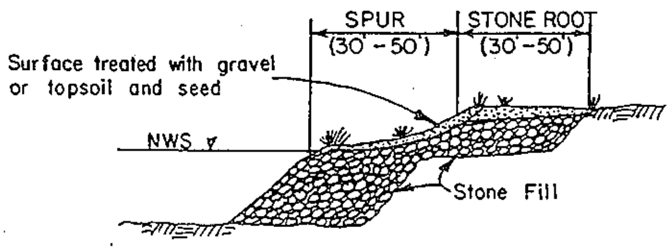
Dikes

Dikes fall in the category of an erosion control or flow diversion structure extending roughly perpendicular from a stream bank that either diverts flow from the bank or reduces flow velocity adjacent to the bank. Dikes can be distinguished as either permeable or impermeable (Winkley 1971; Brown 1985). Impermeable dikes usually are constructed of natural stone or concrete rubble. The primary purpose of these features is to redirect or divert the flow. Impermeable dikes have been used with success on the Yuba River and Lower American River. Permeable dikes are known as retardance features because their primary function is to reduce near-bank velocities (Fenwick 1969). **Figure 4.60** illustrates a typical impermeable dike system, and **Figure 4.61** illustrates a permeable dike. **Figure 4.62** (USACOE 1981) illustrates two additional permeable fence-type dikes, a steel cable fence, and a board-fence dike.

Fencing and netting are low cost, relatively non-disruptive methods of stream bank erosion control for small- to medium-sized streams (Richardson et al. 1987). The following special considerations must be taken into account to ensure success of these structures (Richardson et al. 1987): (1) fence structures must be designed to withstand bedload and floating debris such as cobbles and trees, (2) these structures must promote deposition and vegetation establishment, and (3) erosion protection should be provided at the toe of dikes and at the base of supporting members to prevent failure due to scour and at the bank end of dikes to prevent flanking of the fence.



HARD POINT SYSTEM



TYPICAL SECTION

Figure 4.60. Typical impermeable dike system (from USACOE 1981).

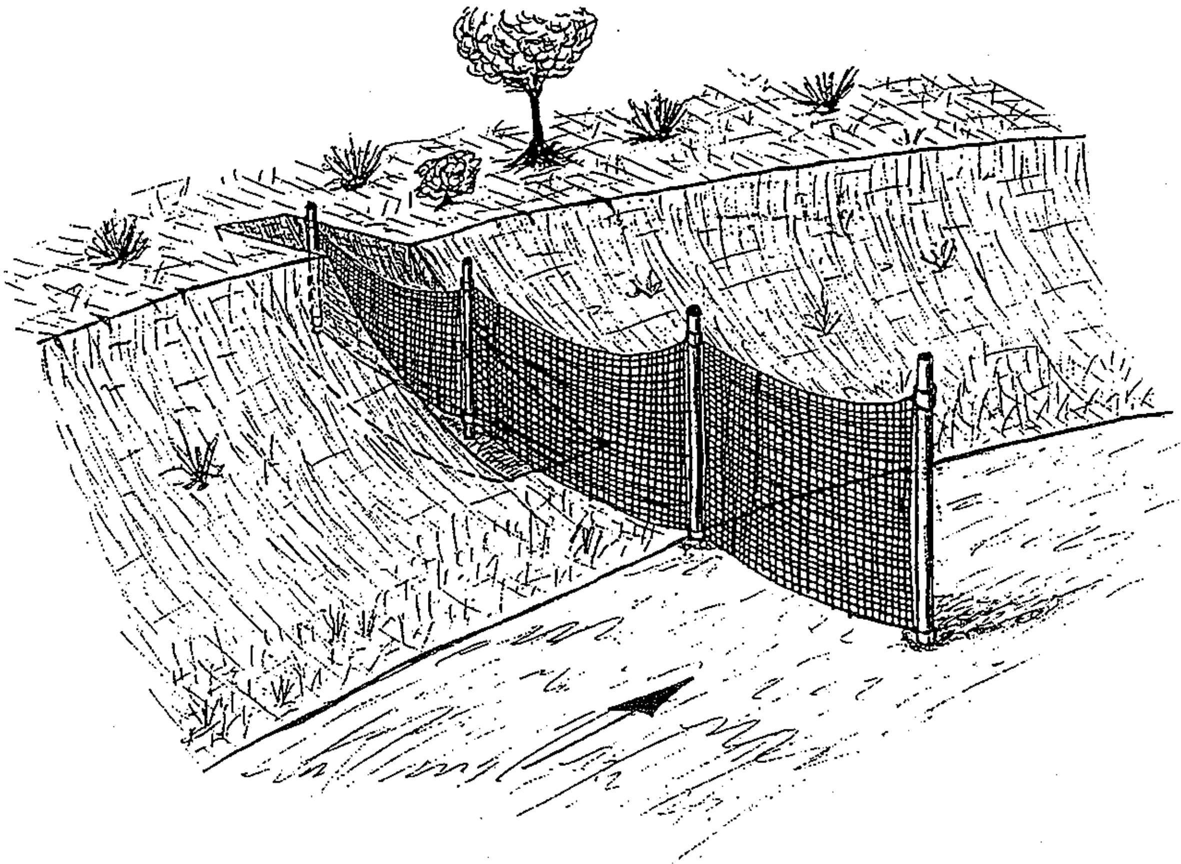


Figure 4.61. Permeable spur dike pictorial sketch.

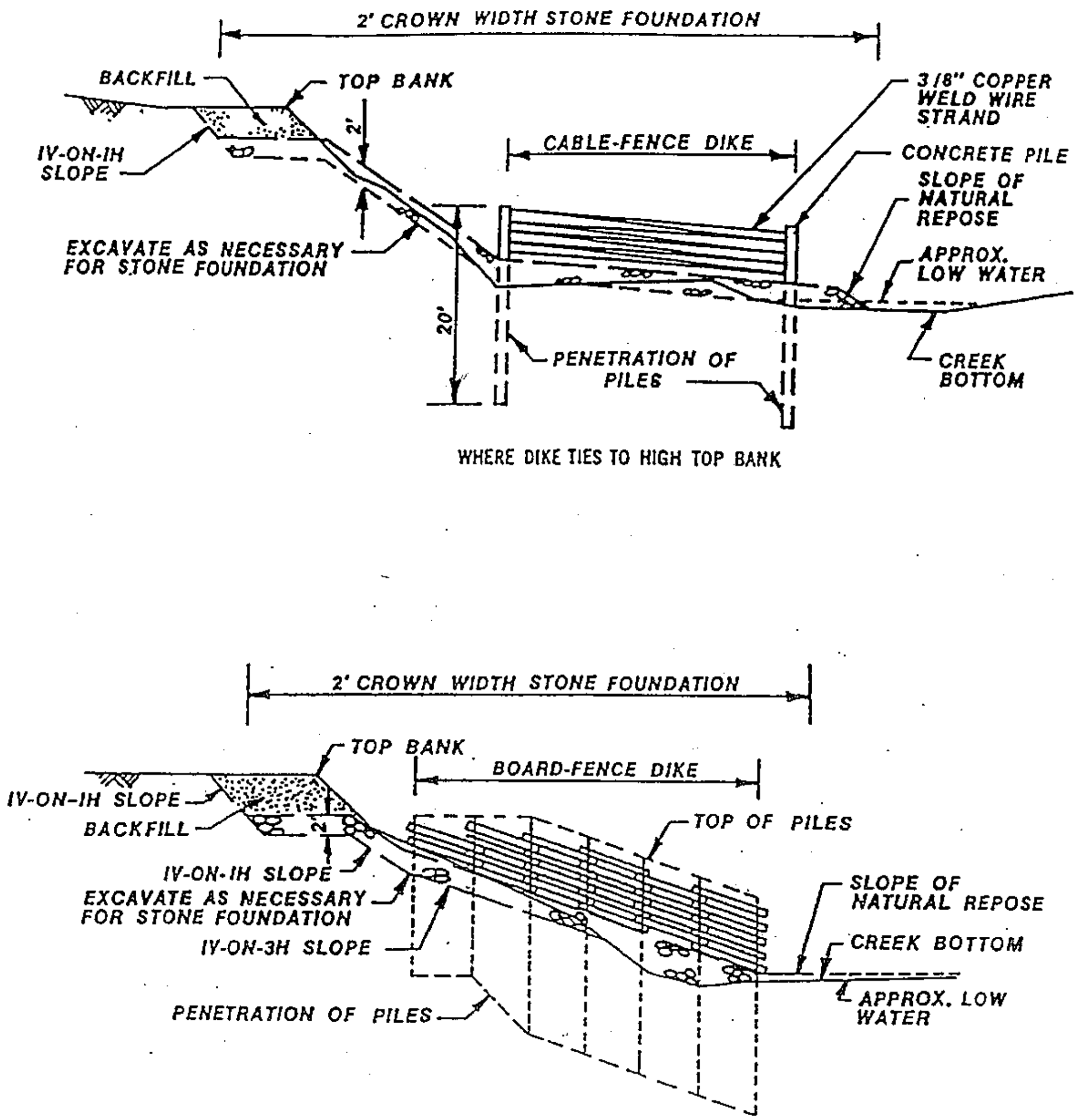


Figure 4.62. Typical permeable fence-type dikes (top, steel cable fence; bottom, board fence [from USACOE 1981]).

Bioengineering

Bioengineering is the applied science of combining living vegetation with conventional engineering solutions to soil and stream bank erosion problems. Biotechnical methods are, therefore, combinations of biological and structural components designed to enhance each other. Depending upon site-specific conditions, the result can be a more environmentally sound, aesthetically pleasing, and cost-effective solution than conventional methods provide.

Biotechnical structures vary depending upon the application and desired effects. The relative contributions from the vegetative and structural components also vary with methods. Some require vegetation only; while others may need reinforcement by structural measures or rock. The biotechnical methods recommended as being appropriate for the Sacramento River Bank Protection Project include the following: solutions ranging from vegetation only, mixes of conventional engineering methods on lower banks, or vegetation on upper banks.

Design of biotechnical systems requires considerations not normally needed in conventional engineering solution. Criteria used in selecting plant species include: (1) the environmental conditions of the site, (2) propagation techniques and plant availability, (3) biotechnical suitability, plant growth, and rate, and (4) aesthetic effect (Schiechtl 1980). Because vegetation is commonly an integral part of the system, construction must be timed carefully in order to optimize growing conditions. Depending upon the species and condition of plants being used (whether seed, cutting or transplant), timing of construction needs to coincide with either the onset of normal wet seasons or periods of dormancy.

Live crib retaining walls incorporating dead and live vegetation can be used to protect a stream bank in a manner similar to an impermeable dike. The crib walls are constructed of logs or railroad ties that have branches of live plants, such as willow or alder, arranged within them (**Figure 4.63**). The wall then is backfilled with enough fine-grained bed material to support growth. This method can be desirable because it is immediately effective, and the walls can be constructed in a relatively short period. A disadvantage of this system is that the logs of the retaining wall will decay with time. However, the function of the timber is gradually assumed by the growing plants. An alternative that adds durability but also increases cost is to substitute prefabricated concrete for the logs (Schiechtl 1980). Live crib retaining walls have been used successfully in the repair and prevention of scour holes (WES 1989).

Live fascine drains are constructed to channel water from a slope. Fascines are live branches tied together with wire in long, sausage-like bundles approximately 1 to 2 ft in diameter (**Figure 4.64**). The fascines then are placed in ditches excavated to the desired drainage depth. The ditches are filled with soil to cover all the branches and to prevent water from running below the bundles. To prevent the fascines from washing away, they are tied together with wire and staked with live pegs. This alternative is simple to construct and is effective quickly. However, the fascines can be constructed only during the dormant season and require periodic maintenance (Schiechtl 1980).

Brush mattress construction is a method of providing cover and protection to large surface areas. The method uses live branches placed close enough together on the ground to form a complete cover (**Figure 4.65**). Layers of brush are secured with wire that is attached to stakes at regular intervals to form a grid. Brush matting, therefore,

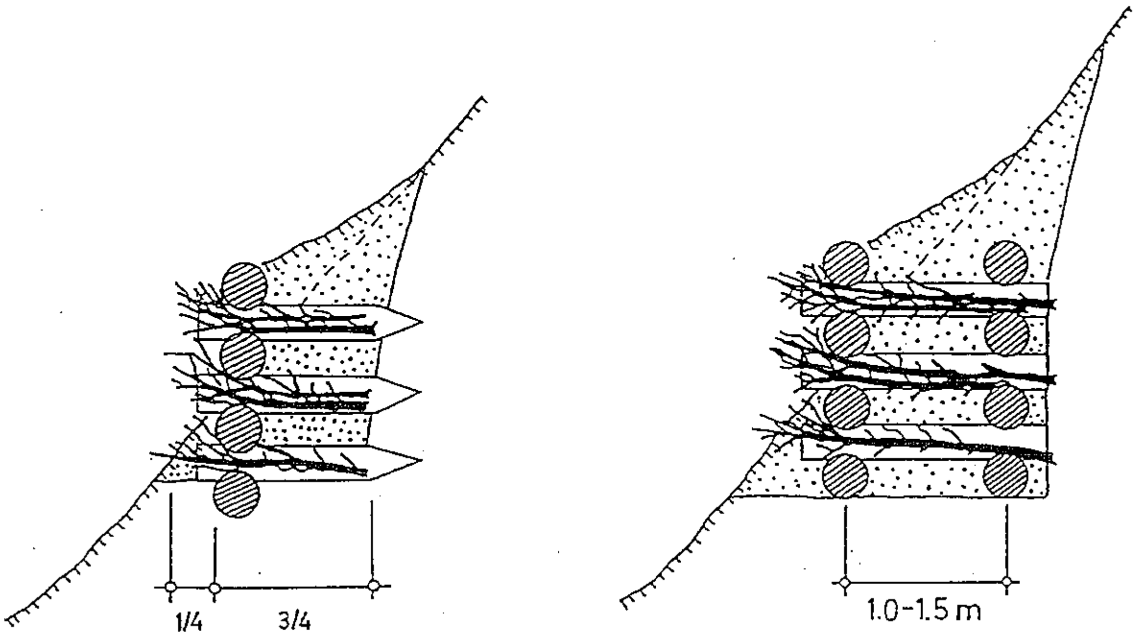


Figure 4.63. Diagram of a live wooden crib wall (left, single-walled; right, double-walled [Schiechtl 1980]).

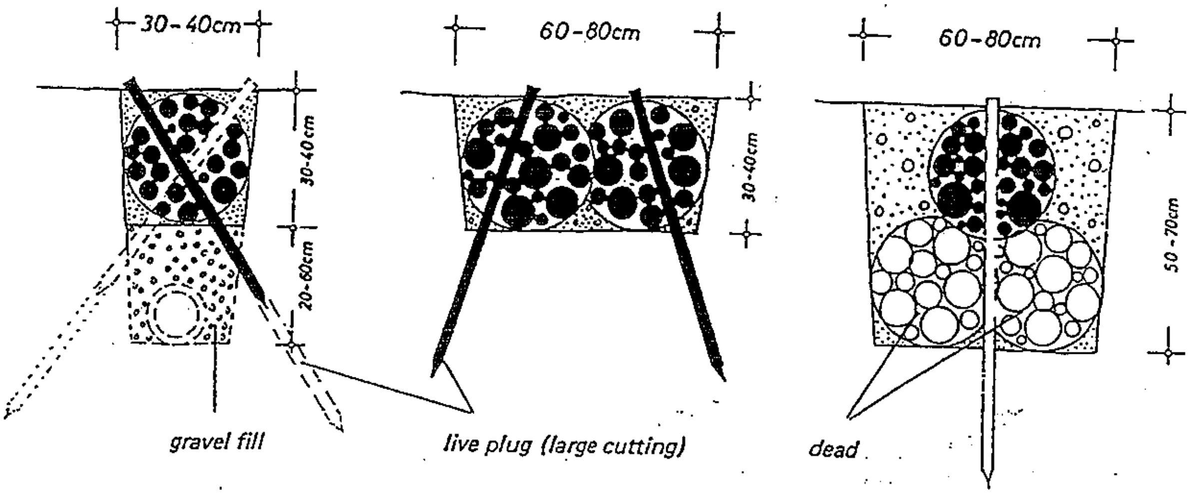


Figure 4.64. Diagram of three types of live facine drawings (Schiechtl 1980).

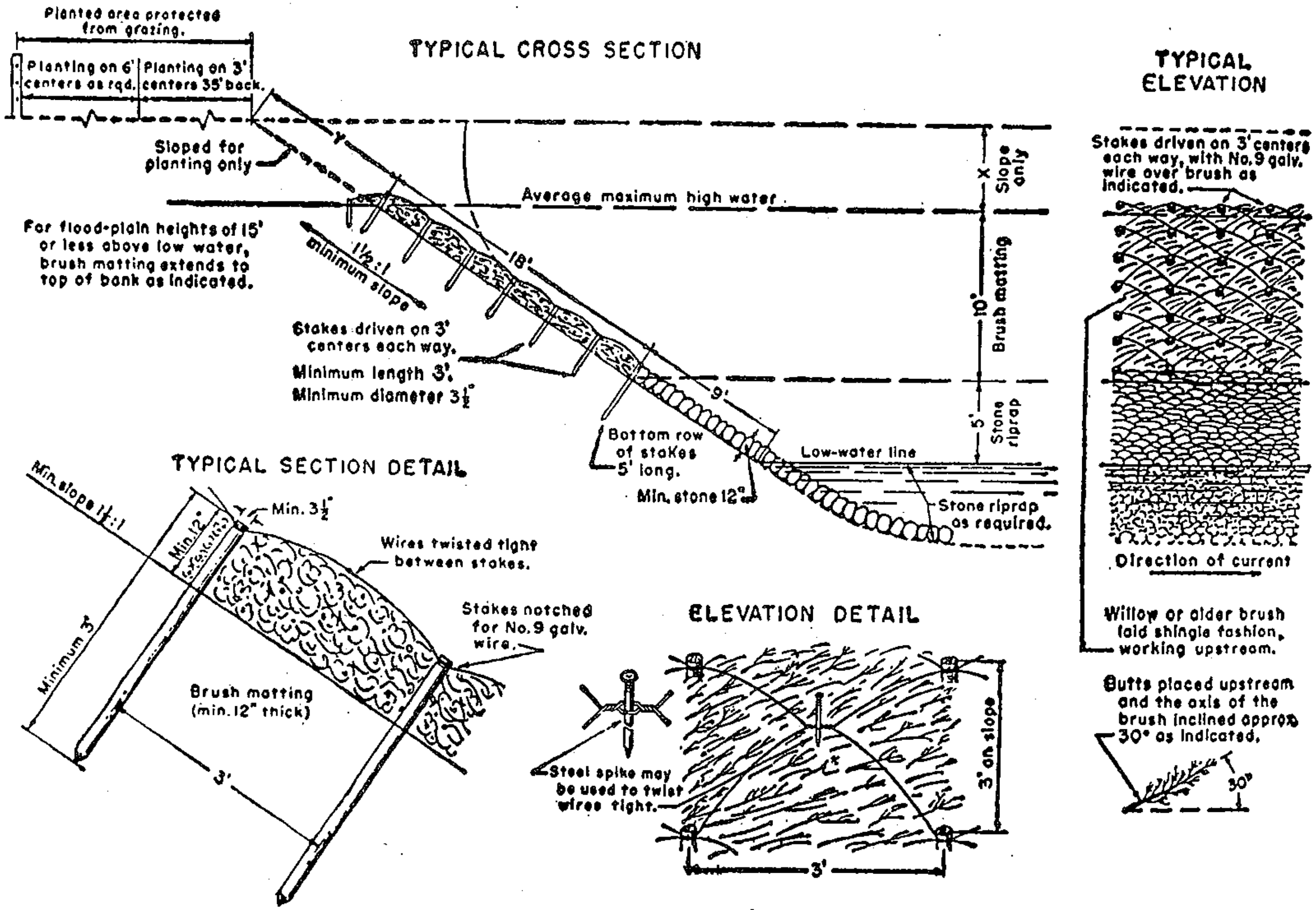


Figure 4.65. Construction procedure and details for brush matting (from Edminister, 1949).

would be most suitable for stream bank protection when used in conjunction with toe riprap or alternative toe stabilization. Immediate effectiveness can be achieved (followed by dense root and thicket development). A disadvantage of brush matting is that it requires a considerable amount of labor and materials (Schiechtl 1980). An alternative to brush matting is the use of three-dimensional geosynthetic grid material, which will stabilize soil while giving plants an opportunity to establish (**Figure 4.66**).

Flow diversion also can be accomplished through biotechnical methods in some locations. Log brush barriers are densely packed layers of branches and logs, which divert streamflow from an eroding bank (**Figure 4.67**). The lowest layer consists of large branches or trunks staked at right angles to the bank. Successive layers of live branches then are placed over the base and covered with rocks or concrete. The structure encourages siltation within the branches through decreased velocities. This method is appropriate for toes of stream banks, which provide immediate effectiveness and could be used in combination with dikes. However, the structures are labor-intensive, can be constructed only during dormancy, and are most applicable to shallow or ephemeral channels.

Branch packings are alternating layers of branches and fascines, which can be effective even in deep water. Before another layer is constructed, the branch layers are tied down with stakes and covered with gravel or cobble (**Figure 4.68**). Live branches should be used in shallow water applications, and dead branches in deeper water where plant growth cannot be expected. Toe protection can be improved by including riprap in the lower bank and the branch packings above it (Schiechtl 1980). Under water decay of branch layers can result in bank settling and a reduction in the effectiveness of bank protection.

Other geosynthetic materials may offer cost-effective alternatives to conventional bioengineering methods (Koerner 1986). However, these alternatives have not been widely supported in the literature.

The success of biotechnical types of bank protection is highly dependent on the mode of failure of the eroding bank. Biotechnical methods may be effective on banks that erode by grain detachment; however, banks that fail massively would not be effectively protected biotechnically. Biotechnical bank protection methods should not be employed at sites where additional bank retreat is unacceptable without further study as to their effectiveness on large rivers with long periods of sustained high flow.

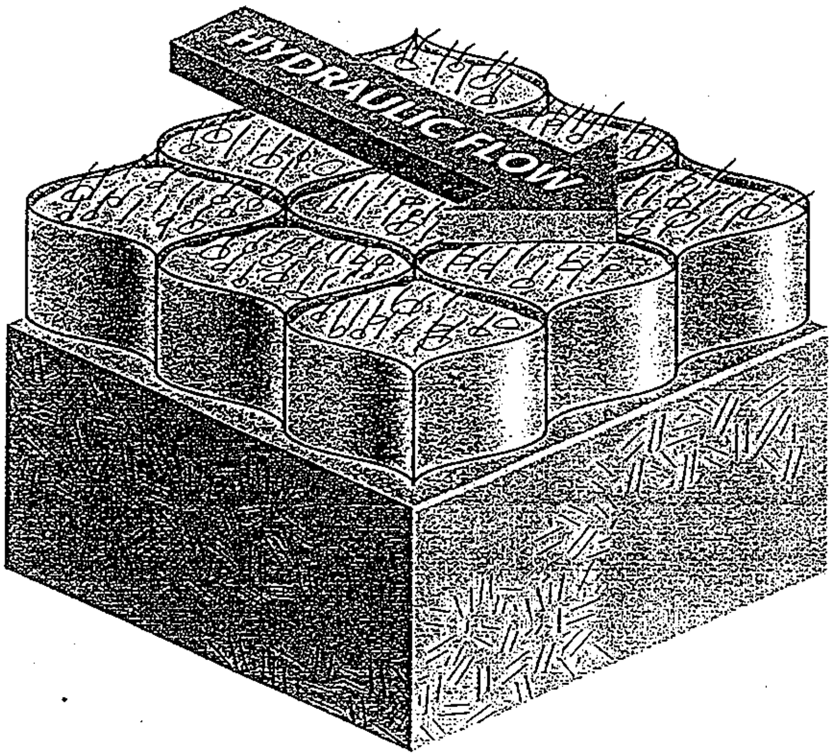


Figure 4.66. A three-dimensional grid of geosynthetic material stabilizes the plant environment against hydraulic shear.

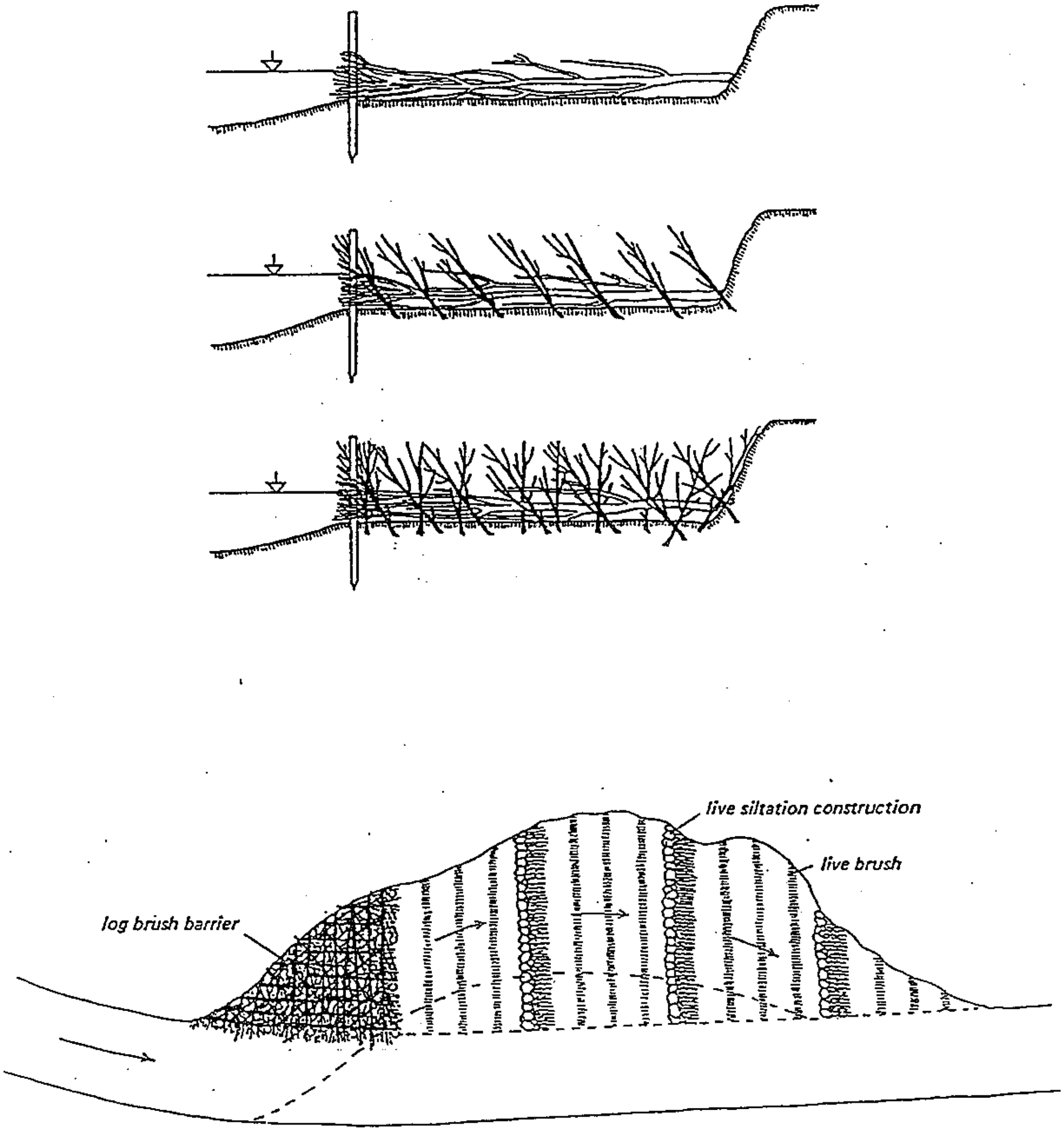


Figure 4.67. Diagram of log brush barriers (top, cross sections; bottom, plan view [Schiechtl 1980]).

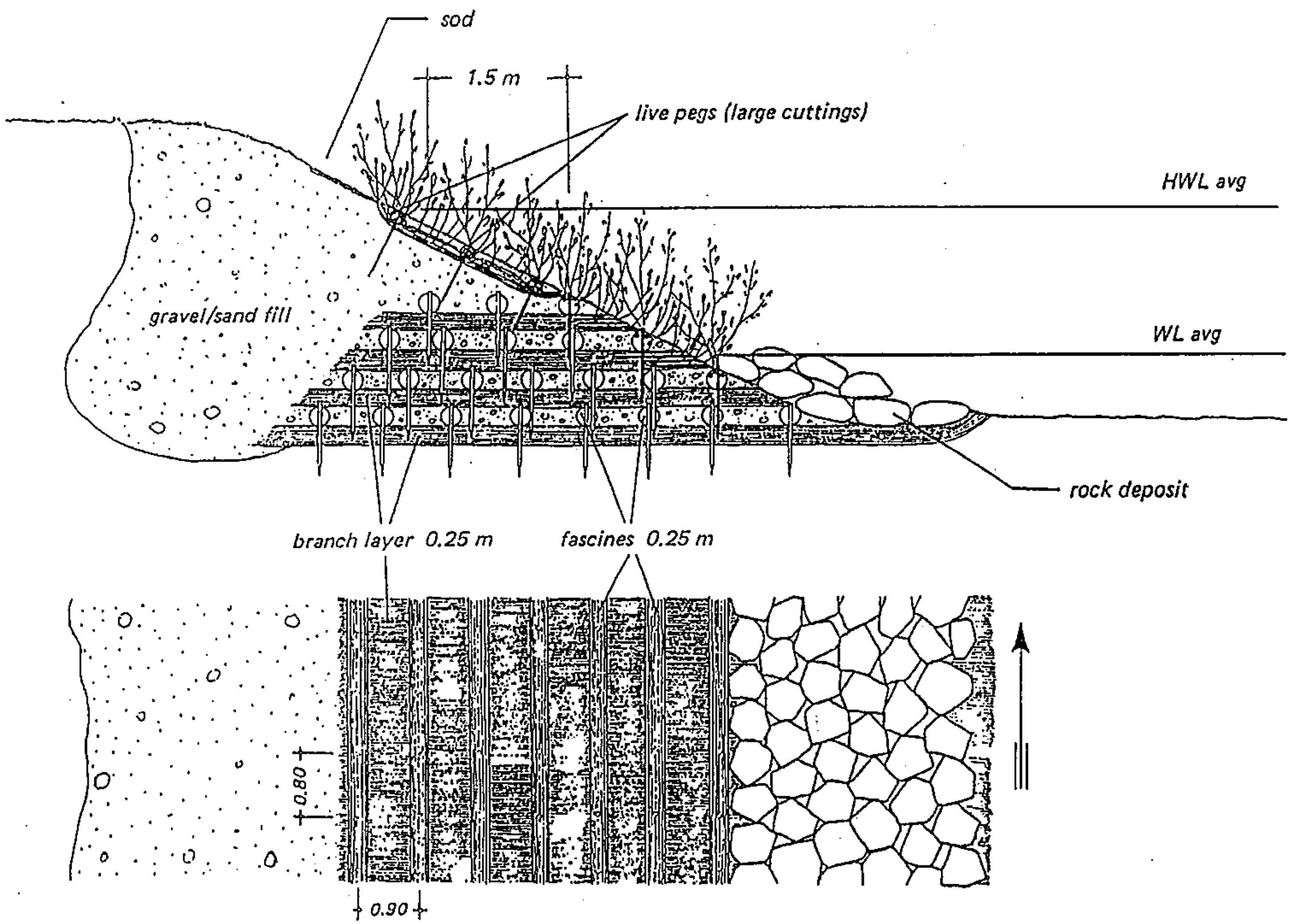


Figure 4.68. Diagram of branch packing for shore protection (from Schiechtl, 1980).

4.6. Bridge Analysis

There are 19 bridge crossings of the American River within the project reach. Many of these bridges contain piers and abutments that are subject to scour during high flows. An analysis of the scour potential at each of the bridges under the various design scenarios is presented in this chapter.

4.6.1. Analysis Procedures

Scour potential was evaluated using the principals and procedures outlined in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 18, "Evaluating Scour at Bridges" (FHWA 1993). A brief review of the key concepts and the procedures used is given here.

Total scour at a bridge crossing is composed of three components:

1. Aggradation and Degradation. These are long-term vertical stream bed changes due to natural or man induced causes within the reach of the river in which the bridge is located. Aggradation and degradation potential was evaluated in the sediment transport analysis presented in Chapter 6.

2. Contraction Scour. Contraction scour in a natural channel involves the removal of material from the bed and banks across all or most of the channel width. This component of scour results from a contraction of the flow. Scour is caused by increased velocities and a resulting increase in bed shear stresses. Contraction of the flow by bridge approach embankments encroaching onto the floodplain and/or into the main channel is the most common cause of contraction scour.

3. Local Scour. Local scour involves removal of material from around piers, abutments, spurs, and embankments. It is caused by an acceleration of flow and resulting vortices induced by the flow obstructions.

An evaluation of bridge geometries and hydraulic conditions in the vicinity of the bridges indicates that in addition to aggradation and degradation, pier scour, and abutment scour are the only scour components of potential concern in the project reach. These scour components were evaluated separately using the following procedures.

Pier scour was evaluated using the CSU equation (Melville and Sutherland 1988):

$$\frac{y_s}{y_s} = 2.0\,K_1\,K_2\,K_3\left[\frac{a}{y_1}\right]^{0.65} Fr_1^{0.43} \tag{4.9}$$

where:

y_s	=	Scour depth (ft)
y_1	=	Flow depth just upstream of the pier (ft)
K_1	=	Correction factor for pier nose shape
K_2	=	Correction factor for angle of attack of flow
K_3	=	Correction factor for bed condition
a	=	pier width (ft)
L	=	length of pier (ft)
Fr_1	=	Froude number $V_1/(gy_1)^{1/2}$
V_1	=	Mean velocity of flow directly upstream of the pier (ft/s).

Values for correction factors K_1, K_2 and K_3 are provided in Hydraulic Engineering Circular No. 18 (FHA 1993).

When the pile cap or footing is not exposed, Equation 4.9 provides an estimate of scour around constant diameter piers. When the footing is exposed, use of the footing diameter to estimate the scour depth leads to overly conservative results. Following Federal Highway Administration recommendations (FHA 1993), in addition to scour computed in the normal fashion using the pier diameter, an alternate scour depth should be computed with the height of the exposed footing (y_f) used for y_1, the footing diameter used for a, and the average flow velocity at the exposed footing (V_f) used for V_1. V_F is given by:

$$\frac{V_f}{V_1} = \frac{\ln(10.93\frac{y_f}{k_s} + 1)}{\ln(10.93\frac{y_1}{k_s} + 1)} \qquad (4.10)$$

where k_s is the grain roughness of the bed (ft) and normally taken as the D_{84} sediment size. The maximum computed scour from the two methods should be used.

Abutment scour was computed using an equation developed by Froehlich (1989):

$$\frac{y_s}{y_a} = 2.27\ K_1\ K_2 \left[\frac{a'}{y_a}\right]^{0.43} Fr^{0.61} + 1 \qquad (4.11)$$

where:

	y_s	=	Scour Depth (ft)
	y_a	=	Depth of floodplain flow at abutment (ft)
	K_1	=	Coefficient for abutment shape
	K_2	=	Coefficient for angle of embankment to flow
	a'	=	The length of the abutment projected normal to the flow (ft)
	F_r	=	Froude number of approach flow upstream of the abutment
		=	$V_e/(gy_a)^{1/2}$
	V_e	=	Velocity (Q_e/A_e) (ft/s)
	Q_e	=	The flow obstructed by the abutment and approach embankment (cfs)
	A_e	=	The flow area of the approach cross section obstructed by the embankment (ft^2).

Values for coefficients K_1 and K_2 are provided in Hydraulic Engineering Circular No. 18 (FHA 1993). Equation 4.11 tends to give overly conservative results. As an alternative, abutment scour can be computed as:

$$\frac{y_s}{y_a} = 4\ Fr^{0.33} \qquad (4.12)$$

4.6.2. Results

Local scour (pier and abutment scour) calculations were carried out for the peak flow of the 100-year event for each design scenario. Bridge geometric data were obtained from the Floodplain Management Section of the Sacramento District and were originally compiled as part of the 1992 FIS for the American River below Nimbus Dam. The data include hand drawn sketches and as-built construction drawings obtained from a variety of sources. Hydraulic variables were determined using the HEC-2 model developed for the study reach. Three cases for downstream stages were considered: (1) the minimum stage coincident with the peak flow from the Sacramento District routings, (2) the stage determined using the lower bound rating curve (Figure 4.18), and (3) the stage determined using the upper bound rating curve (Figure 4.18).

Table 4.19 summarizes computed pier scour depths using downstream stages from the Sacramento District routings. Bridges not included in Table 4.17 do not have piers in the main channel (Guy West pedestrian bridge and Goethe Park bicycle bridge) or are located on competent material (Hazel Avenue bridge) and hence were not analyzed. Results in Table 4.19 show that scour depths tend to be greatest for Scenario 2 for bridges in the lower portion of the study reach (downstream of approximately RM 10) and greatest for Scenario 1 for bridges in the upper portion of the study reach (upstream of RM 10). **Tables 4.20 and 4.21** summarize computed pier scour depths for downstream stages equal to the lower and upper bound rating curves. The results are similar to those computed using downstream stages from the Sacramento District routings. Computed scour depths for bridges in the lower portion of the study reach are slightly higher using the lower bound rating curve (Table 4.20) are slightly higher using the upper bound rating curve (Table 4.21). Regardless of the assumed downstream stage, differences between the scenarios are minor compared to the magnitude of the computed scour depths. It is therefore concluded that the different scenarios will have very little effect on the local scour potential at bridges within the study reach.

Federal Highway Administration procedures (FHA 1993) require that general bed degradation be added to the local pier scour depths to determine the total scour potential. Sediment transport results given in Section 4.4 indicate that the bridges are generally located in stable areas with the exception of the 16th Street bridge, which shows approximately 0.5 ft of degradation under each of the design scenarios. Although relatively small compared to the computed pier scour depths, this additional depth should be considered when evaluating the stability of the 16th Street bridge.

Although differences between the design scenarios are minor, the magnitude of the computed scour depths indicate scour depths that could potentially affect the stability of the structures. Included in Tables 4.19 through 4.21 are estimated footing depths where data were available. The estimated footing depths represent the difference between the bed elevation at the pier nearest the thalweg (taken from the HEC-2 input file) and the bottom of footing elevation (obtained from as-built bridge plans). These depths should be used with caution due to uncertainties in the cross section locations relative to the piers and possible changes in the cross sections since the date of the survey. Comparison of the computed pier scour depths (including the additional 0.5 ft of bed degradation at the two 16th Street bridge crossings) with the estimated footing depths shows that the scour hole would be below the bottom of the footings regardless of design scenario or downstream stage condition.

It is important to note that the calculations are based on Federal Highway Administration procedures (FHA 1993), which do not consider the potential effect of coarse material in development of the local scour hole. The coarse nature of the channel bed in the project reach may limit actual scour hole development, which makes the computed scour depths conservative. However, the calculations do point to potential problems at many of the

Table 4.19. Pier Scour Estimates, 100-Year Event, COE Routed Downstream Stage.							
Bridge	River Mile	Footing Depth (ft)	Computed Pier Scour (ft)*				
			Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5
Jibboom St. (left)	0.18	N/A	31.8	34.0	33.3	34.0	29.3
Jibboom St. (center)	0.18	N/A	12.5	13.4	13.1	13.3	11.5
I-5 (d/s span)	0.25	8.9	9.2	9.7	9.5	9.7	8.4
I-5 (u/s span)	0.27	8.6	9.0	9.5	9.3	9.5	8.2
16th St. (d/s span)	1.99	1.0	27.0	28.5	27.0	28.3	24.6
16th St. (u/s span)	2.00	9.1	8.3	8.8	8.3	8.7	7.6
Old Sac. RR	2.20	N/A	27.1	28.3	26.9	28.1	24.6
UPRR	2.28	28.1	34.5	36.0	34.3	35.8	31.3
SPRR	3.77	N/A	22.5	23.4	22.7	23.4	21.3
HW 51 (I-80)	4.02	0.4	10.8	11.1	10.6	11.0	9.7
H-Street	6.61	18.4	33.6	34.3	32.7	33.9	29.7
Howe Ave. (d/s span)	7.81	5.0	6.5	6.7	6.4	6.6	5.9
Howe Ave. (u/s span)	7.82	5.0	6.5	6.6	6.4	6.6	5.9
Watt Ave.	9.37	7.0	11.4	11.6	11.2	11.5	10.5
Arden-Geothe Pk.	13.60	N/A	13.7	13.2	13.2	13.2	13.0
Private Rd.	20.05	N/A	8.6	8.5	8.3	8.4	8.1
Sunrise Blvd.	20.18	5.9	19.3	19.1	18.4	18.9	17.0
Old Bridge St.	20.46	N/A	16.6	16.3	15.6	16.1	14.2

* This is the local pier scour only and does not include potential general bed degradation (see discussion in text).

Table 4.20. Pier Scour Estimates 100-Year Event, Lower Bound Downstream Stage.							
Bridge	River Mile	Footing Depth (ft)	Computed Pier Scour (ft)*				
			Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5
Jibboom St. (left)	0.18	N/A	34.6	35.3	33.6	34.6	31.2
Jibboom St. (center)	0.18	N/A	13.6	13.9	13.2	13.6	12.3
I-5 (d/s span)	0.25	8.9	9.9	10.1	9.6	9.9	8.9
I-5 (u/s span)	0.27	8.6	9.7	9.9	9.4	9.7	8.7
16th St. (d/s span)	1.99	1.0	27.8	28.3	26.9	27.8	24.4
16th St. (u/s span)	2.00	9.1	8.6	8.7	8.3	8.6	7.5
Old Sac. RR	2.20	N/A	27.7	28.2	26.9	27.7	24.4
UPRR	2.28	28.1	35.3	35.8	34.2	35.3	31.1
SPRR	3.77	N/A	23.0	23.3	22.6	23.0	21.1
HW 51 (I-80)	4.02	0.4	10.9	11.1	10.6	10.9	9.7
H-Street	6.61	18.4	33.8	34.3	32.7	33.8	29.7
Howe Ave. (d/s span)	7.81	5.0	6.6	6.7	6.4	6.6	5.9
Howe Ave. (u/s span)	7.82	5.0	6.6	6.6	6.4	6.6	5.9
Watt Ave.	9.37	7.0	11.5	11.6	11.2	11.5	10.5
Arden-Geothe Pk.	13.60	N/A	13.7	13.2	13.2	13.2	13.0
Private Rd.	20.05	N/A	8.6	8.5	8.3	8.4	8.1
Sunrise Blvd.	20.18	5.9	19.3	19.1	18.4	18.9	17.0
Old Bridge St.	20.46	N/A	16.6	16.3	15.6	16.1	14.2

* This is the local pier scour only and does not include potential general bed degradation (see discussion in text).

Table 4.21. Pier Scour Estimates, 100-Year Event, Upper Bound Downstream Stage.							
Bridge	River Mile	Footing Depth (ft)	Computed Pier Scour (ft)*				
			Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5
Jibboom St. (left)	0.18	N/A	32.6	33.3	31.6	32.6	28.9
Jibboom St. (center)	0.18	N/A	12.8	13.1	12.4	12.8	11.3
I-5 (d/s span)	0.25	8.9	9.4	9.6	9.1	9.4	8.3
I-5 (u/s span)	0.27	8.6	9.2	9.4	8.9	9.2	8.1
16th St. (d/s span)	1.99	1.0	27.0	27.5	26.1	27.0	23.7
16th St. (u/s span)	2.00	9.1	8.3	8.5	8.1	8.3	7.3
Old Sac. RR	2.20	N/A	27.1	27.6	26.2	27.1	23.8
UPRR	2.28	28.1	34.5	35.1	33.4	34.5	30.4
SPRR	3.77	N/A	22.5	22.8	22.0	22.5	20.6
HW 51 (I-80)	4.02	0.4	10.8	11.0	10.4	10.8	9.5
H-Street	6.61	18.4	33.6	34.1	32.5	33.6	29.5
Howe Ave. (d/s span)	7.81	5.0	6.5	6.6	6.4	6.5	5.9
Howe Ave. (u/s span)	7.82	5.0	6.5	6.6	6.4	6.5	5.9
Watt Ave.	9.37	7.0	11.4	11.6	11.2	11.4	10.4
Arden-Geothe Pk.	13.60	N/A	13.7	13.2	13.2	13.2	13.0
Private Rd.	20.05	N/A	8.6	8.5	8.3	8.4	8.1
Sunrise Blvd.	20.18	5.9	19.3	19.1	18.4	18.9	17.0
Old Bridge St.	20.46	N/A	16.6	16.3	15.6	16.1	14.2
* This is the local pier scour only and does not include potential general bed degradation (see discussion in text).							

bridges. Field evidence indicates that problems already exist at some of the downstream bridges. One of the piers on Old Sacramento Railroad bridge (now a bicycle/pedestrian bridge), for example, has settled, which indicates partial failure of the structure. The Old Sacramento Railroad bridge, the Union Pacific Railroad bridge, and the Watt Avenue bridge piers have been protected with riprap or concrete rubble, which indicates past concern. Competent material in the bed of the channel (Pleistocene-age materials) may limit scour at bridges upstream of approximately RM 10 (although borings were not available to estimate the depth to the resistant material).

Table 4.22 shows the results of local abutment scour calculations at four bridge crossings where abutments project into the flow. Other bridges where abutment scour may be significant include the Goethe Park bicycle trail (RM 13.60) and the private bridge just downstream of Sunrise Boulevard (RM 20.18). Information with which to evaluate potential problems at these locations was inadequate for a quantitative evaluation. Abutments at other bridges within the study reach do not project significantly into the flow. The results given in Table 4.22 indicate that abutment scour potential is greatest under scenario 1, which considers the Sacramento District downstream stages. For scenario 2, scour potential is greatest at the downstream stages developed from the lower and upper bound rating curves. Abutment scour depths are consistently lowest under scenario 5.

Table 4.22. Abutment Scour Estimates, 100-Year Event.						
Bridge	River Mile	Computed Abutment Scour (ft)*				
		Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5
COE Supplied Downstream Stage						
16th St. (d/s span)	1.99	19.3	17.9	15.5	16.3	10.8
16th St. (u/s span)	2.00	13.5	11.8	10.1	10.8	7.3
Old Sac. RR	2.20	13.2	12.0	9.6	10.5	4.7
UPRR	2.28	22.2	21.1	18.2	19.6	13.1
Lower Bound Downstream Stage						
16th St. (d/s span)	1.99	17.5	18.3	15.7	17.5	11.3
16th St. (u/s span)	2.00	11.6	12.4	10.4	11.6	7.9
Old Sac. RR	2.20	11.5	12.2	9.9	11.5	5.3
UPRR	2.28	20.5	21.5	18.4	20.5	13.6
Upper Bound Downstream Stage						
16th St. (d/s span)	1.99	19.3	20.1	17.8	19.3	13.2
16th St. (u/s span)	2.00	13.5	14.3	12.0	13.5	8.6
Old Sac. RR	2.20	13.2	13.9	11.5	13.2	7.2
UPRR	2.28	22.2	23.0	20.1	22.2	15.1
* This is the local abutment scour only, and does not include potential general bed degradation (see discussion in text).						

The general bed degradation potential of 0.5 ft at the two 16th street bridges should be added to the local abutment scour depths to determine total scour potential. Differences between the different design scenarios tend to be minor considering the magnitude of the computed numbers, which indicates that potential impacts from the different design scenarios are minor. Local scour at bridge abutments is a poorly understood process and Federal Highway Administration procedures (FHWA 1993) used to evaluate its magnitude tend to be overly conservative. In addition, abutments at the two 16th Street bridge crossings and the Union Pacific Railroad Crossing are protected by riprap or concrete rubble. This indicates that abutment scour may be limited at these locations. Conditions at the locations listed in Table 4.22 should be closely monitored after major flow events. If evidence of significant scour is present, corrective measures may be necessary. Methods for reducing or preventing damage from abutment scour can be found in FHWA (1993).

4.7. Scour Analysis for Five High-Priority Sites Along the Lower American River

4.7.1. Introduction

Five priority bank protection sites along the Lower American River were selected in 1995 based on consideration of criteria developed by RCE/Ayres, SAFCA, and the Lower American River Task Force (LARTF). The sites are shown in Figure 4.49 with approximate locations listed in **Table 4.23**. An analysis of the scour potential at each site was conducted to provide a basis for establishing the toedown depths to prevent failure of the protection. The results of this analysis are presented in the following sections.

Table 4.23. Lower American River Bank Protection Sites.		
Site	Site Length (ft)	Approximate Location
1	2,300	Left bank, RM 2.10
2	600	Left bank, RM 3.75
3	3,600	Left bank, RM 4.40
4	3,100	Left bank, RM 6.80
5	4,300	Right bank, RM 8.55

4.7.2. Analysis Procedures

The appropriate analytical procedures for determining the scour potential at each site depend upon the mechanisms causing the scour.. A summary of the potential scour mechanisms at each site is given in **Table 4.24**. The following paragraphs discuss the different scour mechanisms and the analytical procedures used for determining the potential scour depths.

In addition to scour depth, the width of the scour holes adjacent to the piers must be estimated in order to evaluate the potential impact of pier scour on the bank protection. Following the recommendations given in HEC-18, the top widths of the scour holes (measured from one side of a pier) were assumed to be 1.7 to 2.8 times the scour hole depths, and the bottom widths of the scour holes were assumed to be zero to 1.0 times the scour hole depths. In addition to the bridges at Sites 1 and 2, the pier scour calculations were applied to the Fairbairn Water Treatment Plant intake structure, which is near the upstream end of Site 4. There are no piers associated with the Guy West pedestrian bridge at Site 4 that will affect the proposed bank protection.

Table 4.24. Scour Mechanisms at Each Site.				
Site	Pier Scour	Embankment Scour	Bend Scour	Thalweg Realignment
1	X			X
2	X	X		X
3			X	X
4	X*		X	X
5				X
* Scour around Fairbairn Water Treatment Plant intake structure				

Procedures outlined in HEC-18 for estimating abutment scour at bridges were used to evaluate the potential scour associated with a protruding embankment just upstream of the SPRR bridge at Site 2. (Equations 4.11 and 4.12). Abutment scour calculations were applied at this site since they are considered to be the most applicable of the available scour analysis procedures for evaluating scour associated with protruding embankments. It should be noted that the abutment scour calculations for the UPRR and the SPRR bridges were not repeated here since the abutments are high on the embankments and therefore within the limits of the proposed bank protection.

Higher shear stresses on the outside of bends can result in additional scour that can undermine bank protection if not accounted for in the design. This type of scour is possible at Sites 3 and 4. There are two basic forms of the scour, which can occur. When the local shear stresses are high enough to cause general motion of the streambed sediments and material can be delivered from upstream into the area being scoured, live-bed scour occurs. Under this situation, the maximum depth of scour will be reached when the transported material out of the scoured section equals the material supplied from upstream. When the shear stresses are in general not high enough to cause motion of the streambed sediments and an insignificant quantity of material can be transported into the area being scoured, clear-water scour occurs. In this situation, the maximum depth of scour will be reached when the bed shear stress in the scoured area is less than that required for incipient motion of the bed material. Different analytical techniques are used depending on which type of scour is occurring.

To evaluate clear-water scour caused by shear stresses greater than that necessary for incipient motion, the depth of scour necessary to lower the local shear stresses to critical was computed. The analytical procedure used to compute the scour depth was developed as follows.

$$k = \frac{\tau_b}{\tau_c} \tag{4.13}$$

where τ_b is the shear stress at the location of interest (the outside of a bend) and τ_c is the critical shear stress required to initiate motion. Since shear stress is proportional to the square of the velocity,

$$k = \frac{v^2}{v_c^2} \tag{4.14}$$

where v is the average main channel velocity in the unscoured section and v_c is the velocity at incipient motion.

$$q = vy \tag{4.15}$$

where q is the unit discharge and y is the depth of flow,

$$y_c = \sqrt{k}\,y \tag{4.16}$$

where y_c is the new depth required to reduce the shear stress to critical. Thus the scour depth is given by:

$$d_s = (y_c - y) = (\sqrt{k} - 1)y \tag{4.17}$$

The shear stress on the outside of a bend (τ_b) was computed from the average bed shear stress using the relationship shown in Figure 4.49. The ratio of the shear stress on the outside of a bend to the mean shear stress is selected based on the bend sharpness, defined by the ratio of the radius of curvature to the channel top width. The average bed shear stress is the grain shear, τ_{gs}. τ_{gs} and τ_c were computed using the methods discussed in Section 4.4.

In a fashion similar to the clear water scour calculations, live-bed scour at bends was computed using the ratio of the shear on the outside of the bend to the average shear. The derivation assumes that the outside of the bend will scour until the shear stress reduces to the average shear stress, which is assumed to be in equilibrium with the sediment transport. The resulting relationship is:

$$d_s = (\sqrt{k'} - 1)y \qquad (4.18)$$

where

$$1k' = \frac{\tau_b}{\tau_{gs}} \qquad (4.19)$$

If sufficient coarse material is available in the channel bed, the process of channel armoring can limit the extent of scour that occurs on the outside of bends. The depth of scour necessary (to establish an armor layer sufficient to limit further scour) was computed using the following relation (USBR 1984):

$$y_s = y_a(\frac{1}{P_c} - 1) \qquad (4.20)$$

where y_a is the thickness of the armor layer and P_c is the decimal fraction of material coarser than the armoring size. The thickness of the armor layer was assumed to equal 2 times the critical particle size (D_c).

In straight reaches where the flow is aligned with the bank and no protrusions or obstructions exist to disrupt the flow the potential for local scour is low. The primary danger in this situation is for migration of the channel thalweg against the protected bank. In this case, the bank protection should extend below the channel thalweg by at least 3 to 5 ft to ensure that the protection is not undermined.

General degradation of the channel bed caused by an imbalance between the sediment supply and the transport capacity must be added to any local scour mechanisms to determine the required toedown depths (burial depths below grade) at each site. Based on the sediment routing calculations presented in draft American River report (RCE/Ayres 1993), maximum general degradation is approximately 0.5 ft for all of the sites, which is small in comparison to the computed local scour depths.

4.7.3. Results

The local scour calculations were based on hydraulics computed using the HEC-2 model. The model was changed to evaluate the scour potential for four scenarios using the 100-year existing conditions peak flows (Folsom Reservoir 115,000 cfs objective release and 400,000 ac-ft of storage). The peak discharge conveyed in the leveed reach under these conditions is 180,000 cfs (see Section 4.2.2). The four scenarios relate to different downstream stages and main channel Manning's n values as summarized in **Table 4.25**. These four scour analyses scenarios are different from the five design scenarios described in Section 4.2.

Table 4.25. Hydraulics Scenarios Used to Analyze Scour Potential at Each Site.		
Scenario	Downstream Stage, ft msl.	Main Channel Manning's n
1	Single valued rating curve 33.20	As calibrated
2	Lower bound rating curve 31.10	As calibrated
3	Single valued rating curve 33.20	Calibration - 0.005
4	Lower bound rating curve 31.10	Calibration - 0.005

Figure 4.69 shows computed main channel velocity profiles for the four scenarios. The velocities at the five bank protection sites range from approximately 6 to 13 fps. There is a 1.0 to 2.0 fps variation among the different scenarios at a particular location with the highest velocities associated with the lower main channel Manning's n values. Variations associated with the different downstream stage assumptions extend upstream to approximately RM 8. **Figure 4.70** shows computed hydraulic depth profiles for the four scenarios. Hydraulic depths range from approximately 30 to 45 ft at the five bank protection sites with an approximately 2-foot variation among the different scenarios at a particular site. The highest hydraulic depths are associated with the higher main channel Manning's n values.

Sediment size distributions for recent sediment samples collected along the Lower American River are shown in **Figure 4.71**. These distributions were used to compute grain shear stresses and to evaluate local scour at the bank protection sites. **Table 4.26** lists the individual sediment size distribution applied at each bank protection site and the corresponding D_{50} and D_{84} values used in the calculations.

Table 4.27 summarizes computed pier scour depths for the four scenarios at each structure that has the potential to cause problems with the proposed bank protection. Variability in the computed scour depths associated with the different scenarios range from 0.3 ft at the Highway 51 bridge to 1.4 ft at the UPRR bridge. The variability is small in relation to the computed scour depths.

Figures 4.72 through 4.76 show plots of the potential scour hole at the nearest unprotected pier at each site. The scour holes were plotted using the maximum computed scour depths given in Table 4.27. Scour holes are drawn with bottom widths (measured from one side of the pier) of zero and 1.0 times the scour hole depth and top widths equal to 1.7 and 2.7 times the scour hole depth. These plots give an indication of sites where pier scour can adversely affect the bank protection. Figure 4.72 shows that pier scour at the Old Sacramento Railroad bridge does not appear to present a problem for the proposed protection along the left bank. The scour hole associated with the second pier is more than 50 ft away from the toe of the slope and a scour hole will not develop at the first pier since it is located on the protected bank. Figure 4.73 shows that scour at the left-most pier of the UPRR bridge has the potential to undermine the adjacent bank protection. The protection in the vicinity of the UPRR bridge should be developed based on consideration of the computed scour depth. The edge of the estimated scour hole at the SPRR bridge is within 20 to 50 ft of the left bank (Figure 4.74). Figure 4.75 shows that the scour hole associated with the closest unprotected pier to the left bank at the Highway 51 (I-80) bridge is over 50 ft away from the toe of the slope and should not cause problems with the proposed bank protection.

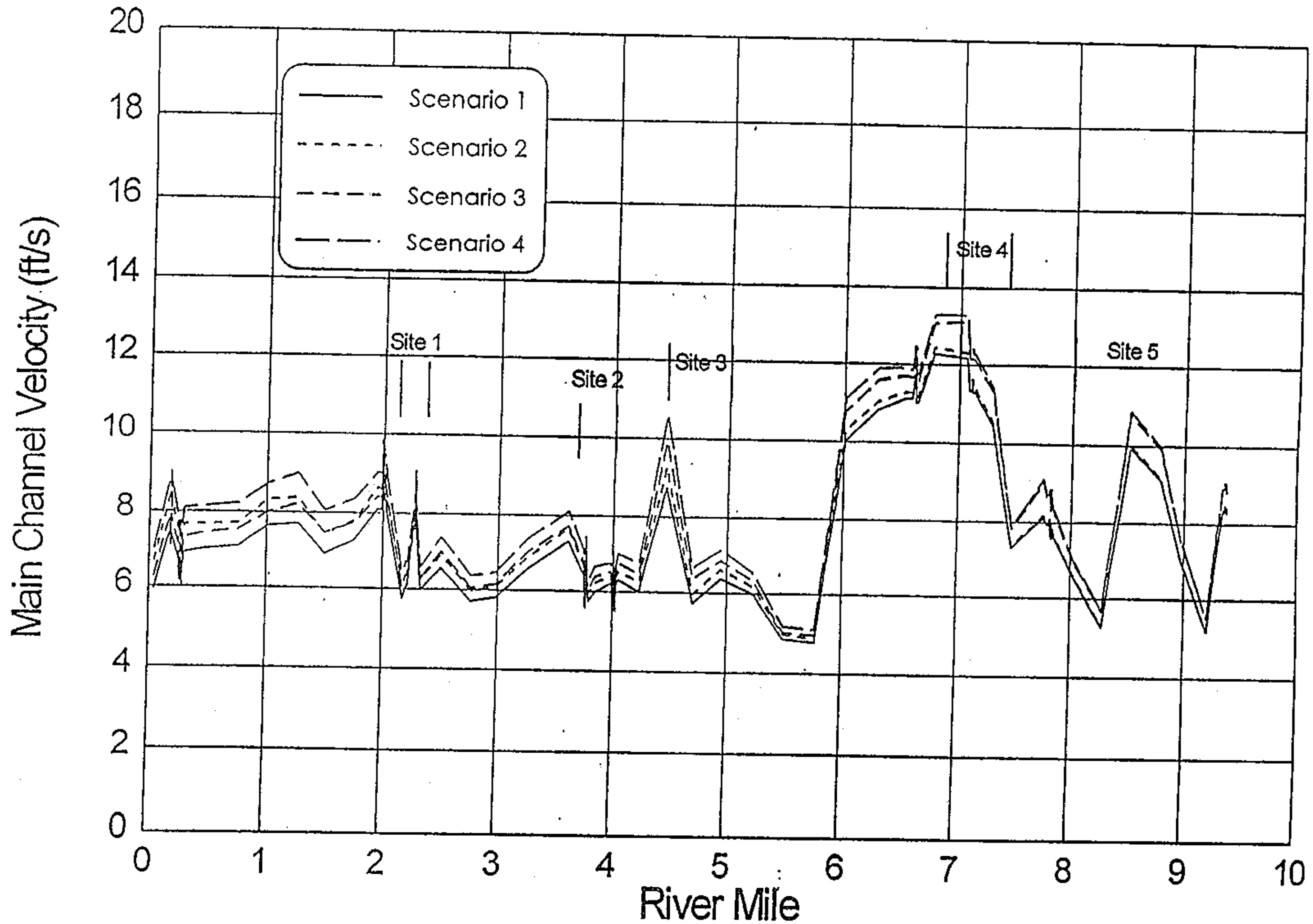


Figure 4.69. Main channel velocity profiles.

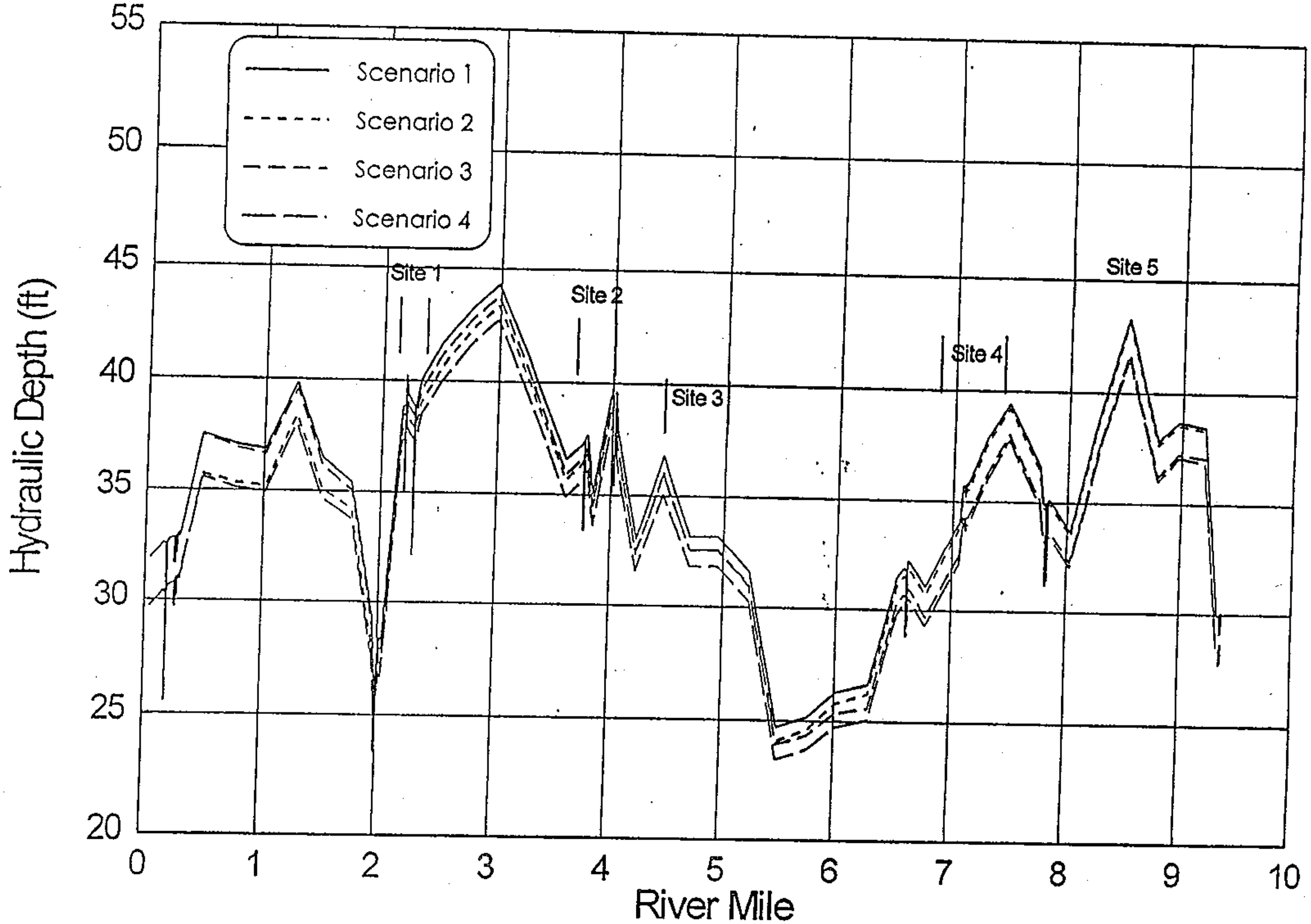


Figure 4.70. Hydraulic depth profiles.

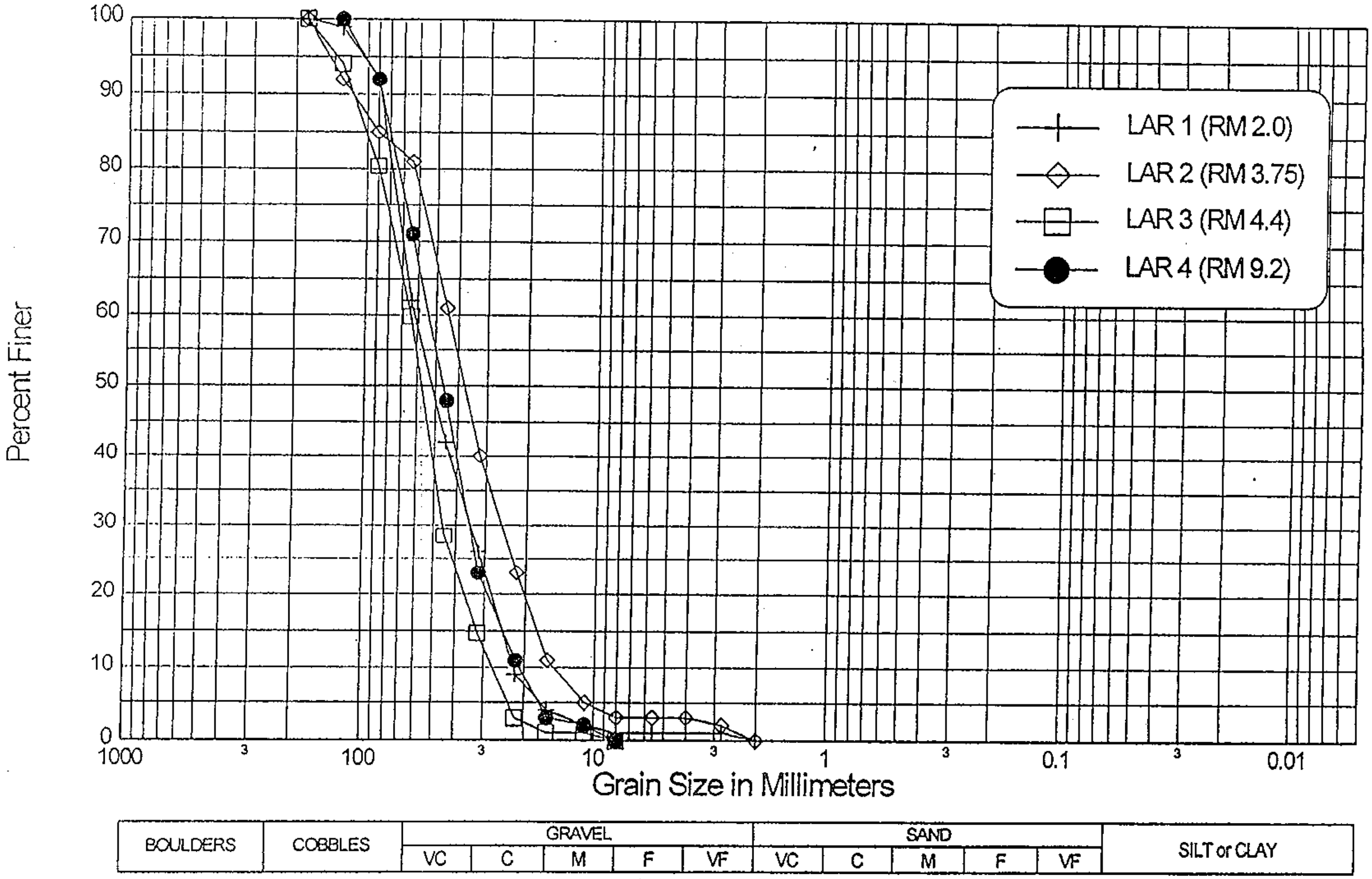


Figure 4.71. Sediment size distributions for recent sediment samples collected along the Lower American River.

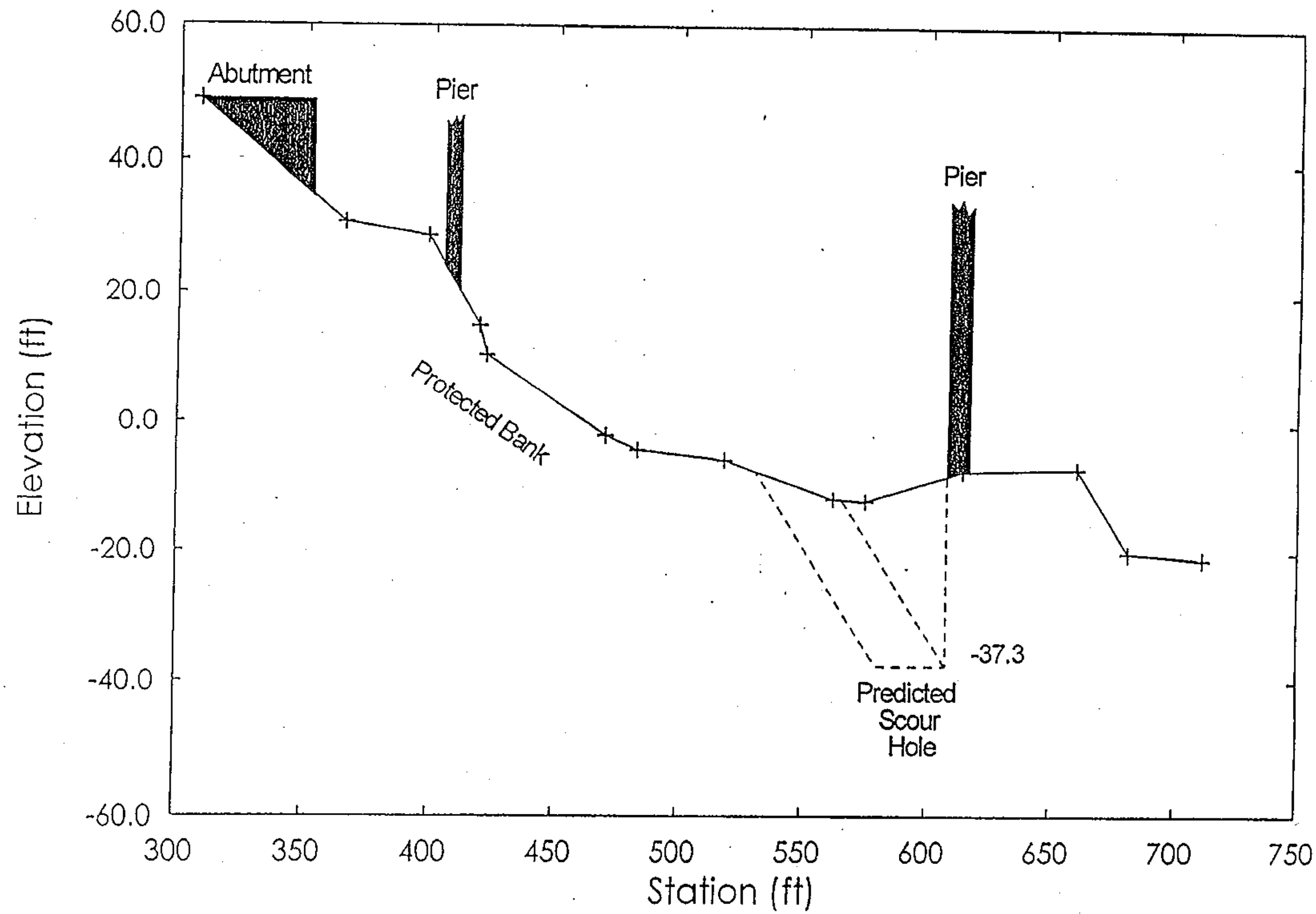


Figure 4.72. Predicted scour hole at the Old Sacramento Railroad bridge pier adjacent to the bank protection at Site 1.

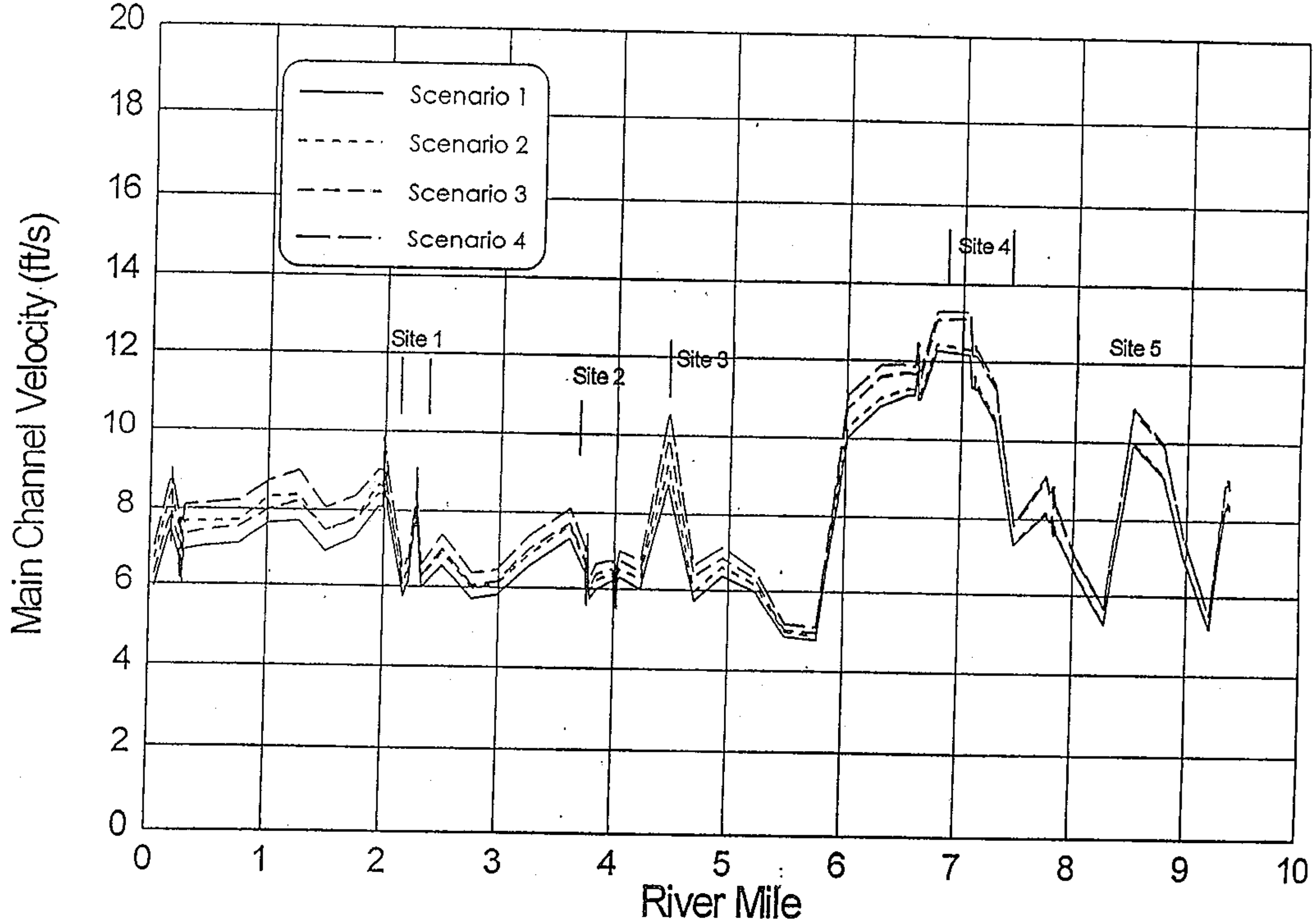


Figure 4.69. Main channel velocity profiles.

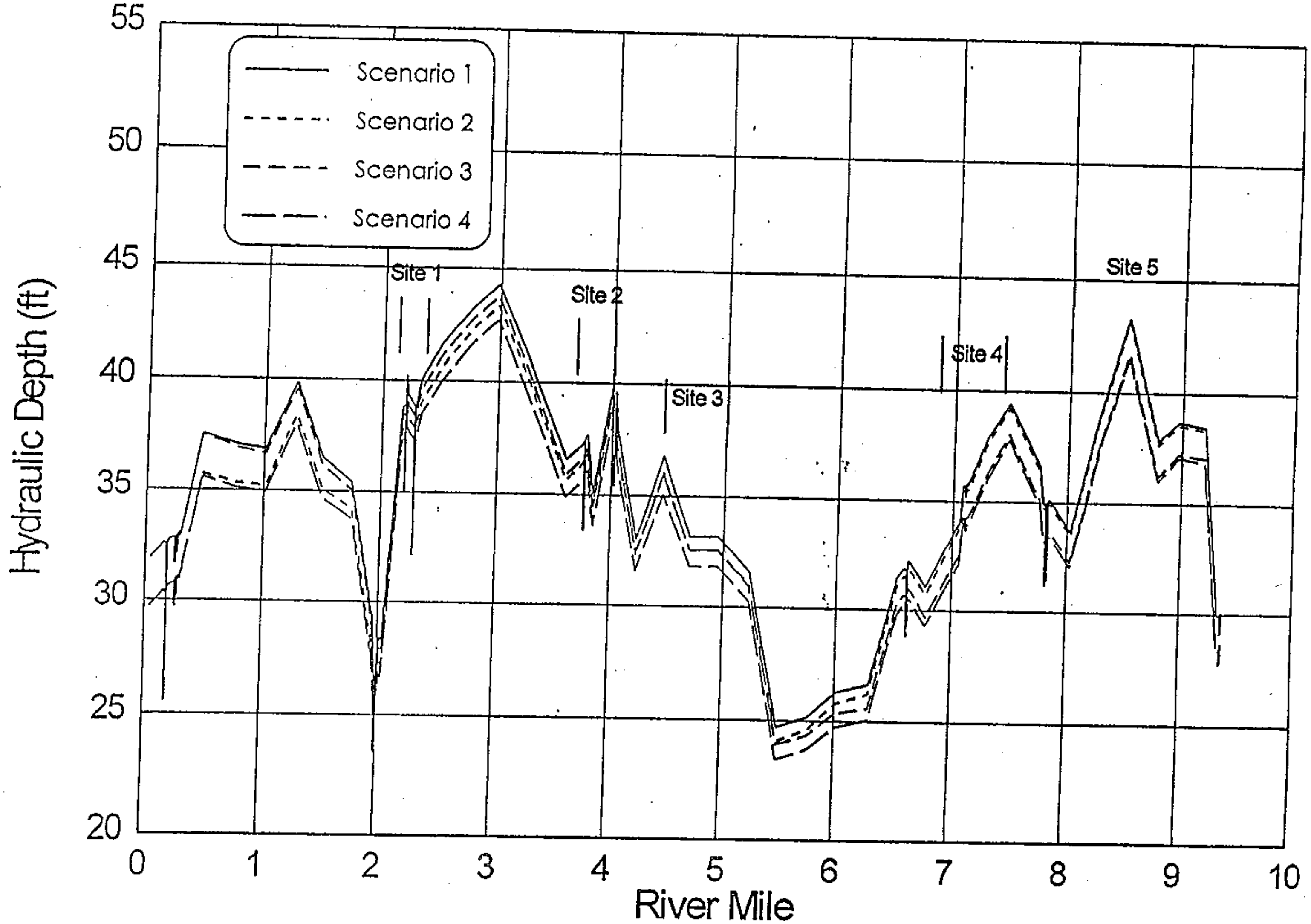


Figure 4.70. Hydraulic depth profiles.

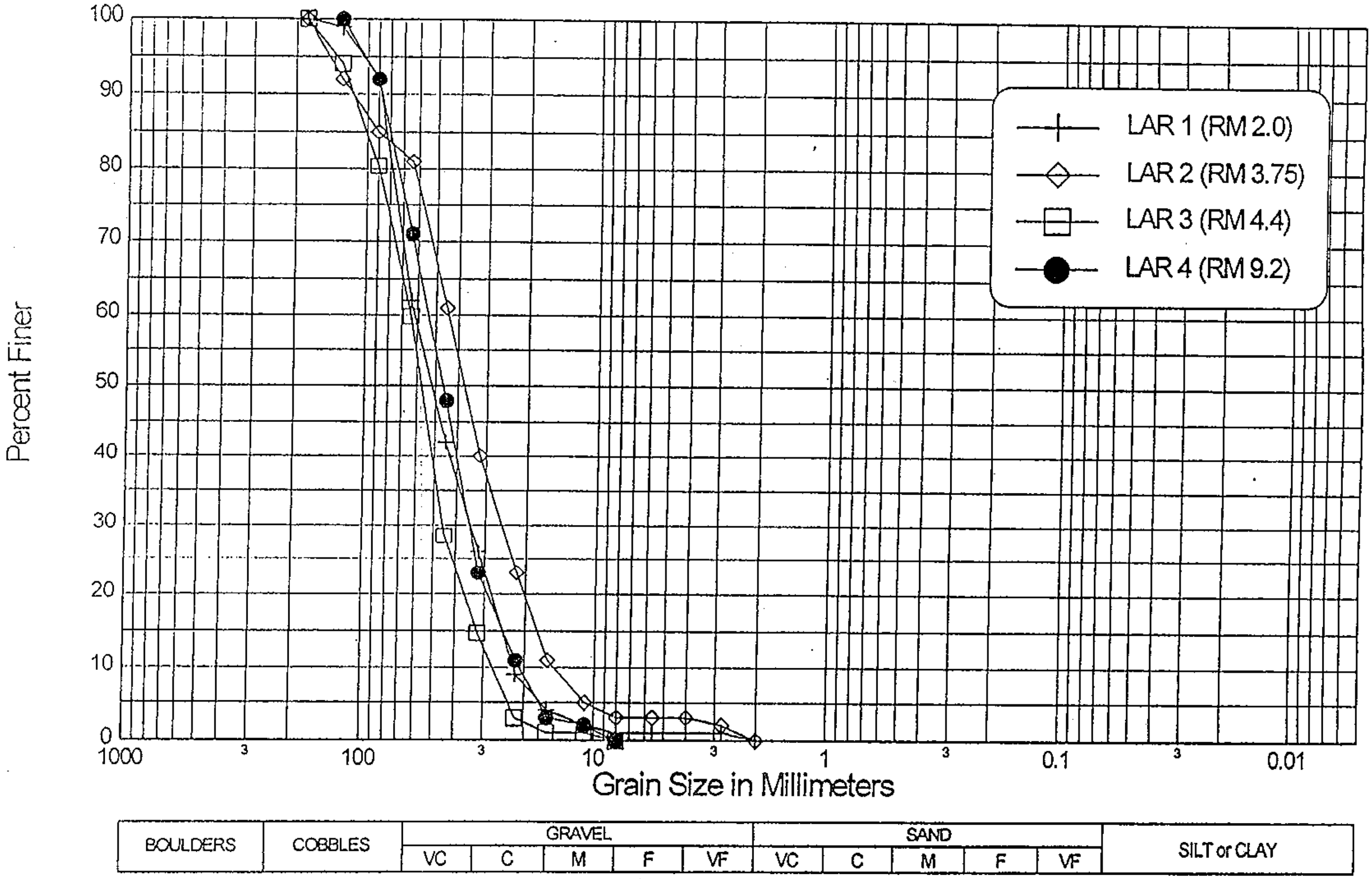


Figure 4.71. Sediment size distributions for recent sediment samples collected along the Lower American River.

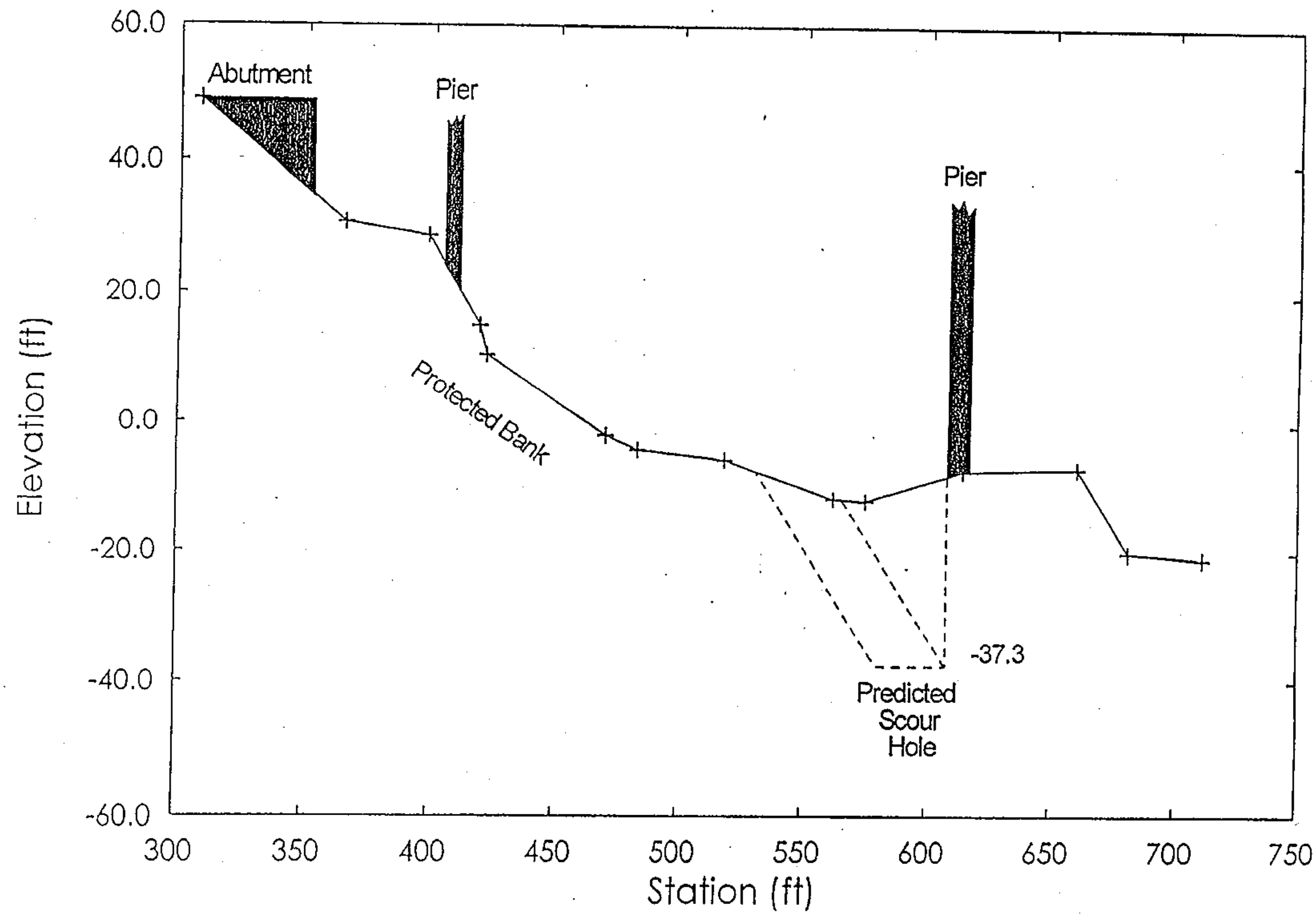


Figure 4.72. Predicted scour hole at the Old Sacramento Railroad bridge pier adjacent to the bank protection at Site 1.

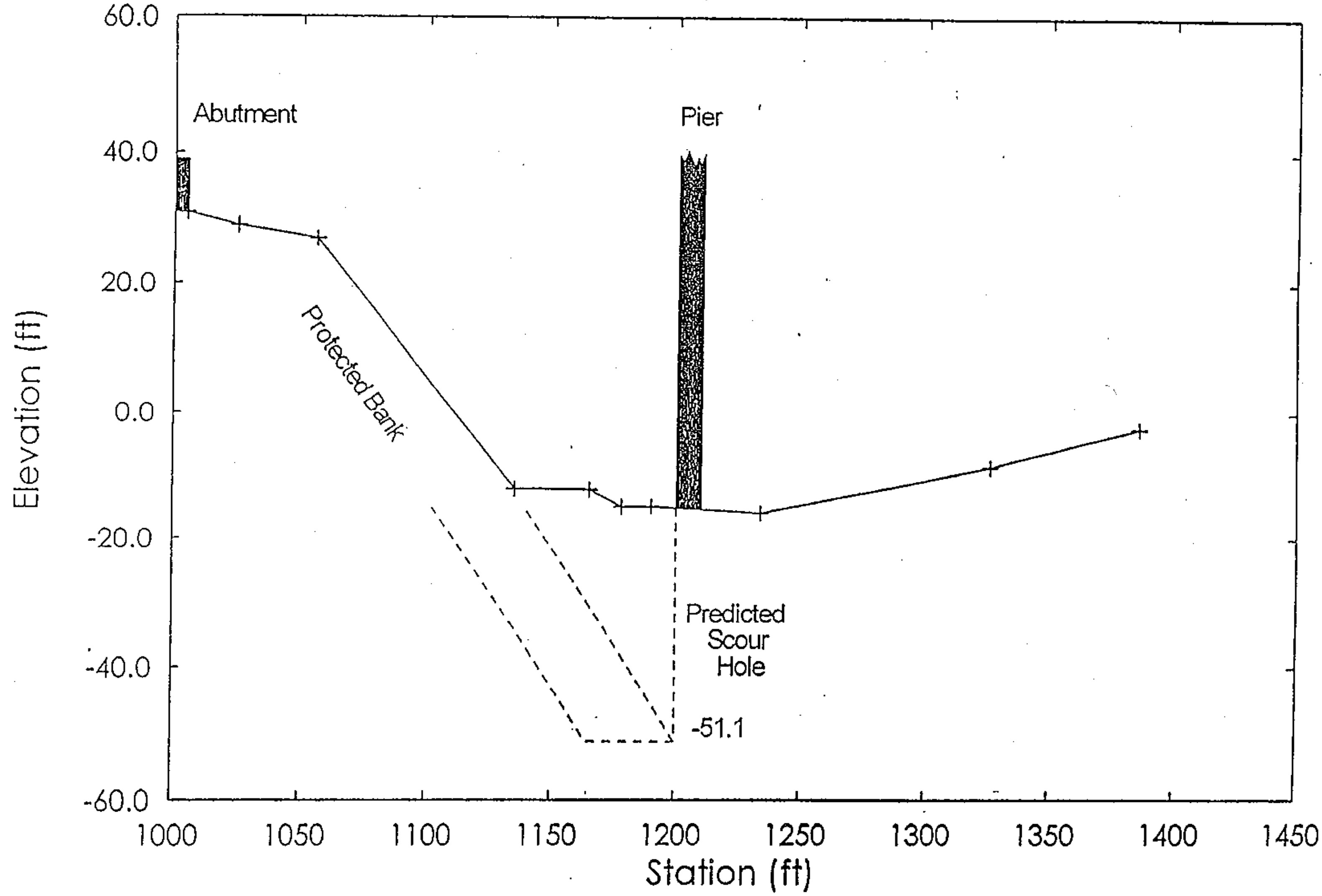


Figure 4.73. Predicted scour hole at the UPRR bridge pier adjacent to the bank protection at Site 1 (source MEI 1996).

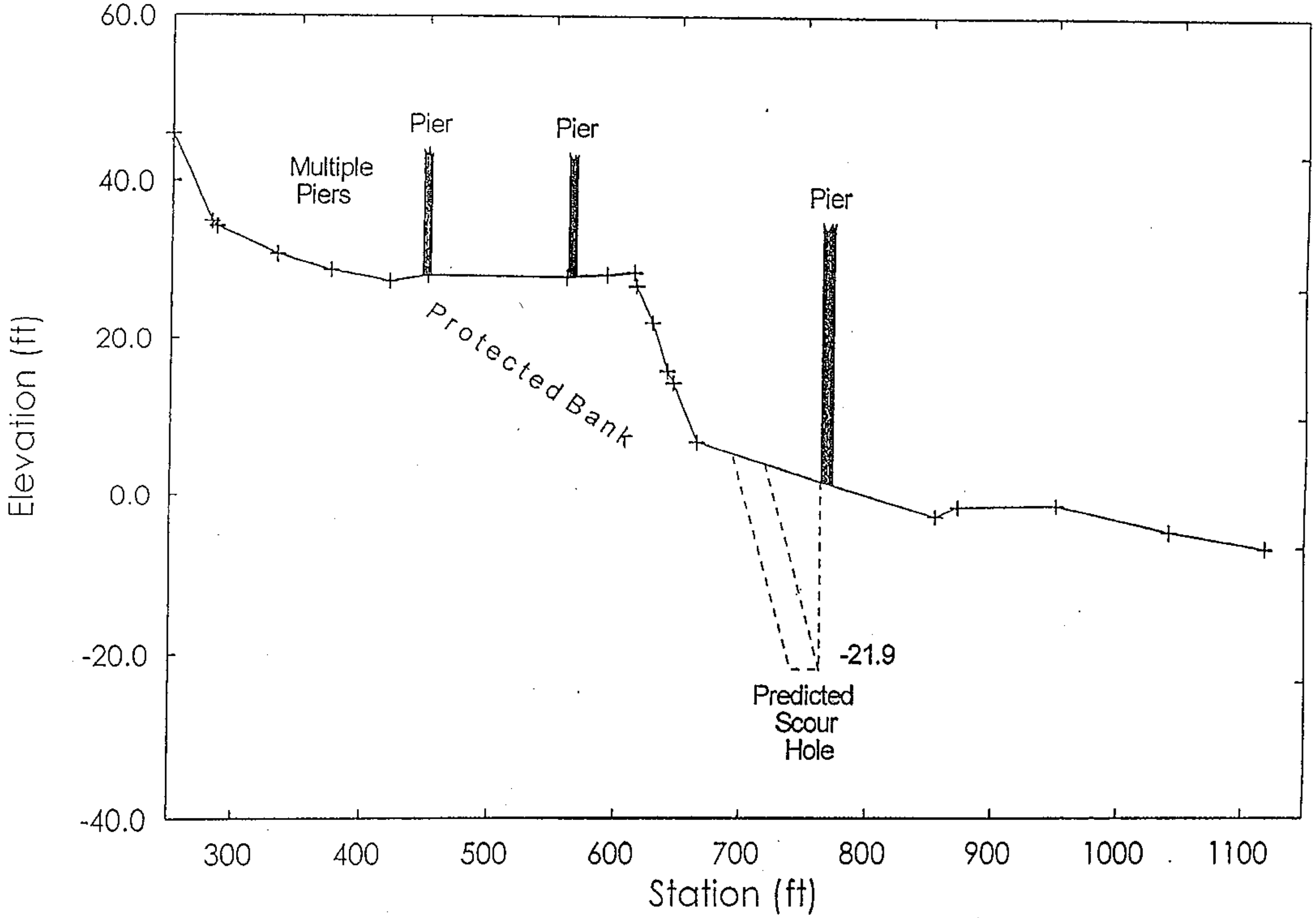


Figure 4.74. Predicted scour hole at the SPRR bridge pier adjacent to the bank protection at Site 2 (source MEI 1996).

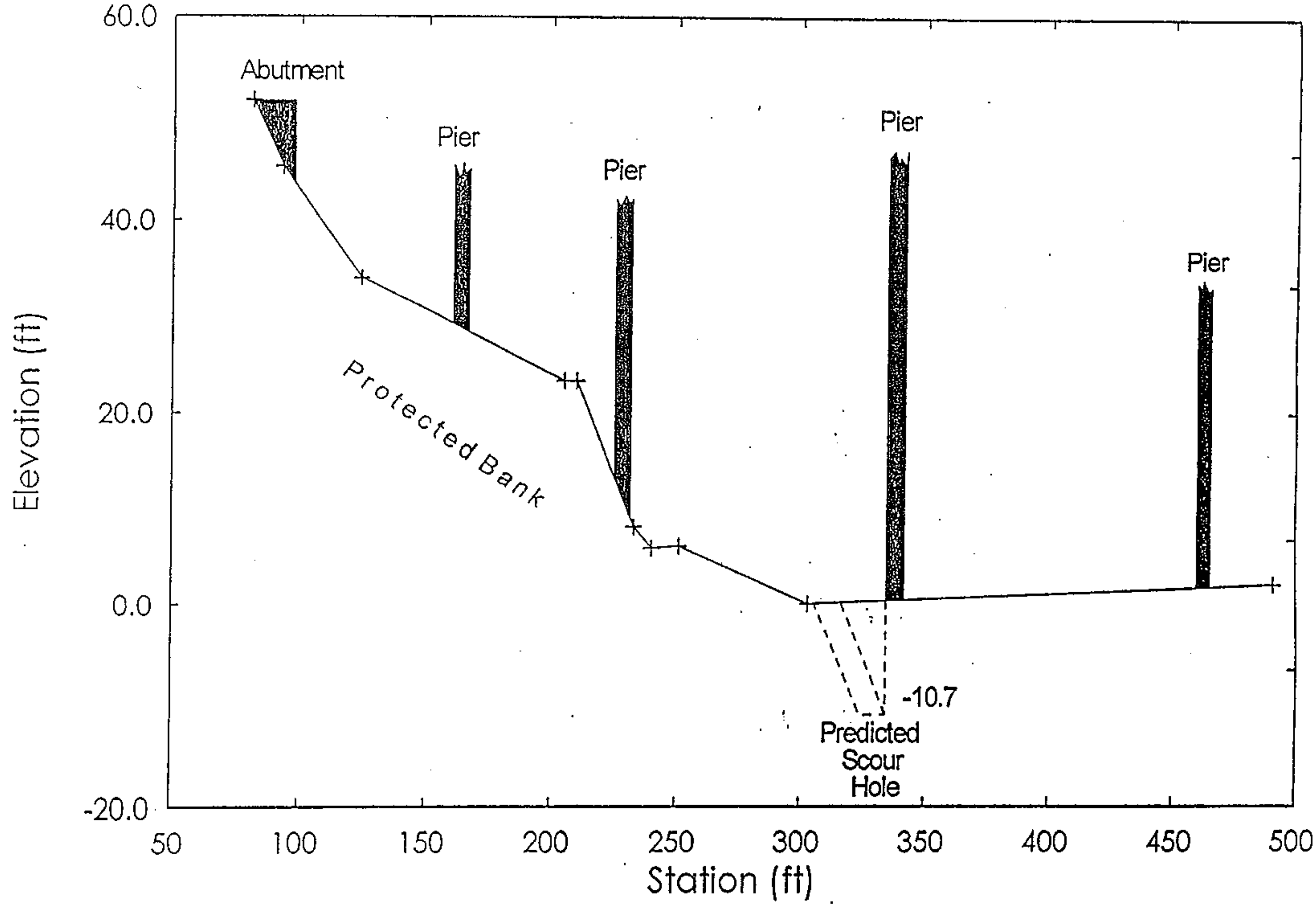


Figure 4.75. Predicted scour hole at the HW 51 (I-80) bridge pier adjacent to the bank protection at Site 2 (source MEI 1996).

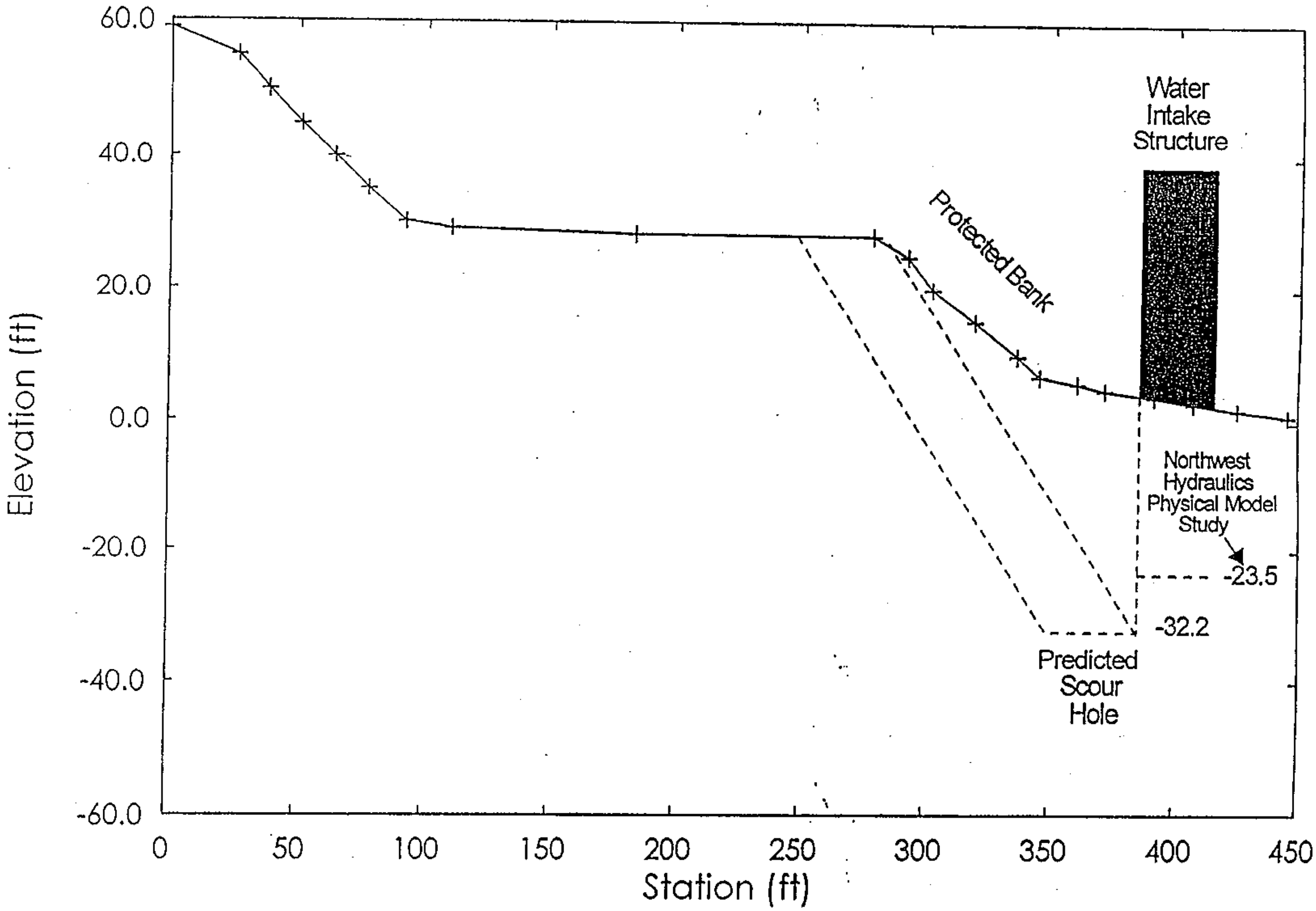


Figure 4.76. Predicted scour hole at the Fairbairn Water Treatment Plant water intake structure adjacent to the bank protection at Site 4 (source MEI 1996).

Table 4.26. Sediment Size Parameters Used in the Scour Analysis at Each Site.			
Site	Sediment Size Distribution	D_{50} (mm)	D_{84} (mm)
1	LAR 1	52.	81.
2	LAR 2	38.	80.
3	LAR 3	58.	98.
4	LAR 3	58.	98.
5	LAR 4	47.	78.

Table 4.27. Pier scour estimates.						
Site	Structure	RM	Computed Pier Scour (ft)			
			Scenario 1	Scenario 2	Scenario 3	Scenario 4
1	Old Sacramento RR	2.20	27.2	27.7	27.8	28.3
1	UPRR	2.28	34.7	35.3	35.4	36.1
2	SPRR	3.77	22.7	23.0	23.2	23.7
2	HW 51 (I-80)	4.02	10.8	10.9	11.0	11.1
4	Fairbairn Intake	7.38	35.6	35.7	36.5	36.7

Figure 4.76 shows that scour around the Fairbairn Water Treatment Plant intake structure is close enough to undermine the proposed protection along the left bank. Also shown on Figure 4.76 is the depth of scour determined from a physical model study of the intake structure conducted by Northwest Hydraulic Consultants, Ltd. (NWH 1990). The smaller scour depth estimated from the physical model study (28 versus 36.7 ft computed using the scour equations) may be due to the use of a smaller discharge (145,000 cfs versus 180,000 cfs). It is also possible that the scour hole did not reach equilibrium in the physical model study. Considering these factors, the results from the physical model study and scour equations are consistent.

The calculations for embankment scour at the protruding embankment just upstream of the SPRR bridge resulted in scour estimates ranging from 43.3 ft for Scenario 4 to 44.4 ft for Scenario 1. Results obtained from the abutment scour equations used in this calculation are generally considered to be conservative.

Table 4.28 summarizes the computed shear stresses used in the evaluation of bend scour at Sites 3 and 4. The results are reported at all cross sections along each site except the cross sections used to model the Guy West pedestrian bridge. **Table 4.29** lists the range of computed live bed and clear water scour depths at each site for each

Table 4.28. Computed Shear Stresses at Sites 3 and 4 Used in the Evaluation of Scour on the Outside of the Bends.											
Site	Cross Section (RM)	Ratio of the Radius of Curvature to the Top Width	Computed Grain Shear (lb/ft^2)				Computed Shear on Outside of Bend (lb/ft^2)				Critical Shear (lb/ft^2)
			Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 1	Scenario 2	Scenario 3	Scenario 4	
3	4.459	5.0	0.71	0.8	0.81	0.93	1.11	1.25	1.26	1.45	0.59
	4.703	2.8	0.31	0.34	0.33	0.37	0.58	0.64	0.62	0.69	0.59
	4.948	2.3	0.38	0.41	0.38	0.41	0.74	0.80	0.74	0.80	0.59
4	7.061	5.1	1.39	1.43	1.43	1.47	2.15	2.21	2.21	2.27	0.59
	7.086	5.0	1.19	1.21	1.21	1.24	1.86	1.89	1.89	1.93	0.59
	7.128	5.0	1.19	1.21	1.21	1.24	1.86	1.89	1.89	1.93	0.59
	7.294	5.1	1.01	1.03	1.07	1.10	1.56	1.59	1.65	1.70	0.59
	7.477	3.8	0.47	0.48	0.48	0.48	0.81	0.83	0.83	0.83	0.59

Table 4.29. Bend Scour Estimates.					
Site	Scenario	Range in Computed Scour Depths (ft)			
		Live Bed		Clear Water	
		Computed	Limited by Armoring	Computed	Limited by Armoring
3	1	9.1 - 13.2	0.4 - 4.8	0.0 - 13.7	0.0 - 4.8
	2	9.0 - 12.9	0.6 - 8.2	1.3 - 16.5	0.6 - 8.2
	3	8.9 - 12.9	0.5 - 8.3	0.8 - 16.8	0.5 - 8.3
	4	8.7 - 12.6	0.8 - 8.7	2.7 - 20.0	0.8 - 9.5
4	1	8.2 - 12.3	1.4 - 9.2	6.8 - 31.0	1.4 - 31.0
	2	8.2 - 12.2	1.5 - 9.2	7.3 - 31.6	1.5 - 31.6
	3	7.9 - 11.8	1.5 - 8.8	7.0 - 30.4	1.5 - 30.4
	4	7.8 - 11.8	1.5 - 8.8	7.0 - 31.0	1.5 - 31.0

scenario. The results are presented as computed using the equations and as limited by bed armoring. The live bed scour estimates range from approximately 8 to 13 ft; while the clear water scour estimates range from zero to approximately 32 ft. When bed armoring is considered, the range in live bed scour depths are reduced to approximately 0.4 to 9 ft. Although armoring limits the computed clear water scour depths in many cases, the overall range is still zero to approximately 32 ft. The bed of the Lower American River is generally immobile for discharges up to approximately 50,000 cfs with local areas remaining immobile through the 100-year event. Sediment routing calculations show that sediment transport rates are low and the bed is relatively stable with only local areas of aggradation and degradation. It therefore appears that clear water scour is the dominant scour mechanism and the higher computed scour depths should be used in design.

Table 4.24 listed thalweg realignment as the only potential scour mechanism for Site 5. Protection along this site should be buried to a depth at least 3 to 5 ft below the deepest portion of the channel bed at all locations. At all other sites, the toe-down should be at least to the depth of the predicted scour as well as 3 to 5 ft below the deepest portion of the channel bed.

The computed scour depths at all locations are ultimate scour depths computed assuming granular material in the bed and banks. The existence of cohesive or cemented materials would tend to slow down the rates of scour, but ultimate scour depths would be similar to those computed unless the cohesion or cementation were sufficiently erosion resistant to stop the scour altogether. Over the 50-year project life, existing cohesive and cemented materials are unlikely to significantly limit the ultimate scour depths.

4.8. Levee Analysis

In September 1987, Wahler and Associates issued a report on the American River Levees that concluded that the levees were generally unstable and did not meet the Corps of Engineers minimum safety factor of 1.4. The COE performed a study of their own and published the Feasibility Report, American River Investigation in December 1991. Generally it concluded that the levee system was stable for the original design flow of 115,000 cfs, but needed significant remedial work if flows were to be increased to 130,000 cfs or higher.

In response to the Corps report, a third report was prepared by WRC-Environmental (WRC-E) and Mitchell Swanson and Associates (MSA) that reviewed the COE Feasibility Report. Their primary conclusion was that there was little difference between the hydraulic characteristics of 115,000 cfs and 130,000 cfs. Therefore, the system is not safe for the design flow of 115,000 cfs.

The purpose of this section is to provide an independent technical review of the subject chapters of the COE Feasibility Report and the Review Report prepared by WRC-E and MSA. The scope of this effort involves a technical review of the COE stability and seepage analysis in the Feasibility Report. Our review and subsequent analysis was accomplished using the existing published data. No additional field investigations, boring, or soil testing has been performed by RCE/Ayres.

The COE (1991) report identified the following stability concerns associated with the American River levees:

1. Seepage and piping underneath the levees
2. Slope stability of the levees
3. Channel and bank erosion on the berms and the levees
4. Foundation liquefaction during a seismic event
5. Overtopping or adequate freeboard
6. Scour at bridge piers and other in-channel structures.

The review of the COE (1991) report prepared by WRC-Environmental and Mitchell Swanson and Associates identified the following issues that relate primarily to channel and levee erosion:

1. Lateral erosion is treated as a maintenance problem rather than a design problem. Repairs are installed as needed and on a piecemeal basis.

2. The hydraulic characteristics of the river are essentially the same for 115,000 cfs as for 130,000 cfs, yet the lower flow amount is considered to be the threshold above, which major improvements are required.

3. Existing bank protection works and rip-rap appear to be undersized for the forces present in the river.

4. The natural behavior of the river has not been accounted for in the COE (1991) report, and this has tremendous bearing on the erosional forces present.

5. The existing level of protection at 115,000 cfs is not reliable since failure almost occurred in the 1986 runoff event.

This section addresses two concerns, seepage and piping and slope stability.

The seepage and piping analysis performed by the COE in the 1991 report is a conservative evaluation of the stability of the American River levees. No new foundation investigation or soil mechanics testing was performed specific to this analysis. Stratigraphy and soil characteristics were taken from previous investigations. The analysis incorporated the following conservative assumptions to account for the limited data available.

1. The horizontal permeability was assumed to be four times the vertical.

2. Permeabilities were estimated at Kv=30 fpd and Kh=120 fpd (greatest test value was Kv=3 fpd for silty sand materials at 80 pcf density).

3. The foundation was assumed to be impermeable below 5 ft.

4. A minimum safety factor of three was required for stability.

Other assumptions that were used and that may not have a conservative effect were:

1. The levee and foundation soils (down to 5 ft) were considered to be uniform.

2. Only one levee and foundation cross section was used for the entire analysis.

In general, the results of the COE analysis are reasonable given the assumptions listed above. They show that seepage pressures and the potential for piping failures will go up significantly as the flows are increased above the 115,000 cfs level.

However, the analysis lacks adequate detail and site specific data to conclusively evaluate the relative stability of the entire leveed reach at the flow level of 115,000 cfs. Exit gradients at the landward side can be much higher when a thin confined layer of pervious materials exist either within the levee or the foundation. The stratigraphy of the section can be as important as the value of permeability selected. The permeability test data were based on two tests on remolded samples.

In discussions with American River Flood Control District (ARFCD), they indicated that in 1986 there were three areas where there was evidence of seepage along the American River levees. One area was upstream of Howe Ave (approximate RM 8) on the landward of the right levee. There were small boils that ARFCD controlled by using one or two high sandbag dikes. This was not considered to be significant problem by ARFCD. A second location was at RM 4.16 and was on the left side. This involved significant seepage and a sandbag dike was constructed to control the flow. It was later discovered that a corroded abandoned irrigation pipe under the levee was the major cause of the seepage and repairs have been made by ARFCD.

A third location was the Southern Pacific Railroad crossing near Business I-80. This was not a major problem according to ARFCD and it appeared that the seepage was coming through the old railroad fill portion of the levee. According to ARDCD, there were no serious piping problems associated with the 1986 high flows.

The slope stability analysis performed by the COE used the same levee cross section that was used in the seepage analysis. Strength parameters were based on existing triaxial tests for total strength parameters and direct shear tests for effective stress parameters. No new testing was performed for this analysis. RCE/Ayres checked the COE stability analysis by using a similar computer program (PCSTABL5). Computed safety factors are as shown in **Table 4.30** (failure circles that are shallower than 1 ft in depth are not shown).

Table 4.30. Comparison of RCE/Ayres and COE Safety Factors for Slope Stability Analysis.

River Flow	RCE/Ayres Safety Factor	ARWI Safety Factor
Q = 115,000 cfs	1.43	1.43
Q = 130,000 cfs	1.40	1.40
Q = 180,000 cfs	1.04	1.0

RCE/Ayres also checked stability for the sudden drawdown case on the riverward side of the levee. Using the same shear strength parameters and levels of saturation the following safety factors were shown in **Table 4.31**.

Table 4.31. Safety Factors for Sudden Drawdown Scenario.

River Flow	RCE/Ayres Safety Factor	ARWI Safety Factor
Q = 115,000 cfs	1.00	Not Evaluated
Q = 130,000 cfs	0.94	Not Evaluated
Q = 180,000 cfs	0.93	Not Evaluated

The COE report did not include results for the sudden drawdown case. However, the COE minimum required safety factor for a sudden drawdown is 1.0.

The process used for selecting strength parameters for the stability analysis was also reviewed. The COE values for effective stress strength parameters were derived from 10 direct shear tests on both remolded and undisturbed samples. The total stress strength parameters were derived from five consolidated undrained shear tests (with pore pressure measurements). Four of the tests were on remolded samples and one on an undisturbed sample. Both sets of data have wide scatter in the values and average values were selected by the COE for the levee and foundation soils. The COE stability analysis shows that the levees just meet the minimum safety factor using the selected average strength parameters. This means for sections that are weaker than average the safety factor can fall well below the required minimum.

RCE/Ayres also computed effective stress shear strength parameters using the triaxial test data and found the average values to be lower than those using the direct shear tests. **Table 4.32** shows the range of effective stress shear strength values for the selected foundation and levee soil densities.

The COE average value appears to be conservative when compared to the direct shear test values; however, they are higher than the average for the triaxial data. While the direct shear test is a widely used method for obtaining strength parameters, it is not considered as accurate as the triaxial test. Within the direct shear test, the failure plane is determined by the test itself rather than the soil properties and the problems associated with controlling volume change and drainage. This is of some concern in this case because the stability analysis demonstrates that COE minimum stability requirements are just achieved with the higher average strength parameters from direct shear tests values. Using average values from triaxial data, the safety factor is only 1.30 for the 115,000 cfs flow case. Similar reductions will result for the two higher flow cases.

Table 4.32. Comparison of Effective Shear Strength Values Developed from Direct Shear and Triaxial Tests.

Range of Effective Shear Strength Values		
	Shear Strength Foundation Soils @ 85 pcf	Shear Strength Levee Soils # 94 pcf
Direct shear test data	27° to 38°	28° to 39°
Corps recommended values	31°	34°
Triaxial shear test data	20° to 33°	28° to 36°
Average triaxial values	26.5°	32°

5. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

5.1. Summary

The American and Sacramento River California Project, that is the subject of the report is located in Sacramento, Placer, and El Dorado Counties, California. The purpose of the project is to increase the level of protection afforded by the downstream elements of the Sacramento River Flood Control Project (SRFCP) by construction of a flood control only, storage dam (dry dam) that is referred to as the Main Dam Element (MDE). Congress through the FY-1992 Defense Appropriations Act requested additional information on the project including potential modifications to the existing flood control levees along the Lower American River (Downstream Levees Element: DLE).

The **Main Dam Element (MDE)** project reach for this study extends from about RM 47.6 (location of proposed dry dam) on the North Fork of the American River upstream to about RM 72 which is the approximate elevation of the top of the dry dam (Elevation 930 ft). The North Fork reach includes the North Fork Dam (RM 52.5) and Lake Clementine that were constructed by the California Debris Commission (CDC) in 1940 to retain hydraulic mining debris. The project reach also extends up the Middle Fork of the American River from the confluence of the North and Middle Forks at RM 50.3 to RM 74 which is also the approximate elevation of the top of the dry dam. Significant amounts of hydraulic mining debris were also introduced into the Middle Fork and until a landslide stopped construction in 1940, the CDC was constructing a sediment retention dam at Ruck-A-Chucky on the Middle Fork. Approximately 255 million cu yd of hydraulic mining debris entered the channels of the Middle and North Forks. Placer mining for gold was also carried out extensively in both Forks. For the purposes of this study the channel stationing for the Middle Fork commences at RM 47.6 at the dry dam site and not at the confluence with the North Fork because the Middle Fork is the primary source of sediment to the dry dam site since Lake Clementine has a 100 percent trap efficiency for bed material derived from the North Fork upstream of the reservoir. The specific objectives for the MDE study were:

1. To review and circumstantiate the results of previous USBR studies of sediment yield at the dry dam site (0.27 ac-ft/mi^2/yr)

2. To determine average annual and design event (200-year) sediment inflow to the dry dam site

3. To determine sediment accumulation and its distribution along the project reach and at the dry dam site during the project life

4. To determine the potential for coarse bed material to enter the flood control sluices and/or diversion tunnel

5. To develop a conceptual design for structural measures to prevent coarse bed material from entering flood control sluices and the diversion tunnel.

The **Downstream Levees Element (DLE)** project reach for this study extends from the confluence of the Lower American River with the Sacramento River (RM 0) upstream to Nimbus Dam at RM 23. Sacramento River Flood Control Project (SRFCP) levees extend on the north (right) bank from the confluence with the Sacramento River to about RM 6. American River Project levees extend from RM 6 to RM 14. On the south (left) bank the SRFCP extend from the confluence to RM 12. Local levees are located from about RM 13 to RM 15 on the south bank. The characteristics of the channel and its floodplain have been impacted significantly by: (1) aggradation (15 to 30 ft) and degradation of hydraulic mining debris derived from the upstream watershed, (2) baselevel changes (8 to 10 ft) resulting from aggradation of the Sacramento River due to accumulation of hydraulic mining debris derived primarily from the Feather River and its tributaries, (3) dredge mining for gold from about Goethe Park to Nimbus Dam, (4) in-channel and floodplain sand and gravel mining that were most prevalent in the 1950s and are responsible for the development of split flow reaches as a result of channel capture of the pits construction of Project (1958) and non-Project levees, (5) emplacement of bank protection measures, (6) construction of numerous bridge crossings and consequent floodplain contractions, and (7) construction of Folsom and Nimbus Dams. The specific objectives for the DLE study were as follows:

1. To perform reconnaissance/feasibility-level geomorphic and sediment engineering investigations with respect to concerns regarding channel stability, bank protection requirements, and other sediment engineering concerns for objective releases from Folsom Dam of 115,000, 145,000, and 180,000 cfs

2. To identify potential problem areas with respect to lateral and vertical channel stability at each of the objective releases and identify potential mitigation measures for any such potential problems

3. To perform a comprehensive technical review of the American River Watershed Investigation California (ARWI) Feasibility Report (COE 1991) and evaluate the validity of comments in a review report of the AWRI Feasibility Report prepared for the Sacramento Area Flood Control Agency (SAFCA) by WRC-Environmental (WRCE) and Mitchell Swanson and Associates (MSA) (WRCE/MSA 1992)

4. To determine project features required to safely convey the three objective releases in the study reach based on the geomorphic, sediment engineering, channel stability analyses, and geotechnical analyses of levee stability

5. To develop recommendations for future technical studies, (including geomorphic, hydraulic, and geotechnical) to be performed during Preconstruction Engineering and Design (PED) to address concerns raised by this study.

The investigation of both the MDE and DLE involved geomorphic, hydrologic, hydraulic, and sediment transport analyses. Field work for both elements was conducted in May of 1993. For the MDE, the Middle Fork was traversed by raft from Indian Bar (RM 74) to the dry dam site. Sediment samples were collected, and Wolman Counts were made on the bed and bar materials. Sediment sources, including tributaries and slope failures were recorded, as were the locations of all significant hydraulic controls. A detailed topographic survey of the Mammoth Bar site and Murderer's Gorge (RM 52.4) was then conducted to enable subsequent hydraulic and sediment transport analyses to be carried out to better define the roles of local hydraulic controls in canyon-bounded river segments on downstream sediment delivery. During the design event the backwatered reach upstream of the Gorge causes 92 percent of the inflowing sediment load to be deposited.

A significant portion of the deposited sediments are then re-entrained by succedent flows and transported downstream of the Gorge. This process is repeated at many different locations at different scales depending on the magnitude of the hydraulic controls. A sedimentation survey of Lake Clementine was carried out to develop watershed sediment yield rates in a watershed that had been severely impacted by hydraulic mining. The sedimentation data indicated that over the 54-year life of the dam (1939-1993) the annual sedimentation rate was 0.22 ac-ft/mi^2. A hydraulic model (HEC-2) and a sediment transport model (HEC-6, RCE-modified version) were compiled for the Middle Fork from the dry dam site to RM 72.9 (Horseshoe Bar Tunnel) and the North Fork from the dry dam site to the North Fork Dam.

For the DLE, the project reach was traversed by jetboat, and 10 cross sections were surveyed at locations originally surveyed in 1987, where possible. Wolman Counts were made at bar-head locations where there was field evidence that sediments had been transported and deposited during the 1993 flows, peak discharge of 16,200 cfs. If required, previously conducted field inventories of the sites of bank erosion, bank protection, locations of Pleistocene-age bedrock outcrop were checked and modified. A hydraulic model (HEC-2) and a sediment transport model (HEC-6, RCE-modified version) were compiled for the project reach from the Sacramento River confluence to Nimbus Dam. Analyses of both vertical and lateral stability of the channel within the project reach were conducted for the various flow scenarios. Specific analyses of bridge scour potential were conducted at each of the bridges for the different flow scenarios. Additionally, scour analyses were conducted for 5 bank protection sites.

Hydrologic data for both the MDE and DLE were provided by the Sacramento District. Total inflow and outflow hydrographs and a reservoir pool stage hydrograph for the design storm (200-year) were provided for the MDE. In addition, a single-valued, stage-discharge rating curve for the proposed sluice configuration was provided for establishing pool elevations for flows other than the design storm. Hydrographs for the five design scenarios for the DLE were developed to bracket the range of potential impacts of the project on the river's geomorphic, sediment transport, and channel stability (vertical and lateral) characteristics. The scenarios included: (1) the existing condition with an objective release (OR) of 115,000 cfs and 400,000 ac-ft of flood control storage (FCS) in Folsom reservoir, (2) FEMA 100-year scenario with an OR of 145,000 cfs and 400,000 ac-ft of FCS, (3) FEMA 100-year with an OR of 115,000 cfs and 590,000 ac-ft of FCS, (4) 125-year with an OR of 180,000 cfs and 650,000 ac-ft of FCS, and (5) recommended project (dry dam) with an OR of 115,000 cfs and 400,000 ac-ft of FCS.

5.2. Conclusions

The geomorphic, hydrologic, hydraulic, sediment transport, and geotechnical analyses conducted for this investigation enable a number of conclusions to be drawn regarding the objectives of the study (Section 5.1). Because the two elements of the study are essentially stand-alone projects the conclusions are addressed separately.

5.2.1. Main Dam Element

The following conclusions can be made regarding sedimentation issues in the Middle and North Forks of the American River and at the proposed Auburn dry dam site:

1. Geologic setting, bedrock outcrop patterns, and other non-fluvial processes such as landslides, rockfalls, and debris flows have a significant effect on sediment transport and deposition within the canyon-bounded reaches. Sediment transport and deposition are controlled by local hydraulic conditions whose effects vary temporally and spatially throughout the project reach (also during the course of a single flood event).

2. The erosion resistance of the bedrock that bounds the Middle Fork controls valley width and the potential for sediment storage. Metavolcanic rocks and ultramafic rocks bound the narrower subreaches with little sediment storage potential (RM 74-70, RM 61.2-59.3, RM 52,4); and metasedimentary rocks tend to bound the wider subreaches where there is sediment storage (RM 70-61.2, RM 59.3-52.4, RM 52.4-50.3).

3. The tunnel at Horseshoe Bar (RM 73) on the Middle Fork effectively prevents bed material delivery to the downstream reaches from the upper watershed of the Middle Fork. Consequently, bed material sediment delivered to the lower reaches must be derived from sources downstream of RM 73.

4. Landslide Rapid at RM 61.2 and Murderer's Gorge at RM 52.4 on the Middle Fork have significant effects on downstream sediment delivery during the design storm because of the backwater conditions that they generate. A detailed analysis of the effects of the Murderer's Gorge contraction indicated that about 92 percent of the inflowing bed material load is deposited in the upstream backwater effected reach during the design event. Modeling of 2 years of succedent flows indicates that a high proportion of the deposited sediment is reentrained and transported downstream of the contraction.

5. Both the Middle an North Forks were severely impacted by hydraulic and dredge mining, which caused significantly increased watershed sediment yields historically. Currently, the historical mining effects are greatly diminished because of a number of factors. These include the construction of North Fork Dam in 1939, which has a 100 percent trap efficiency for bed material. Annual sediment yield over the life of the dam has been 0.22 ac-ft/mi^2 (Table 2.2). On the Middle Fork, the tunnel at RM 73, Landslide Rapid, Murderer's Gorge, and other local hydraulic controls reduce the amount of sediment that is delivered downstream. Comparison of c. 1938 aerial photographs with present conditions indicate that sediment yield to the dry dam site is low under present conditions.

6. Landslides of varying magnitudes and types constitute a significant episodic sediment source to the project reach. The majority of landslides occur in colluvial materials and are relatively small scale. A number of very large prehistoric landslides have been identified within the project reach. Reactivation of these landslides could occur under project conditions. Of particular concern are the landslides on the left (south) bank of the Middle Fork in the vicinity of Poverty Bar at RM 57 and the left bank at the confluence of the Middle and North Forks (RM 50.3). Slope failure at both locations may involve prehistoric fault modifications of the failing materials (Figure 2.8).

7. The USBR estimate of annual sediment yield for the Auburn Dam site based primarily on reservoir sedimentation data was 0.27 ac-ft/mi^2, but the analyses conducted for this study indicate that the annual sediment yield is much lower and is on the order of 0.05 ac-ft/mi^2.

8. The best estimate of annual bed material sediment yield at the dry dam site is 13,500 tons, but the potential range of values is 6,750 to 27,000 tons (Table 3.4).

9. The best estimate of the design storm bed material sediment yield at the dry dam is 70 tons, but the potential range of values is 35 to 140 tons (Table 3.4).

10. The best estimate of the project life (50 year) bed material sediment yield to the dry dam site is 675,000 tons, but the potential range of values is 338,000 to 1,350,000 tons (Table 3.4).

11. During the design storm event about 80 percent of the sediment passing through the sluices at the dry dam is in the very coarse sand to fine gravel size range, and the remainder is composed of medium to coarse gravels (Figure 3.21). Because of upstream sediment storage during the design event, the sediment volume passing through the sluices during the succedent flows is much higher and the sediments are much coarser (65 percent medium to coarse gravels, 4 percent cobbles, and 31 percent coarse sand and fine gravel).

5.2.2. Downstream Levees Element

The analyses conducted during this investigation permit the following conclusions to be drawn regarding channel stability (vertical and lateral), bank protection, bridge scour, and levee stability issues associated with the five design scenarios for the Lower American River:

1. The existing channel and floodplain conditions have resulted from the combined effects of a number of man-induced perturbations. These include hydraulic mining, dredge mining, sand and gravel mining, levee construction, bank protection, and flow regulation. The highly irregular longitudinal thalweg profile (Figure 2.26) is controlled by outcrops of erosion resistant Pleistocene-age bedrock units upstream of Goethe Park (RM 14), and as a result, little further degradation is expected to occur. Downstream of RM 14 Pleistocene-age, bedrock crops out in the bed of the channel at RM 9.5 just upstream of Watt Avenue (Figure 2.27). An abandoned channel fill that is controlling the local bed elevation at about RM 6 may erode in the future, but its effects on the vertical stability of the river upstream will be minimal.

2. Hydraulic analysis of the reach indicates that bankfull discharge from RM 0 to RM 13 is on the order of 50,000 cfs increasing to 100,000 cfs between RM 13 and RM 18 and reducing to 50,000 cfs from RM 18 to RM 23 (Figure 4.23).

3. Although the hydraulic analysis (Figure 4.24) indicates that the levees will contain a discharge of 180,000 cfs, this conclusion should be viewed with caution because of uncertainties associated with the location of the cross sections and the levee profiles.

4. Incipient motion analyses indicate that the bed material in the project reach is generally immobile at discharges less than 50,000 cfs. As discharge increases the bed material is mobilized; but at some locations, the bed is immobile even at a discharge of 180,000 cfs (Figure 4.31).

5. Analysis of the vertical stability of the bed of the channel indicates that the bed is relatively stable under all of the flow scenarios, even though there are local areas of aggradation and degradation under each of the scenarios (Figure 4.39; Table 4.10).

6. Sediment transport routings indicate that the bed material sediment yields to the Sacramento River under all of the flow scenarios are extremely low (Figure 4.47) ranging from a maximum of 1,724 tons in scenario 4, to a minimum of 1,570 tons in scenario 2. Under existing conditions (scenario 1), the yield is 1,584 tons. Under all of the scenarios, the vast majority of sediment is in the size range of 1-8 mm (Table 4.15).

7. Bank work index values indicate that on the basis of the integration of the flow duration curve a threshold value for bank erosion is about 100. Utilizing the weighted average of various return period flood events the threshold value is about 50 (Figures 4.50, 4.51). The difference among any of the four scenarios and the existing condition in the leveed reach are insufficient to increase the work index values above the threshold value of 50. For weighted average conditions, scenarios 3 and 4, which increase the storage in Folsom Reservoir, appear to reduce the potential for bank instability in the downstream 5 miles of the reach. Scenario 5, (the dry dam) increases the potential for bank instability in the lower 5 mi.

8. Based on the SRBPP criteria for protection of the levees, there are only 4 locations within the leveed reach that qualify for high priority status. These include RM 2.1L, RM 4.8L, RM 8.6R, RM 10.3R. Because of the risk to the levees and the fact that both sites are gaps in existing revetments, full bank rock revetment is recommended for RM 2.1L and RM 4.8R. Full bank rock revetment of stone dikes could be utilized at the other two sites. Approximately 1,500 ft of damaged revetments require rehabilitation.

   Based on Lower American River criteria, additional bank protection needs were identified. These include; 1,400 LF requiring immediate protection (Type 1a), 9000 LF requiring protection within 12 years (Type 1b), 5,000 LF of repair of existing revetments (Type 1c), 1,100 LF of bridge abutment protection, and 2,500 LF of new bridge protection (Type 1a). Within the project life it is expected that a further 5,400 LF of bank protection will be required.

9. Evaluation of bridge pier scour potential at 18 bridges within the project reach using FHWA procedures indicated that under existing conditions (scenario 1) there is significant scour potential at all of the bridges. The remainder of the scenarios do not significantly alter the computed pier scour depths (Tables, 4.19, 4.20, 4.21). Abutment scour estimates at four bridges indicate that there may be an abutment scour problem under existing conditions, which is not exacerbated by the other scenarios (Table 4.22).

Scour analyses at 5 high-priority bank protection sites identified by the COE, DWR, and SAFCA, for a 100-year flood event under existing conditions (Scenario 1) shows that the sites are influenced by pier and abutment scour processes at bridge crossings. Sites 3 and 4, located at RM 4.40L and 6.80L are also located in bendways and may experience bend scour from 9.5 (limited by armoring) to about 32 feet, respectively. All of the sites are susceptible to thalweg realignment and in addition to provisions for pier, abutment, and bendway scour, the protection at each site should extend to 3 to 5 feet below the deepest position of the channel bed.

10. Seepage and piping problems in the levees will occur as the flows increase above 115,000 cfs. Stratification in levee foundation materials will further lower the factor of safety. Since average strength values were utilized in the slope stability analysis, locations with weaker soils will have safety factors below COE minimum values. Use of triaxial shear test data rather than direct shear test data used by the COE (Table 4.25) will also cause a reduction in the minimum safety factor. Analysis of the effects of sudden drawdown on riverward slope stability indicate that safety factors are marginal at 115,000 cfs and decrease at higher flows (Table 4.24).

11. Qualitatively, the potential impact of any of the four alternative project scenarios on the stability of the Sacramento River is dependent on the need for modifications to the Sacramento Weir. All four of the alternative scenarios will increase the amount of flow (total volume and/or peak discharge) in the leveed reach of the Lower American River. This occurs because under existing conditions, significant flow loss occurs during the 100-year flood in the reach upstream of the levees, which reduces the amount of flow passing through the leveed reach. In addition, it is presumed that levees not meeting the freeboard requirements would be raised in conjunction with implementation of any of the alternative scenarios. Without modification to the weir, according to analyses performed by Murray, Burns, and Kienlen, increased flows in the leveed reach associated with the various scenarios cannot be passed into the Sacramento River at stages below the current design stage for the NEMDC. Modifications to the weir would change the flow distribution between the mainstem Sacramento River and Yolo Bypass (Joe Countryman, personal communication). If the weir is maintained at its current capacity, more flow will probably be forced downstream in the Sacramento River, which would increase flood stages and stress on the levees and channel bed. The potential for instability associated with the increased stress cannot be evaluated without more detailed information. Because of the relatively low sediment yield from the American River to the Sacramento River under all scenarios, including existing conditions, changes in sediment transport characteristics associated with the scenarios are believed to insignificant.

5.3. Recommendations

Based on the analyses conducted for both the Main Dam Element and the Downstream Levees Element, the following recommendations are made with respect to sedimentation issues at the MDE and channel stability issues for the DLE.

5.3.1. Main Dam Element

The sediment transport analyses performed for the Main Dam Element considered the following conditions: (1) on an average annual basis, (2) for the 200-year design storm, and (3) for accumulated conditions over a 50-year project life based on the average annual results. A significant finding of this study is that the amount of bed material sized sediment brought into the study reach on an average annual basis is approximately in balance with the amount transported past the dam site for both base and dry dam conditions (removal of approximately 700 tn/yr for base conditions and accumulation of approximately 1,100 tn/yr for dry dam conditions over the approximate 22-mile long reach). For design storm conditions, significant sediment accumulation occurs in the reach under both conditions (295,000 tons for base conditions versus 560,000 tons for dry dam conditions). This result indicates that hydraulic controls in the reach cause accumulation of sediment during high flows even under base conditions. In addition, the model results indicate that the zones of accumulation during large flows tend to scour under lower flows.

The average annual analysis was based on HEC-6 sediment routing results for a 4-year period with 2 years of above average flows and 2 years of below average flows. The peak discharge during this portion of the simulation was approximately 18,000 cfs; thus the results do not adequately account for the occurrence of larger storms between the 18,000 cfs peak and the 200-year design storm. Because of the demonstrated dynamics of areas affected by hydraulic controls (e.g., Mammoth Bar and Landslide Rapid), the distribution of sediment in the area affected by the dry dam pool and its tendency to progressively move downstream toward the dam over the project life is highly dependent on the actual number and sequencing of large storm events up to and including the design storm.

Based on the above discussion, it is recommended that additional analyses be performed to simulate sedimentation conditions in the study reach for a long term series of normal flows in combination with larger storm events. It may be necessary to evaluate several possible combination to determine worst case conditions. This would involve analysis of the long term flow record for the study reach and development of one or more flow sequences that could occur and applying the HEC-6 model to those flow sequences. The result would provide a better indication of the anticipated sedimentation conditions within the area affected by the dry dam pool and amount of sediment that would eventually work its way downstream to the dry dam sluices.

Countermeasure for Reducing the Potential for Entrainment of Coarse Sediment into the Sluices during High Head Conditions

The sedimentation analysis presented in Chapter 3 indicates that approximately 70 tons of sand and gravel will be carried though the dry dam sluices during the design event. This material is derived from re-entrainment of sediment from the channel bed as the pool elevation reduces to near run-of-the-river conditions on the recessional limb of the hydrograph. The dry dam pool effectively prevents inflowing sediment load from reaching the sluices during higher flows. While this material may cause abrasion of the sluices, it is transported toward the end of the hydrograph when the head on the dam, and thus velocity through the sluices is relatively low. If the tunnel is lined with abrasion resistant material, the amount of erosion should not be excessive. In addition, since the transport occurs at near the end of the hydrograph, it may be possible to inspect the sluices prior to the occurrence of the next storm.

Of greater concern is the potential for sediment accumulation near the sluice inlets during low flows and subsequent re-entrainment of this material during high head conditions by local acceleration of the flow into the sluices. In concept, the preferred method for minimizing this problem is prevent sediment accumulation within the acceleration zone immediately upstream of the sluice inlets. A conceptual sketch of this method is shown in **Figure 5.1**. A sediment trap is recommended some distance upstream of the sluice inlets to trap inflowing sediment during run-of-the-river conditions before it reaches the sluice inlets. In addition, a concrete apron should be placed on the channel bed between the sediment trap and the sluice inlets to prevent local scour in this area and to facilitate removal of any sediment that passes through the sediment trap. The distance among the sediment trap should be place upstream of any local effects associated with acceleration of flow into the sluices under design head conditions. Due to uncertainty regarding the extent of these effects, it may be necessary to perform physical modeling to determine the distance. The potential for cavitation and damage to the concrete apron should be evaluated and appropriate measures (e.g., material types, placement of the apron below the invert of the sluice inlet). These are included in the final design to mitigate these damages.

Table 5.1 summarizes the volume of sediment predicted to reach the sluices during antecedent and succedent flows. These volumes serve as an initial estimate of the amount of material that would need to be collected in the sediment trap. The HEC-6 model results predict that approximately 4.0 ac-ft/yr of bed material will be delivered to the sluices during antecedent flows and about 8.4 ac-ft/yr will be delivered during succedent flows. The distribution of the material is also indicated in the table. For succedent flows, approximately 30 percent of the material is coarse sand and fine gravel, 65 percent is medium and coarse gravel and about 5 percent is cobbles. For antecedent conditions, approximately 4 percent is coarse sand and fine gravel, 91 percent is medium and coarse gravel, and 5 percent is cobbles. The succedent flow values are believed to be reasonably representative of conditions that would occur during periods immediately after a large flood; while the antecedent flow value should represent conditions during periods several years after passage of the flood. To minimize the size of the sediment trap, an annual maintenance program or removal of accumulated sediment is recommended.

5.3.2. Downstream Levees Element

The hydraulic, sediment transport, and bank work analyses conducted for the DLE are fundamentally dependent on the geometric data that were used to develop the hydraulic model. Uncertainties in the geometric data (specifically the locations of the cross sections surveyed in the lower 14 miles of the project reach in 1987) suggest that it will be necessary to develop a more precise hydraulic model in the Preconstruction, Engineering, and Design (PED) phase of the project. If changes to the Sacramento Weir have to be made to accommodate the greater volume of Lower American River flows, or if the weir is left in its present configuration,the downstream rating curves for the project reach will have to be modified to reflect those changes. Under these conditions it will be necessary to revisit the sediment transport based analyses, which include the bank work and bridge scour analyses, for at least the preferred alternative.

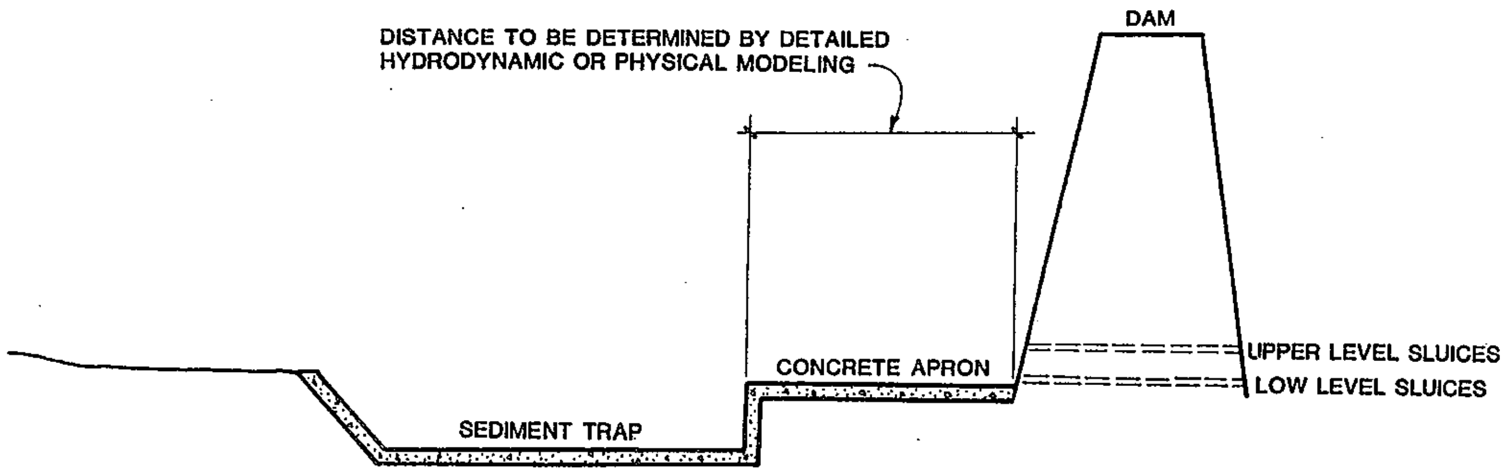


Figure 5.1. Conceptual illustration of method to minimize potential for entrainment of sediment sluice under high head conditions.

Table 5.1. Summary of Bed Material Volumes Transported to the Dry Dam Sluices for Antecedent and Succedent Conditions.		
Sediment Yield (ac-ft/yr*)		
Size Range (mm)	Antecedent Flows	Succedent Flows
1-8	0.16	2.52
8-64	3.64	5.46
64-256	0.20	0.42
TOTAL	4.00	8.40

At Type 1, there is little flexibility in choosing the appropriate form of bank protection for the levees. Rock-based revetments or dikes are the only forms of bank protection that will ensure the integrity of the levees. Experimental, biotechnically based or modified forms of protection are unproven for large rivers such as the Lower American. At Type 2 and 3 sites, it may be possible to utilize experimental forms of bank protection such as those advanced in the WRCE/MSA (1992) report and to determine their effectiveness under a wide range of flows. However, if channel or floodplain excavation alternatives are to be considered as suggested in the WRCE/MSA (1992) report, the hydraulics and related channel stability analyses will have to be revisited.

Evaluation of the levee stability analysis in the ARWI Report indicates that substantially more information and data are required to evaluate levee stability both under the existing condition scenario and the other four scenarios. Because layering in the levee foundations is important to any assessment of levee stability, it is recommended that foundation investigation borings be conducted. Additional levee and foundation configurations should be analyzed for potential seepage and piping problems. Further triaxial shear testing of soil materials should be carried out to better define the range of conditions within the levees. Stability analyses should be revised after the foundation conditions and the soil strength parameters have been verified.

The bridge scour analysis in Chapter 4 indicated significant potential for pier and abutment scour at several of the bridges in the Lower American River study reach. This potential problem is present for existing conditions with only minor changes associated with the various project scenarios.

The Federal Highway Administration (FHWA) has developed methods for protecting existing bridges from scour. These methods are discussed in FHWA (1993). The recommended method for protecting existing bridge piers from scour is illustrated in **Figure 5.2**. This method involves placement of a riprap mattress around the pier. Design guidelines for this method are as follows:

- The riprap mattress should extend a minimum of two pier widths from the face of the pier.

- Minimum thickness of the mattress is three times the median (D_{50}) riprap size.

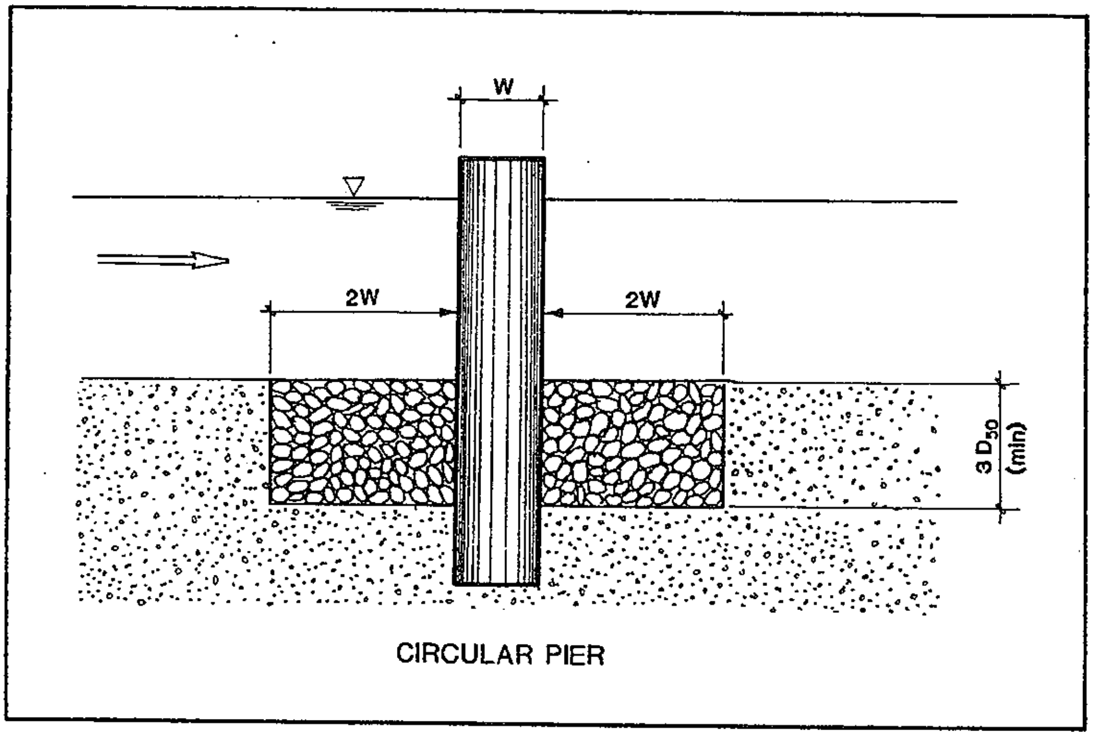


Figure 5.2. Recommend countermeasure for local scour at bridge piers.

- The bottom of the mattress should be placed as or below the computed contraction scour depth, and the top should extend up to the channel bed to facilitate inspection.

It should be noted that such measures are not considered to be a long-term solution to pier scour and that the only long-term measure prescribed by the FHWA for existing bridges is monitoring.

Figure 5.3 shows a schematic of the recommended riprap protection for abutment scour. The method essentially involves placement of a riprap blanket around the nose of the abutment. Design guidelines for the blanket are as follows:

- The riprap should cover abutment slopes and approach embankments.

- A riprap blanket should be placed on the channel bed (or floodplain for abutments on overbank area) extending horizontally a minimum of 2 flow depths or 25 ft (whichever is less) from the toe of the abutment. The flow depth used for this determination is the average flow depth in the contracted section adjacent to the abutment.

- The blanket thickness should be not less than the larger of 1.5 times the median (D_{50}) riprap size, or one times the maximum (D_{100}) size.

- If the rock riprap is placed by dumping through water, the thickness should be increased by 50 percent to provide for uncertainties associated with this type of placement.

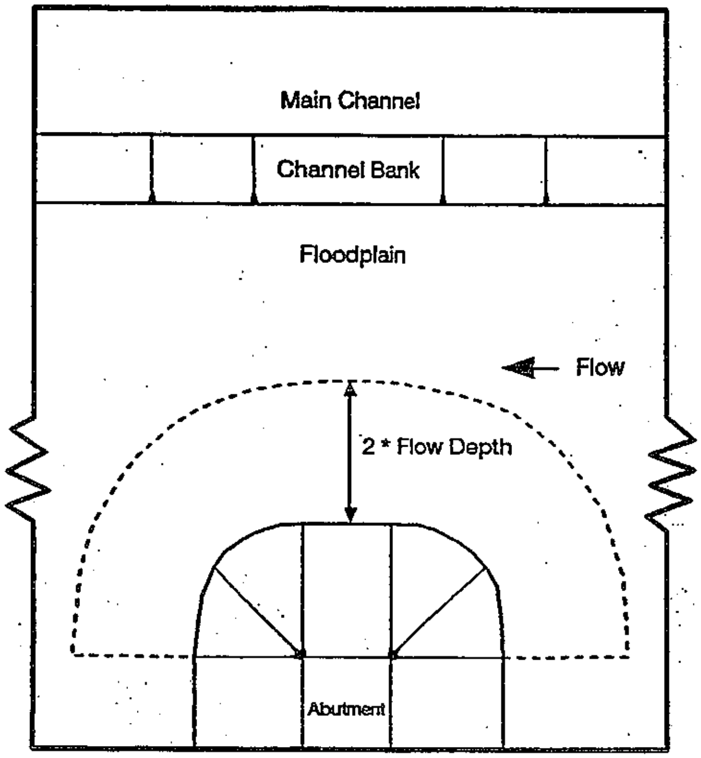


Figure 5.3. Recommended countermeasure for local scour at bridge abutments.

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7. GLOSSARY

Aggradation	- the process of building up a surface by deposition. An aggrading stream is actively building up its channel or floodplain by being supplied with more sediment load than it is capable of transporting.
Alluvial Fan	- an outspread, gently sloping mass of alluvium deposited by a stream flowing from a narrow canyon onto a plain or valley floor. Viewed from above, it has the shape of an open fan, the apex being at the valley mouth.
Alluviation	- the deposition of sediments by rivers anywhere along their courses.
Alluvium	- a general term for sedimentary deposits made by streams on river beds, floodplains and alluvial fans. The term applies to stream deposits of recent time.
Anticline	- a fold, generally convex upward, whose core contains the stratigraphically older rocks.
Backwater	- the elevation of water surface necessary to provide sufficient flow energy in a channel to respond to friction and form resistance.
Basal	- the bottom stratigraphic unit of a sedimentary series which rests on a surface of erosion.
Base Level	- the theoretical limit on lowest level at which a stream is neither aggrading or degrading.
Bed Load	- the part of a stream's load that is moved on or immediately above the stream bed, such as the larger or heavier particles rolled along the bottom; the part of the load that is not continuously in suspension or solution.
Bed Material Load	- the part of the stream's total load which comprises the material in a stream bed when it is dry. This material is usually composed of both bed load and suspended load material.
Bifurcation	- a forking or division into two branches.
Block/slab Failure	- failure by gravity of a coherent mass of material by vertical fall or topple where the underlying support has been removed (in this case, where a bank has been undercut by a stream channel).

Cenozoic - geological era that includes the Tertiary and Quaternary periods (approximately the last 65 m. yr).

Channel lag - refers to the coarsest sediments in the river that are generally located in the thalweg region.

Chute - a narrow channel through which water flows rapidly.

Clast - an individual rock; generally used to describe sedimentary deposits.

Clastic - pertaining to a rock or sediment composed principally of fragments, derived from pre-existing rocks or minerals and transported some distance from their places of origin.

Compressive Stress - deformation resulting from normal stresses pushing together material on opposite sides of a real or imaginary plane.

Concave Bank - the bank on the outside of a bend way.

Creep - the slow, imperceptible downslope movement of rock and soil particles under gravity.

Crossbedded - inclined laminations of strata that are transverse or oblique to the main plane of stratification.

Cross Section - a transect in the vertical plane oriented across a channel or some other feature.

D_{50} - the median grain size diameter. An expression of the average particle size of a sediment or rock, obtained graphically by locating the diameter associated with the midpoint of the particle-size distribution; the middlemost diameter that is larger than 50 percent of the diameters in the distribution and smaller than the other 50 percent.

Deformation - A general term for the processes of folding, faulting, shearing, compression or extension of rocks as a result of various earth forces.

Detritus - loose rock and mineral material produced by mechanical disintegration or abrasion, and removed from its place of origin.

Distal - a sedimentary deposit of fine clastics, formed far from the source area.

Distal Sediments - finer-grained sediments that are deposited farthest away from the source area for the sediments.

Erosion	- the wearing-away of soil and rock.
Failure Plane	- the surface or contact of a mass failure at which shear is initiated in soils and weathered rock.
Fanglomerate	- refers to sediments of both fluvial and debris flow origin that are deposited on alluvial fans.
Fault	- a fracture or fracture zone along which there has been displacement of the sides relative to one another parallel to the fracture.
Floodplain	- that portion of a river valley, adjacent to the channel , which is built of sediments deposited during the present regimen of the stream and is covered with water when the river overflows its banks at flood stages.
Fluvial	- of or pertaining to rivers or produced by the action of a stream or river.
Fold	- a bend in bedding, foliation, cleavage, or other planar features in rocks. A fold is usually a product of deformation.
Geomorphology	- the study of the classification, description, nature, origin and development of landforms and their relationships to underlying structures, and the history of geologic changes as recorded by these surface features.
Imbricated	- overlapping, as shingles or tiles on a roof.
Incised	- cut down into or entrenched.
Intrusive	- of or pertaining to the process and rock formed by the emplacement of molten rock material in pre-existing rock.
Lag	- the coarse-grained material that is left behind after flow has washed away the finer material.
Lacustrine	- refers to sediments that have been deposited in a lake.
Lateral Migration	- movement of a channel perpendicular to the direction of flow.
Lithification	- the conversion of a newly deposited sediment into a solid rock, involving such processes as cementation, compaction and crystallization.
Lithology	- the physical character of rocks, especially in hand specimen and in outcrop based on such characteristics as color, mineralogic composition and grain size.

Mass Failure	-	unit downslope movement of a portion of the land surface, as in creep, landslide, or slip.
Mass Wasting	-	the downslope movement of soil and rock material under the direct influence of gravity.
Mitigation	-	the act of alleviation; to render less severe.
Monocline	-	a one-limbed fold in strata which are usually flat-lying except in the fold itself.
Morphology	-	the shape of the earth's surface.
Outcrop	-	that part of a geologic formation or structure that appears at the surface of the earth; also, bedrock that is covered by surficial deposits such as alluvium.
Pavement	-	the surficial layer of coarse grained sediments that are rarely transported and which protect the underlying sediments from erosion and transport.
Planar Lamination	-	horizontally bedded strata.
Planform	-	the shape of something as viewed from above.
Pleistocene	-	the earlier of the two epochs that comprise the Quaternary period, the latter of which is the Holocene or Recent epoch.
Plutonic	-	Pertaining to igneous rocks formed at great depth.
Proximal Sediment	-	coarser-grained sediments that are deposited close to the source area for the sediments.
Point Bar	-	the depositional surface on the inside of a bend way composed of coarser grained accreted sediments.
Quaternary	-	geologic period that comprises the latter part of the Cenozoic era (approximately the last 2 m. yr).
Radius of Curvature	-	the measure of the curvature of a bend way.
Riparian	-	pertaining to or situated on the bank of a river.
Rotational Slide	-	a landslide in which shearing takes place on a well defined, curved surface, concave upward, producing a backward rotation in the displaced mass.
Scroll Bars	-	concentric sandy ridges that form on the upper point bar surface at the approximate elevation of the bankfull flow of the river. They record the lateral migration of the bend.

Sedimentary	- pertaining to or containing solid fragmental material, or formed by its deposition into layers in loose unconsolidated form after being transported by wind, water or ice.
Sediment Transport Capacity	- the capability of a channel to carry a given volume of sediment based on local hydraulic and geometric parameters.
Seismic	- pertaining to an earthquake or earth vibration which may be natural or artificial.
Sigmoid	- Curved like the letter S.
Sinuosity	- the measure of how straight or curved a channel is. It is the ratio of its actual channel length to the straight-line distance down valley.
Slickens	- the fine grained component of hydraulic mining-derived sediment that comprised the initial sediment surge into the Sacramento Valley in the late 1800s. The slickens are thinly-bedded silt, clay and fine sand deposits which are generally low in organic content and resistant to erosion. The term is taken from Gilbert (1917).
Slump	- the downward slipping of a mass of rock or unconsolidated material, moving as a unit, usually with backward rotation on a more or less horizontal axis parallel to the slope from which it descends.
Splay	- a lobate sedimentary deposit on the floodplain that originates from a breach in the natural levee.
Strath Terrace	- a surface cut on bedrock along a valley floor representing a local base level, the top of which has been formed primarily by lateral erosion and usually is covered by a veneer of alluvium.
Stratigraphy	- the arrangement of sedimentary layers or units based on geographic position and chronologic order of sequence.
Strike-Slip	- the component of the movement or slip that is parallel with the direction of a fault.
Structural Trough	- a long, narrow depression in the earth's surface created as a result of folding, faulting or other deformation.
Subpavement	- those sediments lying below and protected from erosion by an overlying or surficial layer such as gravel or cobbles.

Suspended Sediment - those sediments that are part of the total stream load that is carried for a considerable period of time in suspension, free from contact with the stream bed; it consists mainly of clay and silt.

Syncline - a fold, generally concave upward, whose core contains the stratigraphically younger rocks.

Tectonic - of, or pertaining to rock structure and external forms resulting from deformation of the earth's crust.

Terrace - a relatively level bench or steplike surface breaking the continuity of a slope.

Thrust Fault - a fault with a dip of 45° or less over much of its extent, on which the overlying side of the fault appears to have moved upward relative to the lower side of the fault.

Unconformable - strata that do not succeed the underlying rocks in immediate order of age or in parallel position.

Vertical Accretion - growth of a sedimentary deposit by upward deposition.

Wolman Count - a fixed interval method of field sampling of the coarse bed and bar materials using a template that is calibrated in the same intervals as standard laboratory sieves (from 2 to 256 millimeters).

APPENDIX A

Review of Mitchell Swanson and Associates
and
WRC - Environmental

Review of

"A Review of Key Issues from the
American River Watershed Investigation
Feasibility Study
and
An Analysis of Alternatives for
Increasing Flood Protection in Sacramento"

By: Mitchell Swanson and Associates & WRC - Environmental

Introduction

In February 1986, record flood flows occurred in the American River Basin. Recent studies have concluded that large floods may occur on the American River more often than previously estimated and that the level of protection provided by the existing flood control system is significantly less than a 100-year event. Because of the potential for loss of life and extreme property damage, the U.S. Army Corps of Engineers, Sacramento District, and the State of California Department of Water Resources, Reclamation Board prepared the American River Watershed Investigation (ARWI) Feasibility Report (COE 1991). The report was published in December 1991. In March 1992, WRC - Environmental and Mitchell Swanson and Associates submitted a review of ARWI (MSA/WRC, 1992) to the Sacramento Area Flood Control Agency. The Swanson/WRC report was intended "To provide and independent review of the technical data, assumptions, and the environmental impact analysis developed in the American River Watershed Investigation (ARWI) for the recommended 200-year dry dam at Auburn.... The purpose of this work is to analyze the critical issues surrounding the ARWI plan formulation, objectively evaluate the merit of some of the criticisms that the ARWI plan has drawn, and identify alternative approaches to increase the level of flood protection provided to people and property occupying the American River floodplain." This review, performed by Resource Consultants & Engineers, Inc. (RCE), is similar in nature, and is intended to evaluate the assumptions, methods and conclusions of the Swanson/WRC report.

The following review will identify the report section, page number(s), the quoted or paraphrased statements in the Swanson/WRC report, and the review comments by RCE.

Section: Methods and Scope

Pg. 3: Statement: Folsom Dam and Auburn dry dam operations were evaluated using spreadsheets.

Comment: Reservoir routing and operations should be analyzed using accepted hydrologic computer programs such as HEC-1 or HEC-5, especially when the conjunctive operations of dams are being simulated.

Section: Existing Conditions

Pg. 6: Statement: The Swanson/WRC report questions the use of Expected Probability Estimate (EPE), stating that the American River has a long flow record (87 years) for western U.S. rivers. The report suggests that statistical uncertainty should be dealt with by adding factors of safety (such as adding freeboard) and assessing risk.

Comment: Use of the EPE is a method of dealing with statistical uncertainty. For large sample populations, the computed and expected probabilities should be the same. For small sample populations, uncertainty is greater and is accounted for with the EPE. Freeboard can be used to deal with statistical uncertainty if the required freeboard is related to statistical calculations. Traditionally, freeboard is a method of dealing with uncertainty associated with hydraulic calculations. While an 87-year flow record is long for the western U.S., it is still short in comparison to the full flow history of the river. Also, while the Water Resources Council Bulletin 17B (WRC, 1981) does not recommend the use of EPE, that's because no recommendation is made; the decision is left up to the individual federal agency.

Pg. 7: Statement: The Swanson/WRC report suggests that by adjusting the adopted skew coefficient to 0.0, that the statistical distribution is changed from the recommended log-Pearson III distribution to a log-normal distribution.

Comment: Correctly stated, a log-Pearson III distribution with 0.0 skew coefficient is the same as a log-normal distribution. When the computed skew differs from the generalized (regional) skew, the WRC Bulletin 17B recommends adopting an intermediate skew. Corps use of a 0.0 skew coefficient may well be warranted because the generalized skew at the gage in question is 0.0 and increases to the northwest where the flood flows originate.

Pg. 13: Statement: The Swanson/WRC reports states that There is a reluctance by the Corps to raise releases above 20,000 and 50,000 cfs because of possible damage to public facilities in the floodway of the Lower American River Parkway.

Comment: No comparative analysis is presented in the Swanson/WRC report to show whether there would be significant additional losses due to the recommended more frequent releases in the 20,000 to 50,000 cfs range.

Pgs. 14-17: Statement: The Swanson/WRC report recommends improvements to Folsom Dam and its operations, including lowering of the spillway so higher releases may be made earlier in an event.

Comment: Lowering the spillway allows the reservoir to be operated with less encroachment into surcharge storage. While this is a benefit, a maximum release of 115,000 cfs is still projected for approximately 6 days under either scenario. Therefore, there is probably no difference in downstream flooding.

Pg. 21: Statement: "Bank protection works and rip-rap appear to be undersized ... (the armoring rock appears small) for the forces present in the river."

Comment: No calculations are presented in the Swanson/WRC report on what the true forces are or what the required rock size should be.

Pg. 22: Statement: Flow angle of attack at banks is often 90 degrees.

Comment: The figure (Figure 9) which supposedly illustrates a 90 degree angle of attack doesn't. A 30 degree angle of attach may be justified from this figure.

Pgs. 23: Statement: The levees should not be considered stable for extended flows of 115,000 cfs because the levees nearly failed during 2.5 days of flow between 115,000 and 130,000 cfs.

Comment: The fact that the levees didn't fail for flows exceeding 115,000 cfs is an argument for stability at 115,000 cfs.

Pg. 23: Statement: "... the fact that the hydraulic forces in the American River at discharges of 115,000 to 130,000 cfs (and up to 150,000 cfs) are essentially the same...."

Comment: This statement appears in the Swanson/WRC report frequently. According to the data presented in the Swanson/WRC report, velocity increases by nearly 8 percent when the discharge increases from 115,000 to 130,000 cfs. The likely increase in sediment transport capacity for this increase in velocity is approximately 36 percent. If erosion is a levee stability issue for the American River, then a 36 percent increase in sediment transport potential is not essentially the same.

Pg. 23: Statement "However, there are more complex but very important hydraulic forces observable through geomorphic analysis that can increase shear stress dramatically, but defy common methods of hydraulic modeling and quantitative analysis (such as HEC-2 and other engineering hydraulic analyses) that are often used in conventional bank protection design. The hydraulic forces are increased dramatically (reportedly up to 15 times) by secondary currents in channel bends, instantaneous bursting of turbulent currents, high angles of flow impingement on the banks, and the formation of eddies."

Comment: While the equations used for riprap design may not explicitly include relationships for turbulence etc., they were developed using data collected under turbulent conditions. These equations also include factors of safety and adjustments for angles of attack and flow deflection.

Pg. 35: Statement: "First, lateral erosion is treated on a piecemeal basis as a maintenance problem, not a design problem.

Comment: The Corps does not choose to treat lateral erosion as solely a maintenance problem, it is imposed on them.

Pg. 39: Statement: "Finally, there or no factors of safety built into the design of the present levee system."

Comment: This is quite simply untrue. There is freeboard above the projected design flow elevations and levee designs must include factors of safety in the design.

Pg. 40: Statement: "The discussion above demonstrates that the hydraulic differences are not great enough to preclude higher objective releases. For all intents and purposes, the criteria for stable levees are essentially the same at 115,000 cfs as they are for 130,000 cfs and up to 150,000 cfs. Figure 13 shows the differences in flow depth at these discharges, and Figure 12 shows the differences in elevation between the 115,000 cfs water surface and others up to 180,000 cfs and the levee elevations. The maximum water surface elevation difference between 115,000 and 155,000 cfs is 4 feet at the Mayhew Drain in Rancho Cordova, 3 feet at Howe Avenue, 2.5 feet at Business 80 and 1 foot at 16th Street. All of these discharges are in the same hydraulic ballpark and the incremental differences between constructing the needed reliability at 115,000 cfs and 150,000 cfs is not great."

Comment: The discharges may be "in the same hydraulic ballpark", but there would be significant increases in sediment transport as discharge increases and the required levee improvements for higher discharges would not be trivial.

Section: Plan Formulation

Pg. 65: Statement: "Designed widening of the channel cross sectional area by the creation of low terraces and enhanced secondary channel can reduce the overall channel velocities and reduce the rate of sediment flushing and channel degradation."

Comment: For this proposal to be implemented, the design must account for the degradation of the channel bed downstream of the widened reach.

Pg. 65: Statement: "Finally, for those project areas that really are in need of riprap protection, there are a variety of alternative designs that will allow for the establishment of riparian and upland tree vegetation. Not only will this vegetation provide for the preservation or enhancement (in the case of upgrading existing riprap and levees) of lower American River resource values, but it will serve to more thoroughly protect the levees and riprap areas by creating slower water velocities along the banks and levees (see Figures 20 through 27)."

Comment: If the reduced velocity along a reach is not accompanied by a proportional increase in cross section area, the resulting higher water surface will necessitate levee improvements.

Pg. 74: Statement: "Within the context of the floodflows that are expected to occur as a result of the Selected Plan, there will be greater occurrences and duration of high flows that can transport coarse sediment than would be the case with shorter duration higher objective release flows. This results in the conclusion that higher objective releases would actually reduce the rate of long term coarse sediment transport."

Comment: Increased flows do not generally result in decreased sediment yields. Some very convincing analysis would be required before this statement could be justified.

Pg. 75: Statement: "Loss of stream energies at these locations leads to the deposition of materials and, in the case of the areas just above and below Watt Avenue bridge, these aspects are so predominant that downstream transport of coarse sediment is totally interrupted."

Comment: This is another unfounded statement, probably based on the existence of gravel bars at this location. If gravel deposition is occurring in this reach, the expected hydraulic response is to increase velocity and energy, such that gravel transport would become reestablished.

Pgs. 81-83: Statement: The Swanson/WRC report suggests that slope instability at the proposed Auburn dry dam site is under predicted. Part of the justification is an analysis of landslides resulting from a 1986 coffer dam failure.

Comment: The coffer dam comparison may not be comparable in terms of failure mechanism due to different draw down rates. Also, draw down rates could be controlled using a gated outlet.

Pg. 91: Statement: "While it probably is true that landsliding is an important canyon forming process along the American River, the rate of regional uplift is extremely slow and correspondingly, active natural landslides are rare along all the lower mainstem canyons of the Sierra. There may be varying areas of landslide deposits on the walls of these mainstem canyons but under the influences of typical slope forming processes, with few exceptions, they are rarely active and have well developed soils and vegetation."

Comment: How can it be true that landsliding is an important canyon forming process and that active natural landslides are rare in the area.

Pg. 115: Statement: The Swanson/WRC report argues that frequent inundation upstream of the Auburn dry dam will result in excessive sediment accumulation in the area. The report states: "Therefore at the elevation of, and the channel location of, the 10 year recurrence floodflow there would be 111 occurrences of inundation pool flooding in a 200 year period, or once every other year there will be a cycle of channel inundation and sediment accumulation at that location."

Comment: The Swanson/WRC analysis of flood recurrence uses a non standard method for calculating flood frequency. Specifically, in a 200-year period there will be 1 200-year event, 2 100-year events, 4 50-year events, 6 33.3-year events, 8 25-year events, 10 20-year events and so on to 100 2-year events. They conclude that every other year (111 times in 200 years) there will be a 10-year flood or greater. Since flood frequency analysis generally is interpreted that frequency is associated with the exceedance of a given event, one can actually expect that in a 200 year period, there will be 20 events of a 10-year or greater magnitude. Therefore, the conclusions drawn from the Swanson/WRC flood recurrence analysis of the Auburn dry dam site are unfounded.

Section: Alternative Analysis

Pg. 123: Statement: "The review of existing information and the plan formulation process places new light on the selection and analysis of alternatives for upgrading flood protection in Sacramento, principally due to two factors: (1) The reliability of the American River levees in their present condition, and (2) a potentially higher risk of environmental damage and operational impacts in the North Fork and Middle Fork Canyons with a dry Auburn Dam. With these new factors in mind, an alternative analysis was compiled that focuses on levee improvements and upgrading Folsom Dam as priorities, and seeks ways to minimize the environmental impacts of any project to the North Fork and Middle Fork Canyons as well as the Lower American River Parkway and Folsom Reservoir."

Comment: The Swanson/WRC report focuses much attention on the unreliability of the existing levees, yet goes on to recommend levee improvements in conjunction with higher releases from Folsom Dam. This approach puts even greater faith in the soundness of levees along the American River. The statement that there is potentially higher risk of environmental damage due to Auburn dry dam is not founded on appropriate science.

Section: Conclusions and Recommendations

Pg. 135: Statement: "It is essential to upgrade the levees in any plan. It appears that the increment of effort required to upgrade the levees from 115,000 cfs to 180,000 cfs with 3-feet of freeboard does not appear large. Therefore, the levee upgrade plans should go beyond those needed to contain 115,000 cfs and should attain the maximum capacity and reliability available."

Comment: It would be far from a trivial matter to upgrade the levees from 115,000 cfs to 180,000 cfs with 3-feet of freeboard. Three feet of freeboard may also be a somewhat arbitrary height and may be inadequate given the uncertainties in every aspect of the design.

REFERENCES

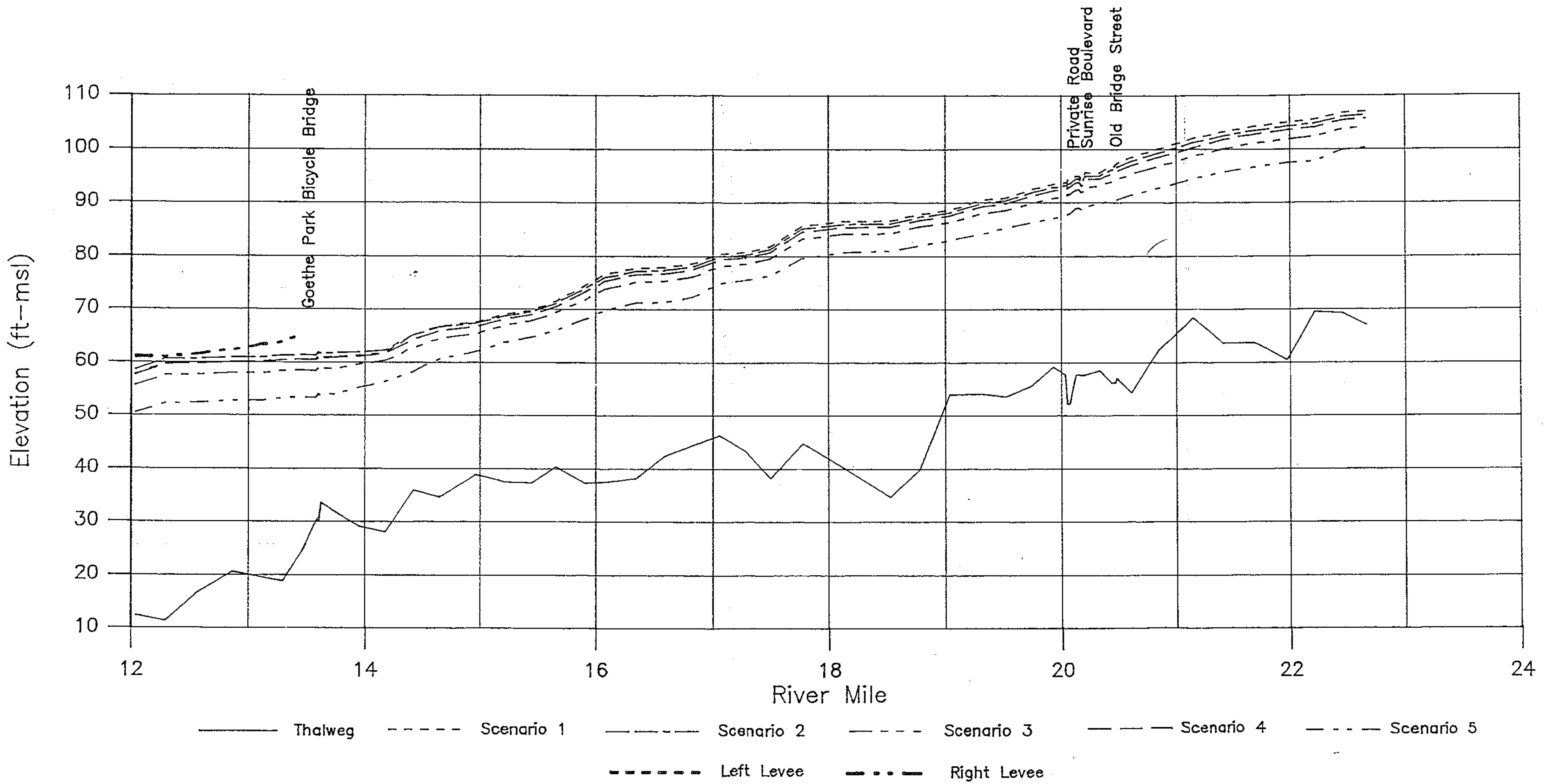
Mitchell Swanson and Associates and WRC Environmental, 1992. A Review of Key Issues from the American River Watershed Investigation Feasibility Study and An Analysis of Alternatives for Increasing Flood Protection in Sacramento.

U.S. Army Corps of Engineers, 1991. The American River Watershed Investigation Feasibility Analysis (ARWI) and Environmental Impact Statement, unpublished report with appendices, dated December 1991.

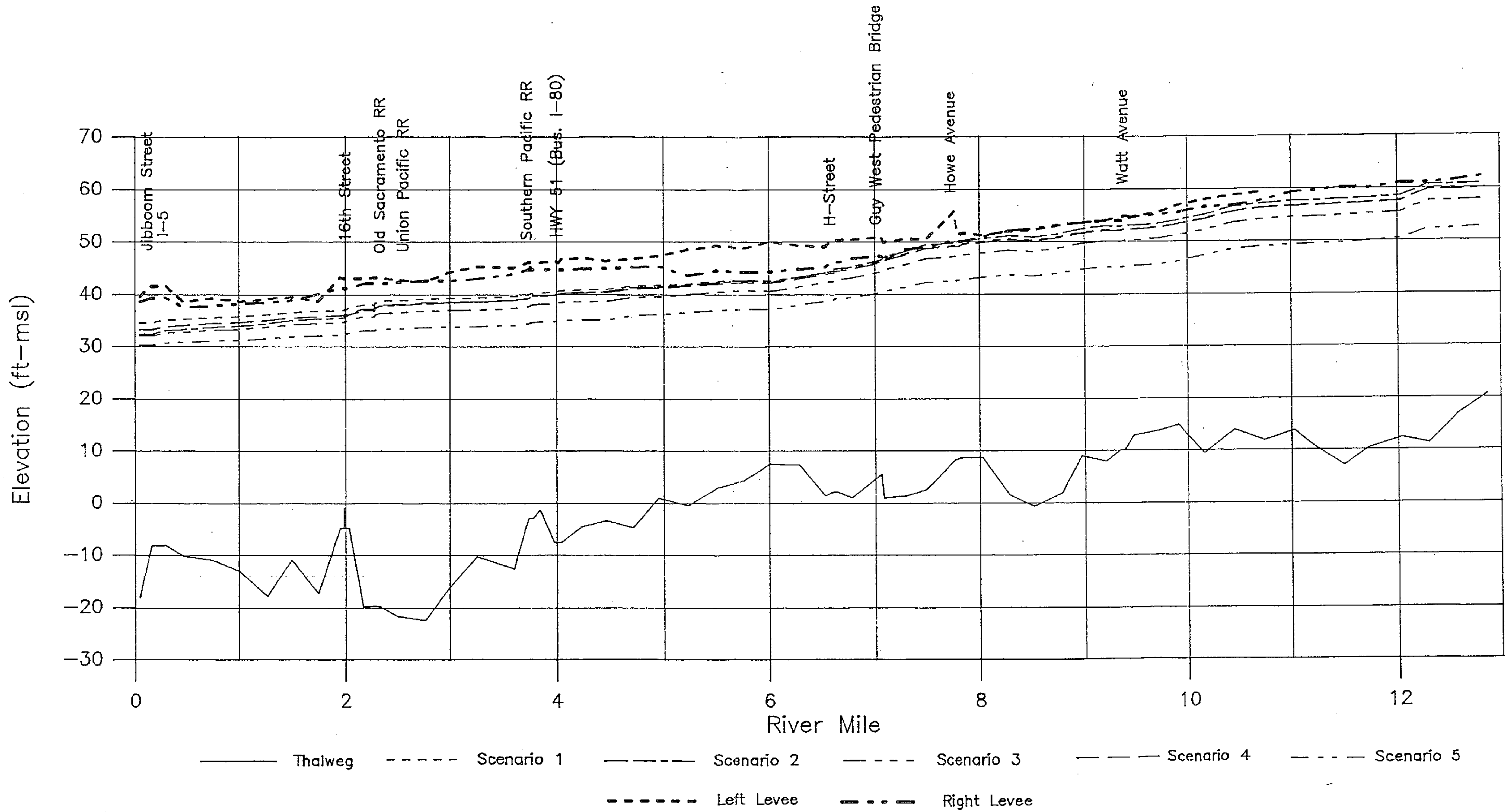
U.S. Water Resources Council, 1981. Guidelines for Determining Flood Flow Frequency, Bulletin No. 17B of the Hydrology Committee.

APPENDIX B

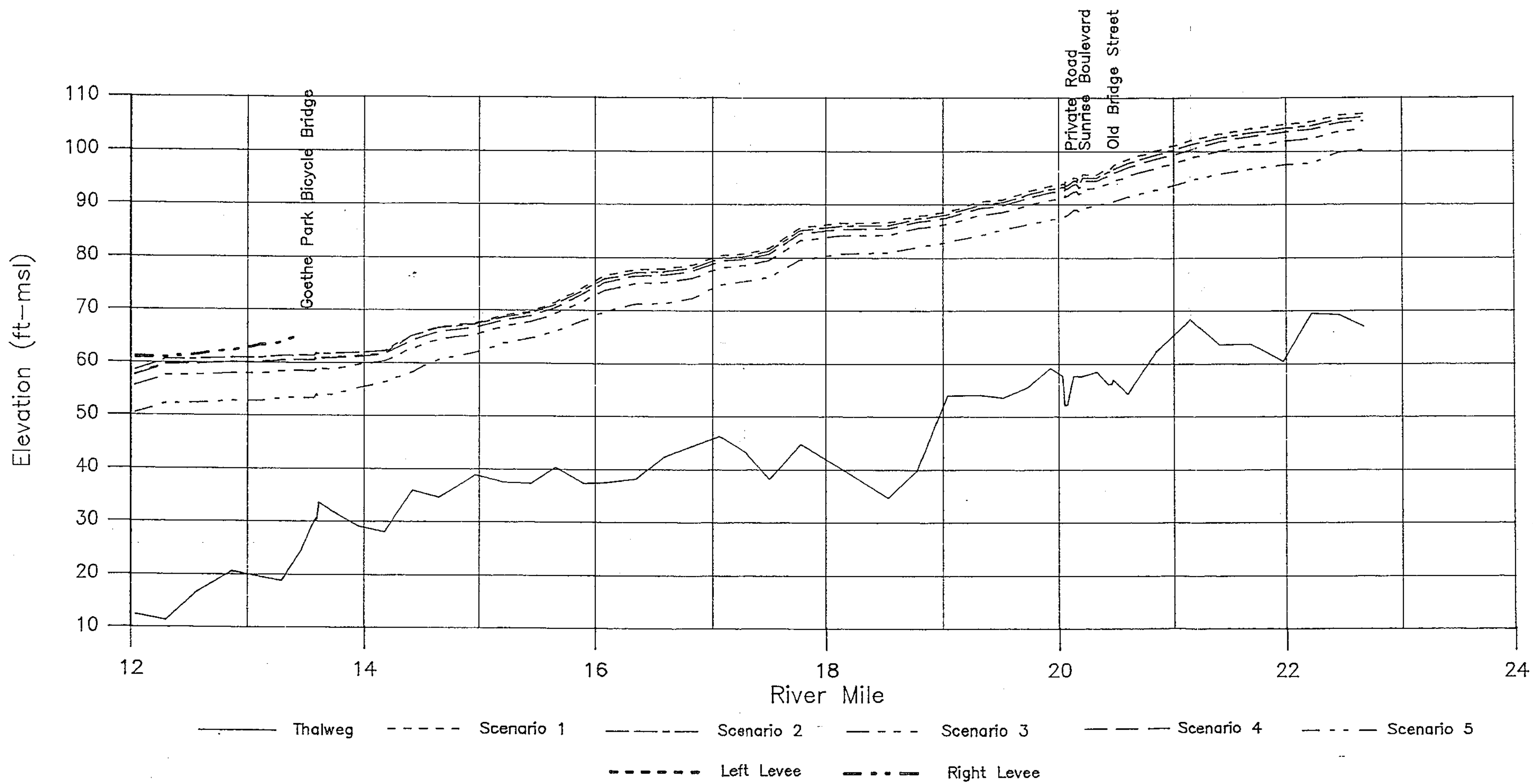
Water Surface and Levee Profiles, 100-Year Flood Lower American River



Water surface and levee profiles,
100 year flood Lower American River.



Water surface and levee profiles,
100 year flood Lower American River.



Water surface and levee profiles,
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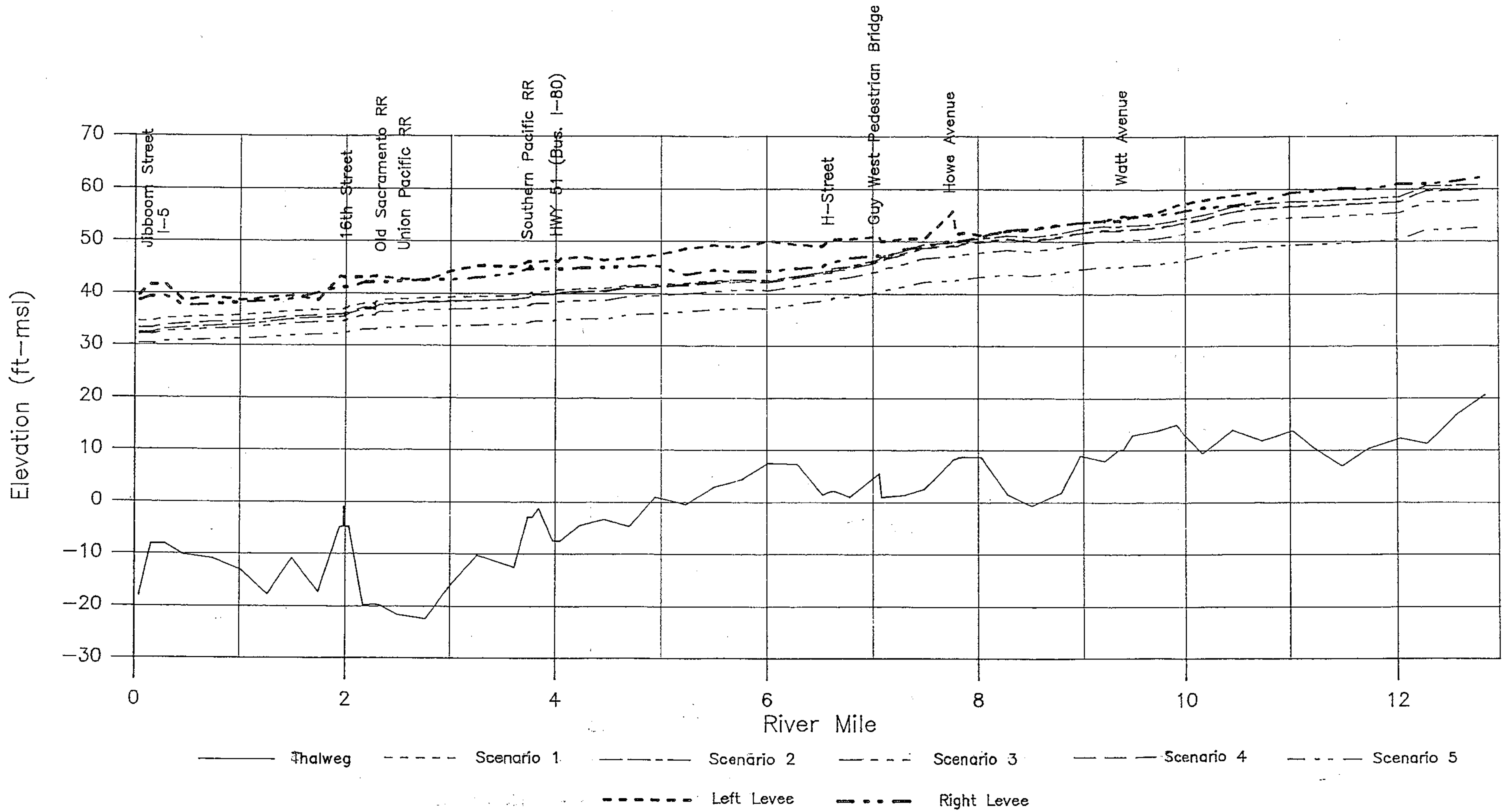

Water surface and levee profiles, 100 year flood Lower American River.